THE INFLUENCE OF CFRP ON THE BEHAVIOR OF RE-INFORCED CONCRETE SUBJECTED TO BUCKLING

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ABSTRACT: The most useful method of strengthening reinforced concrete columns consists of wrapping the columns with a Carbon Fiber Reinforced Polymer CFRP compared to others various methods such as retrofitting, filling new reinforced concrete or jacketing welded metal sheets around the column. This method proves to be fast and effective.

Strengthening using carbon fiber fabric plays an active role in flexural resistance (increase of the stiffness) particularly when the fibers are glued parallel to the longitudinal axis of the column [Verok 2005]. In the case of perpendicular gluing of CFRP, the resistance is considered to be a passive. Therefore, the creation of lateral confinement occurs which prevents a concrete from local swelling and bulging out.

For both cases, numerous studies have been reported the behavior of circular-columns subjected to a compressive loading [Chaallal & Shahawy 1999, Lorenzis 2001, Sause & Harries 2004]. However, limited studies on the behaviour of rectangular columns under the same type of loading have been published [Chaallal & Shahawy 2000]. Few investigations have considered the interaction of axial load and biaxial bending on the rectangular columns.

In this context, an experimental study of the behaviour of sixteen confined slender reinforced concrete columns have been carried out. The results of only four columns with slenderness ratio of $l/h=22.8$ ($l/r\cong79$) are presented in this paper. To highlight the influence of the confinement rate on the structural behaviour of the columns, the number of layers wrapped around the column was varied.

The columns were subjected to vertical compression loads with doubly eccentricity. The results of this study confirm that the increase in the confinement rate increased the strength capacity of the column, as well as its ductility. The experiments also show that the effect of CFRP confinement on the increase in slender reinforced columns strength is limited.

1 INTRODUCTION

The buckling of slender columns is a recurrent instability problem that can be found in seismic zones. In order to rectify the size dimensions of reinforced concrete columns due to design errors, the implementation of strengthening techniques is possible. Traditionally, column strengthening is carried out through jacketing using adapted metal sheets, or even by caging the old column in a new enlarged concrete column. Currently, the strengthening technique for RC structures using CFRP wrapping has proven to be a more reliable solution, and offer a good flexibility and simplicity.

Strengthening using CFRP plays an active role in flexural resistance (increase of the stiffness) particularly when the fibers are glued parallel to the longitudinal axis of the column In the case of perpendicular gluing of the fiber, the resistance is considered to be passive [Verok 2005]. Therefore, the creation of lateral confinement occurs which prevents a concrete from local swelling and bulging out. For both cases, numerous studies have been carried out on the behavior of circular-columns [Hadi 2005] subjected to compressive loading. However, current cases of columns, are subjected to a doubly eccentric compression, not to mention that the most used sections of columns are rectangular or square [Chaallal & Shahawy 2000].

The objective of this study is to evaluate the behavior of the CFRP confined reinforced concrete slender columns with square section (the scale is reduced to the 1/3). Three levels of confinement were set up (1 to 3 layers of CFRP), and the columns were subjected to the effect of doubly eccentric compression loading. Moreover, the results obtained from four columns, having the same mechanical and geometrical characteristics except for the number of CFRP layers, are presented.

