



Experimental Behaviour of R/C Columns Confined by FRP

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INTRODUCTION

The subject of this paper concerns the problem of repairing and seismic retrofitting the existing buildings. In particular, a study on the behaviour of reinforced concrete (R/C) columns externally strengthened with carbon or glass fibre reinforced polymers (FRPs) is presented.

The use of FRP materials in the field of civil engineering offers several advantages in comparison to the materials traditionally employed, such as better durability, excellent mechanical properties, etc..

In recent years the growing interest in FRP materials to retrofit reinforced concrete and masonry structures is confirmed by both the great number of practical applications and the development of some international codes dedicated to this argument: in Italy, two years ago, the National Council of Research (C.N.R.) edited a technical document reporting the Italian guidelines [1].

An advanced state of art of the scientific research in this field was recently presented in [2].

This paper presents the preliminary results of a wide experimental program still in progress at the Laboratory of Structures of the University of Salerno (Italy). Full scale square and rectangular reinforced concrete columns were tested under a constant axial load and monotonic or cyclically reversed horizontal loads.

The experimental tests were conducted on columns strengthened by wrapping one or more carbon (CFRP) or glass (GFRP) fiber reinforced polymers sheets around the concrete elements and on columns retrofitted by using both the external CFRP confinement system and longitudinal steel angles placed in correspondence of the four corners of the concrete elements; finally, some unstrengthened columns were also tested as control specimens to evaluate the benefits of the abovementioned retrofitting techniques.

Studied columns were designed to be representative of existing building structural components, i.e. of structural elements designed according to code provisions used in the past, that is without those details needed in seismic areas, and realized with concrete characterised by a low compression strength; for this last reason several compression tests were conducted on concrete specimens realized using different mixtures, in order to identify the mixture able to reproduce a concrete having a predefined value of the mean cylindrical strength (i.e. $f_{cm} \approx 25$ MPa).

Furthermore, in order to investigate the influence of bond between steel rebars and concrete on the nonlinear behaviour of the critical zone (plastic hinge), the longitudinal reinforcement was realized using both smooth and deformed steel rebars.

The performed tests – carried out in displacement control - have allowed evaluating the improvements in terms of strength and ductility due to the external FRP wrapping. Furthermore, the analyses of the nonlinear cyclic behaviour of the tested columns have provided to draw useful information about the stiffness degradation and the energy dissipation phenomena.

Keywords: concrete columns, FRP, seismic retrofitting

EXPERIMENTAL PROGRAM

The experimental program, whose preliminary results are presented in this paper, consists of monotonic and cyclic tests performed on full scale square and rectangular R/C columns.

The concrete samples were tested vertically, in a constant axial load condition, while a lateral load was applied using an actuator placed at a distance of about 1700 mm from the base of the column.

Tests were conducted under displacement control in order to investigate the structural response of the columns even on the softening branch of the force-displacement relationship; the load process was stopped to a conventionally defined collapse, i.e. on such softening branch in correspondence of a horizontal load equal to the 90% of the maximum one.

In order to analyze the behavior of existing structures, the columns have been realized using a low strength concrete (f_{cm} equal to about 25 MPa); for this purpose an optimization of the concrete mixture has been conducted by testing several concrete specimens.

For the same reason, reinforcement details (i.e. lap splice lengths, anchorages, hoop spaces, etc.) were arranged following design rules used in the past - nowadays not admitted in seismic zone - while some of the tested samples were reinforced by using smooth steel rebars instead of deformed ones; in this way the influence of the different bond between steel rebars and concrete on the nonlinear response of the tested columns was also investigated.

Some smooth rebars were tested under tension in order to evaluate their mechanical characteristics; the tensile tests provided a mean value of the yield strength (f_{ym}) equal to about 370 MPa, a yield strain equal to about 0.19%, and a maximum strength equal to about 535 MPa; therefore, such type of steel rebars can be considered more or less equivalent to the "AQ50" one, which represents a steel class currently used in the Italian practice during the years '50-'70.

The used deformed steel rebars, instead, can be classified as "FeB44k" type, and are widely used in Italy today; the characteristic yield strength of these rebars (f_{yk}) is equal to about 440 MPa.