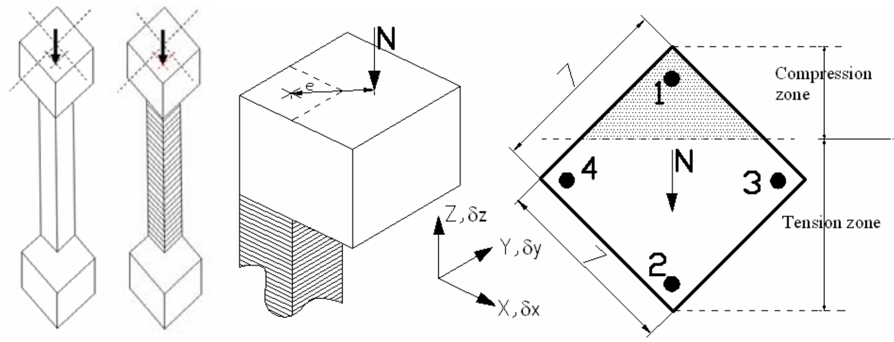
2 EXPERIMENTATION

2.1 *Fabrication of the specimens*

To respect the scale ratio (reduction to the 1/3) during the fabrication of the specimens, micro concrete with W/C = 0.6 was used. The mix composition is: 240 kg/m^3 of cement, 860 kg/m^3 of sand and 865 kg/m^3 of gravel (diameter <4mm). The concrete compressive strength was 29 MPa and the yield stress of steel was 500 MPa. The longitudinal reinforcements were made with steel bars T4 having 4-mm diameter, and the shear stirrups using ordinary wire of 2-mm diameter with spacing of ten centimeter.

The mixing procedure, the casting and the curing of the concrete were kept similar for all the specimens. After hardening of the concrete, the mechanical cleaning for the four faces of the columns was done to ensure the maximum adherence of the CFRP with concrete, and the angles are slightly rounded to avoid a rupture of the reinforcement by angular tear. SIKA France provided the one-way carbon fiber fabric (SikaWrap 230C) having tensile strength of 4300MPa, a tension elasticity modulus of 238 GPa and an ultimate elongation of 1.8% and the epoxy adhesive (SIKADUR 330) having tensile strength of 30MPa, a tension elasticity modulus of 4.5 GPa and an ultimate elongation of 0.9%.

The four columns tested had a length of 1600mm, and the square cross section was 70x70mm. The geometrical slenderness ratio was l/h =22.8 (l/r≅79). The ends of columns were casted with 200x200x150 mm bumps to avoid punching and to make the application of the biaxial load easier (Fig.1). The columns pot7c0, pot7c1, pot7c2 and pot7c3 had 0, 1, 2 and 3 layers of CFRP, respectively, and the eccentricity of the load was 51.21mm.



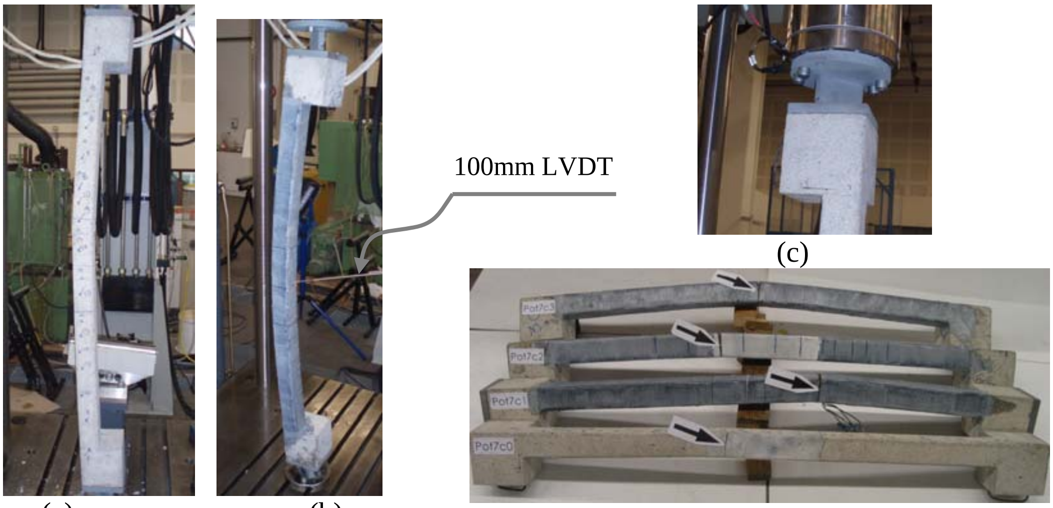
(a) (b) (c) (d)

Figure 1. (a) column geometry, (b) Confined column, (c) Application of the load, (d) column cross section

Experimental installation

The tests have been conducted on 4 columns using ZWICK traction-compression-torsion controlled machine (displacement or force).