The whole experimental program involved twenty five concrete specimens: sixteen samples consisted of square columns of dimensions 300x300x2200 mm having a stub of dimensions 1400x600x600 mm; the other nine samples consisted of rectangular 300x700x2500 mm columns with a stub of 1400x600x800 mm; the geometry and the steel configuration of specimens are depicted in Fig.1.

The stub represents a discontinuity similar to a footing or a beam-column joint.

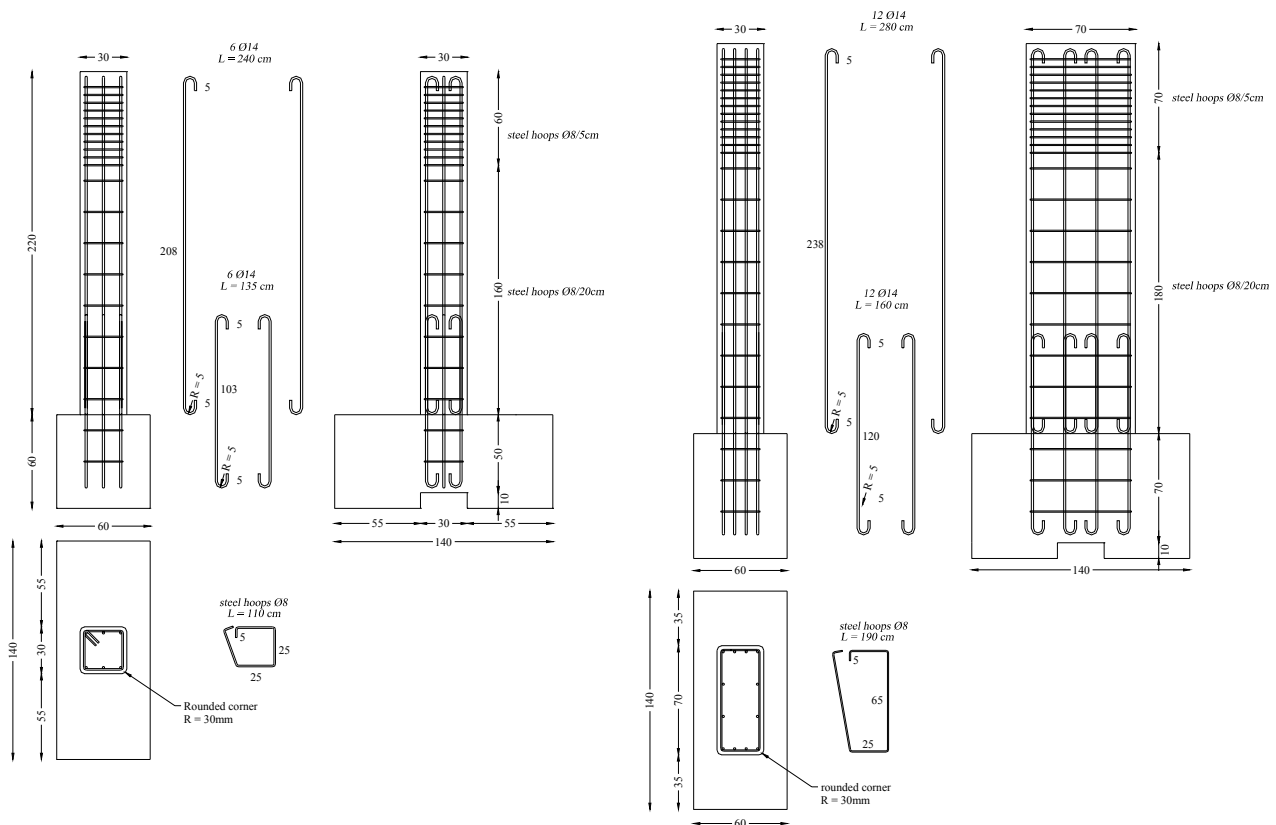


Fig. 1. Geometry and steel rebars configuration of the concrete specimens.

According to the practice, the specimens were cast and vibrated in vertical position; the cast was characterized by a discontinuity at the interface between the column and the concrete base.

During the columns' preparation some cubic specimens having sides of 150 mm were also cast and subsequently cured under the same environmental conditions. Compression tests on these cubic samples were performed just before the test of the column to evaluate the concrete strength at that time.