The top and the bottom of the columns are fitted with a device allowing a desired eccentric position of load and able to direct the inflection of columns. The specific device was designed to ensure hinged end conditions. To obtain the complete ascending and descending branches of biaxially load deflection curves (axial and transverse) N-δz and N-δt, the load was controlled by displacement. One 100-mm LVDT was used to measure the transverse deflection δt at mid height of the column. δt is also calculted with the data obtained from two LVDT in X and Y axis ($\delta t=(\delta x^2+\delta y^2)^{1/2}$).



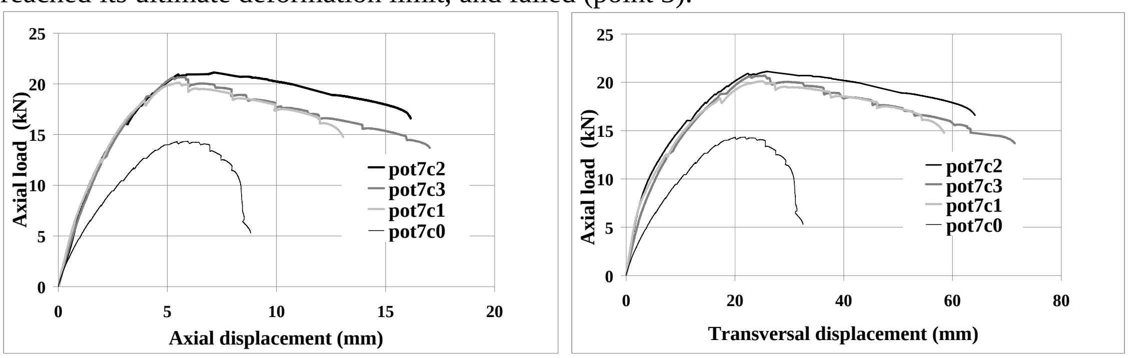
(a) (b) (c) (d)

Figure 3. (a) Unconfined column buckling, (b) Confined column buckling, (c) Top mechanism of loading, (d) Columns major crack positions after test (at the arrowhead).

3 TESTS RESULTS:

3.1 *Behavior of the columns*

Fig. 3 shows the four monotonous curves obtained during the tests. Typical response in terms of force and displacement is composed of three branches Fig. 4. The first one [0-1] was a straight segment representing the elastic phase of the column, which concrete and steel reinforcements became deformed in the elastic stage. The nucleation of the first crack indicates the end of this stage with the first inflection of the curve (point 1 in Fig. 4). The second stage (phase [1-2] Fig. 4) is characterized by a decreasing slope curve according to the increase of load (Figs. 3 and 4). The damages by loss of both bending and compression stiffness are observed, therefore the cracking of concrete in the tension zone is propagated from the center to the top and the bottom of the column. The pattern of cracks is located every ten centimeters. The end of this stage is characterized by the maximum load and the second inflection of the curve (point 2 in Fig.4). The post peak behavior (phase [2-3] Fig. 4) is the third stage. In this level of loading, the shapes of the curves present a difference following the column confinement degree. Indeed, in an unconfined column case, fast decreasing of the load with the increase in displacements is noted, and the tension zone cracks appear more and more quickly. The neutral axis is then pushed more towards the compressed zone, then the compressed concrete area decreases. At this stage, the ultimate deformation ε_{cu} was reached and the column collapsed. At this time the compressed concrete is crashed. Also, it can be observed that the steel reinforcement in tension reached its ultimate deformation limit, and failed (point 3).



(a) (b)

Figure 3. Load according to (a) axial displacement δz (b) transversal displacement curves δt.