Independently on the type (smooth or deformed), the amount of longitudinal steel rebars was fixed equal to about 1.0% of the concrete gross cross-sectional area ($\mu=1.026\%$) in case of square columns and to about 0.9% for rectangular ones ($\mu=0.88\%$); 14 mm diameter rebars were used as longitudinal reinforcement, while 8 mm rebars, spaced at 200 mm, as steel hoops; the lap splice length of longitudinal rebars was of 600 mm.

The concrete cover was equal to 25 mm.

With reference to the sixteen square samples, eleven were reinforced by using smooth rebars and only five by deformed rebars; regarding the nine rectangular columns, six had smooth longitudinal rebars and three deformed ones.

Only six samples were not retrofitted and used as terms of comparison to evaluate the benefits introduced with the tested retrofitting techniques; the others (19) were strengthened by means of a passive confinement system obtained by externally wrapping unidirectional carbon or glass FRP layers around the concrete element. In particular, a discontinuous wrapping was provided along the element.

Starting from the column-stub interface the first portion of the column (of about 500 mm) was completely confined, while the remaining part of the column have been strengthened by means of 150 mm wide FRP bands 200 mm spaced (see Fig.2.).

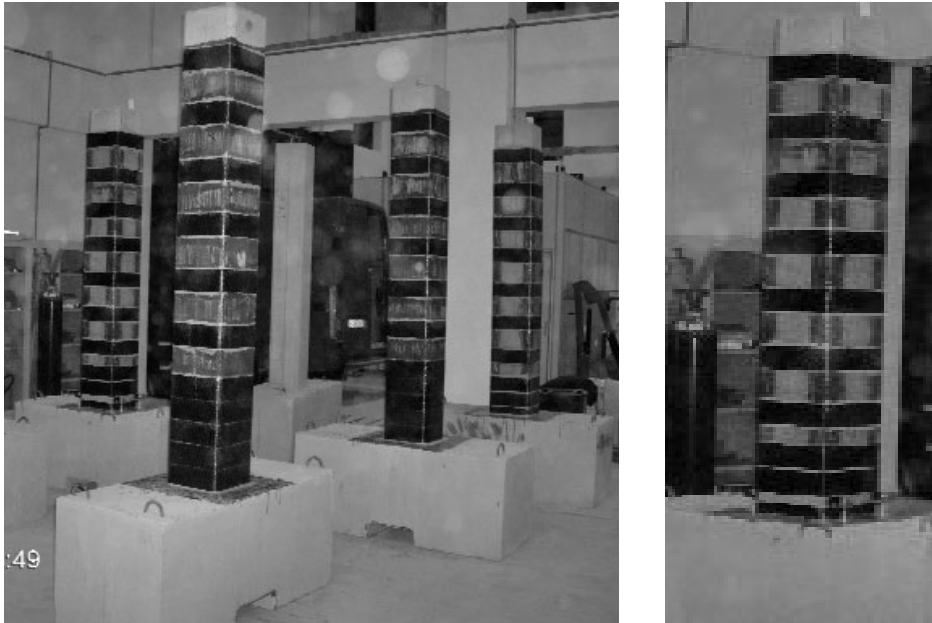


Fig. 2. FRP-confined square r/c columns.

The corners of columns were rounded to facilitate the FRP wrapping and to avoid excessive stress concentrations during the test; for this purpose a steel form with rounded concave corners having 30 mm radius was used during the cast.

In order to improve the flexural strength, before the wrapping of the FRP sheets, seven of the abovementioned nineteen confined columns were also reinforced by positioning steel angles in correspondence of the column corners: in four cases these angles have been anchored to the concrete base by means of steel rods encased in the foundation block; the other three samples had angles unconnected to the base (see right side of Fig.2). This type of retrofitting of the columns was designed and executed in cooperation with the INTERBAU firm (Milan, Italy).

Table 1 shows the properties of the FRP materials used to retrofit the r/c columns: the first column reports the type of fibre, the second one the thickness, the third column the elastic modulus, the fourth shows the ultimate tensile strength, while the corresponding ultimate strain is reported in the last column.

The CRFP and GRFP sheets have been provided by SIKA and MAPEI firm respectively.

Fiber	t_f [mm]	E_{FRP} [GPa]	$\sigma_{u,FRP}$ [MPa]	$\epsilon_{u,FRP}$ [%]
CFRP	0.22	390.0	3000	0.80
GFRP	0.48	80.6	2560	3-4

Tab. 1. Material properties.