In the case of the confined columns the phenomenon of damage is different; in Fig. 3 the slope of this branch is quasi constant, compared to the unconfined column, the load decreases slowly with the increase in axial and transverse displacements. When the compressed concrete reaches its limit in compression, contrary to the unconfined column, it is not rejected and remains in place retained by confinement and is opposed to its swelling. Thus column strength is enhanced. The neutral axis advances less quickly towards the compressed zone, deformation capacity is larger. The total collapse occurs when the tension steel reinforcement is broken. After this the compressed concrete remains in place, the failure in tension of some CFRP fibers in the compressed zone is noted. The results of an another study carried out on 100x100x200mm confined concrete columns axially loaded, showed that the compressive strength increases considerably according to the increase in the number of layers of CFRP, Fig.5.

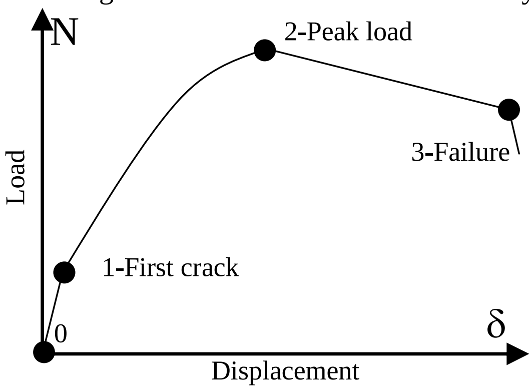


Figure 4. Column behavior

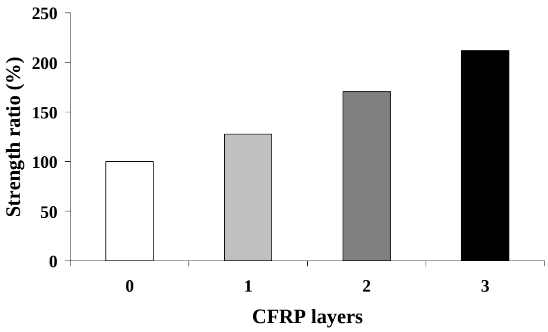


Figure 5. 10x10x20cm axially loaded strength columns according to CFRP layers number

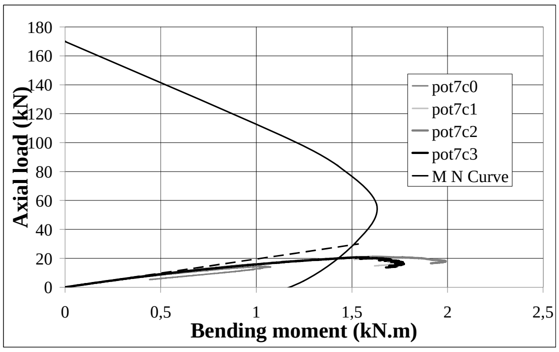


Figure 6. Load-Bending moment curves

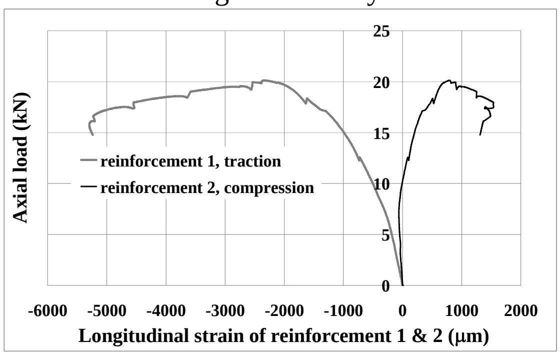


Figure 7. Load according to tension and compression steel reinforcements

Fig. 3 shows that when the peak load of the unconfined column is 14kN, the confined columns peak load is close to 20kN, the increase limit is about 30%, no matter the number of layers. This increase is small compared to that noted for the 100x100x200mm axially load columns, 27% for one layer and 70% for 2 layers, and continually increases with the number of CFRP layers, Fig. 5. An important conclusion is: the strength capacity of the biaxially loaded columns tested is affected very little by the number of layers of CFRP, moreover, it is noted that confinement plays a role in strength increasing under eccentric loading compression. The confined columns N-δ curves are almost similar; however they are different from the unconfined column curve. The initial modulus of N-δ curves is defined as the origin slope of those curves. An appreciable increase of initial bending modulus between unconfined and confined column is shown in Fig. 3; it passes from 1.95kN/mm to 3.40kN/mm (75% of increase), the compression initial modulus passes from 7.17kN/mm to 8.63kN/mm (20% of increase). One is forced to note that the CFRP contribution in enhancing the rigidity in spite of orientation of fibers transversally to the axes of columns. The bending moment of the column median section is calculated from M=N*(e+δt), where N is the load, e the initial eccentricity and δt the experimental measured transverse displacement of the median section. All curves of variation of the bending moment according to the load, Fig.6, are located in the zone lower at the balanced state, the rupture by buckling instability is noted [CEB/FIP, Cranston 1972]. The unconfined column reached its rupture far from the interaction N-M curve whereas those of the confined columns approach, one could say that the CFRP played a palliative part for this ruin premature defect.

Principal experimental N-δ curves values corresponding to the three point 1 to 3 Fig.4, are represented by bar charts in Fig.8. It can be noted, that the confined columns have higher ductility than that of the unconfined column Fig.8a and Fig.8b; this ductility is as high as the number of confinement layers, which is confirmed by the third branch length of the curves N-δ. Adopting the principle of the stored energy represented by surfaces under the curves load-displacements, to calculate ductility give increases exceeding 300%. Fig.8c shows that there is an increase in compressive strength according to the number of layers, although it may be of little importance; Fig.8d shows a gain in bending moment compared to the unconfined column, beyond the peak this moment becomes nearly constant. After cracking the tension reinforcement steel is the only tension zone resistant material. Confinement increased the resistance of the compressed concrete which generated an increase in the compression capacity, however it had little influence on the flexural strength in post peak see Fig.9.

3.2 ***Behavior of reinforcement steel***

To measure the steel deformations ε_s, the four steel reinforcement bars of column pot7c1 are fitted with deformation gages. The obtained N-ε_s curves have the same shape as the curves N-δ. They present the same branches and characteristic points. The first of which is elastic stopped at the inflection point of coordinates: N $\cong$ 6kN and $\varepsilon_{acier\ tendu}$=226µm/m. The tension reinforcement peak deformation corresponds to 2360 µm/m, which is close to used steels peak. Thus, the peak of the N-δ curve corresponds to the yielding. The compressed reinforcement steel has, at this moment, reached deformation of 814µm/m. While basing itself on the assumption of perfect adherence between steel reinforcement and concrete, the compression strain concrete is far from its ultimate strain ε_{cu}. At the column rupture, point 3 of the curve N-δ in Fig.4, the tension reinforcement steel is broken with a deformation of 5000µm/m, whereas, the strain reaches only 1600µm/m. For the particular case of the unconfined column, the buckling of the compression reinforcement steel before the rupture of the tension reinforcement is noted.

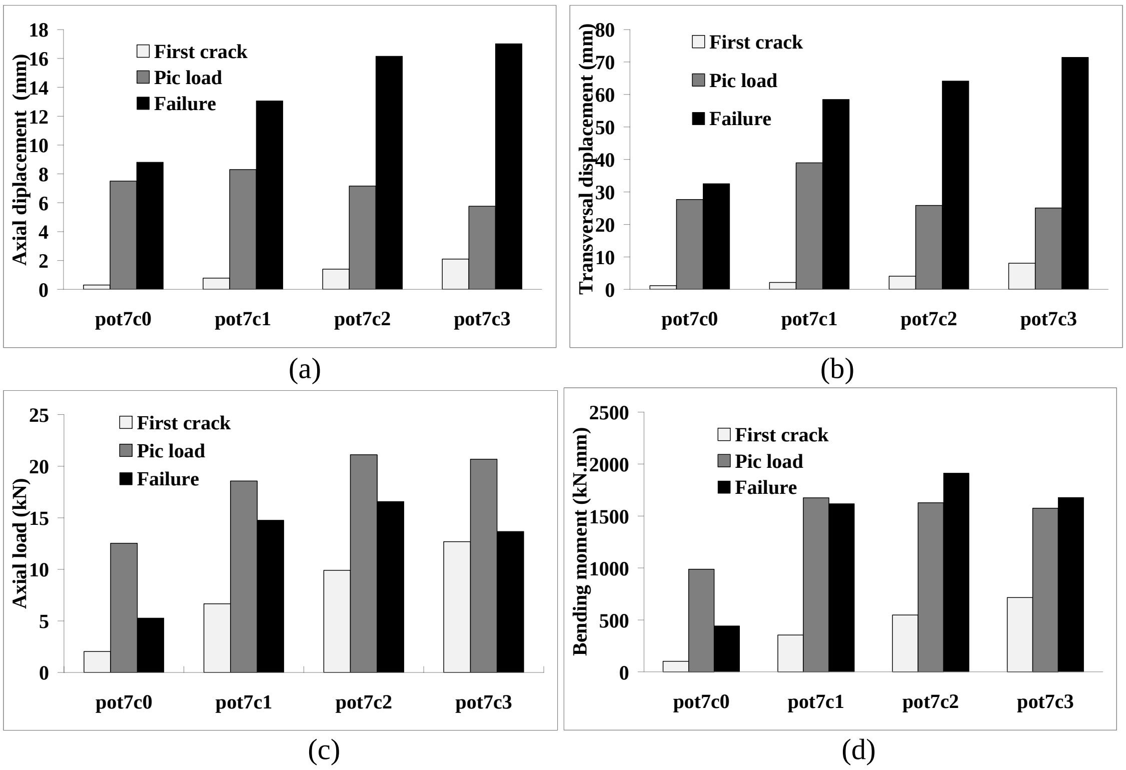


Figure 8. Experimental δ_z (a), δ_t (b), N(c) and bending moment M (d) values at points 1, 2 et 3.

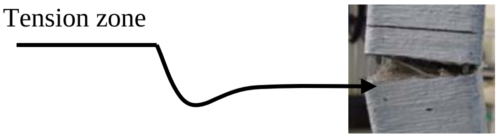


Fig.9. typical major crack column

4 CONCLUSION

The four columns presented in this work, with geometrical slenderness ratio l/h=24.28, have a beneficial CFRP confinement effect, which consists of:

- The considerable increase of resistance with a gain of 30%.
- The contribution of 75% in initial flexural rigidity and of 20% for compression rigidity.

Variation of the number of CFRP layers, from 1 to 3, did not generate a noticeable resistance gain; indeed, values of the ultimate loads of the three confined columns are very close. There is a limit for the rate of confinement beyond which it seems there is low influence on ultimate resistance; in our case one layer of CFRP is enough to develop the maximum confinement possible. The relation between this limit and the geometry of the columns is questionable. Moreover the number of CFRP layers seems an important component in ductility enhancing. Indeed, it increases as the number of layers also increases. The rupture of the unconfined column occurred by crushing of the compressed concrete, whereas, the confined columns presented a ruin by traction of the tension reinforcement steel. In the literature it is explained that the first type of ruin relates to the columns subjected to a small eccentricity [Germain 2006], and the second relates to the columns subjected to a large eccentricity. Our three confined columns showed a big deformation capacity that generated an increase in equivalent eccentricity of 80 to 219%. It is the case of tensile breakage of tension steels.

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