The main characteristics of test specimens are reported in Table 2; in the first column of the table is indicated a designation – which contains the main information of the test – used to differentiate each sample.

The first letters of these labels indicate the type of test: “M” means monotonic; “C” for cyclic; the following numbers represent the number of the column (from 1 to 25); the letter “S” stands for column reinforced with longitudinal smooth rebars, while the letter “D” means that deformed rebars were used.

In case of strengthened columns, the letter “G” stands for passive confinement system made of glass fiber; “C” stands for carbon confinement system; the labels “A1” and “A2” denotes that the column was strengthened by using both a CFRP confinement system and steel angles connected or unconnected to the concrete base respectively.

TEST	B [mm]	H [mm]	μ [%]	type of test	Type of steel rebars	Type of fibre	FRP Layers	
M5-S	300	300	1.026	monotonic	smooth	-	-	
C3-S	300	300	1.026	cyclic		-	-	
C1-S-G	300	300	1.026	cyclic		Glass	4	
C4-S-G	300	300	1.026	cyclic		Glass	2	
C2-S-A2	300	300	1.026	cyclic		Carbon	2	
C6-S-A1	300	300	1.026	cyclic		Carbon	2	
C10-S-A1	300	300	1.026	cyclic		Carbon	2	
C11-S-A2	300	300	1.026	cyclic		Carbon	2	
M12-S-C	300	300	1.026	monotonic		Carbon	2	
C-13-S-C	300	300	1.026	cyclic		Carbon	2	
C14-S-C	300	300	1.026	cyclic		Carbon	4	
M17-S	300	700	0.880	monotonic		smooth	-	-
C18-S	300	700	0.880	cyclic			-	-
M19-S-C	300	700	0.880	monotonic			Carbon	2
C20-S-C	300	700	0.880	cyclic	Carbon		2	
C21-S-C	300	700	0.880	cyclic	Carbon		4	
C22-S-A2	300	700	0.880	cyclic	Carbon		2	
C7-D-C	300	300	1.026	cyclic	deformed	Carbon	2	
C8-D-C	300	300	1.026	cyclic		Carbon	4	
C9-D	300	300	1.026	cyclic		-	-	
C15-D-A2	300	300	1.026	cyclic		Carbon	2	
C16-D-A1	300	300	1.026	cyclic		Carbon	2	
C23-D	300	700	0.880	cyclic		deformed	-	-
C24-D-C	300	700	0.880	cyclic	Carbon		2	
C25-D-C	300	700	0.880	cyclic	Carbon		2	

Tab. 2. Details of concrete specimens.

In a successive phase - not described herein – after the tests conducted up to the above-defined “conventional collapse”, the damaged unconfined columns will be strengthened by using both the CFRP confining system and the steel angles. The tests results of such repaired columns will be compared with those obtained in the first phase, relative to the “A1” and “A2” samples.

TEST SETUP AND TEST PROCEDURES

The test set-up was designed to subject the concrete samples to a constant value of the axial load and to monotonic or cyclic horizontal loads; the specimen was mounted vertically with the reinforced concrete foundation being held by a steel equipment fixed on the lab’s floor (see Fig. 3).

The vertical load was applied to each column by pretensioning two high strength steel rods by means of an hydraulic jack - placed at the top of the column - having the capacity of 1000 kN and was measured using a load cell of similar capacity; in order to keep the axial force at a constant value the jack was connected to a 25 litres nitrogen accumulator.

The value of the axial load N was calibrated in order to achieve, for all specimens, a value equal to 14% of the following non dimensional parameter:

$$v = \frac{N}{B \cdot H \cdot f_{cm}} \quad (1)$$

where: f_{cm} is the average value of the cylindrical concrete strength, evaluated by performing compression tests on concrete specimens cast with each column; B and H are the two dimensions of the column section.

The high strength steel rods were connected by means of cylindrical hinges to two transversal steel beams, one placed on the top of the sample and the other under the concrete stub. In order to not introduce at the column-stub interface an eccentricity of the vertical load, the lower hinge was placed at the centroid of the base column section.

The reversing horizontal loads were applied, in displacement control at a quasi-static rate, by means of a “MTS” ± 500 kN hydraulic actuator; in case of cyclic tests an increment of the displacement every three cycles was considered in order to evaluate the strength and stiffness degradation at repeated lateral load reversals. The loading displacement history is shown in Fig.4.

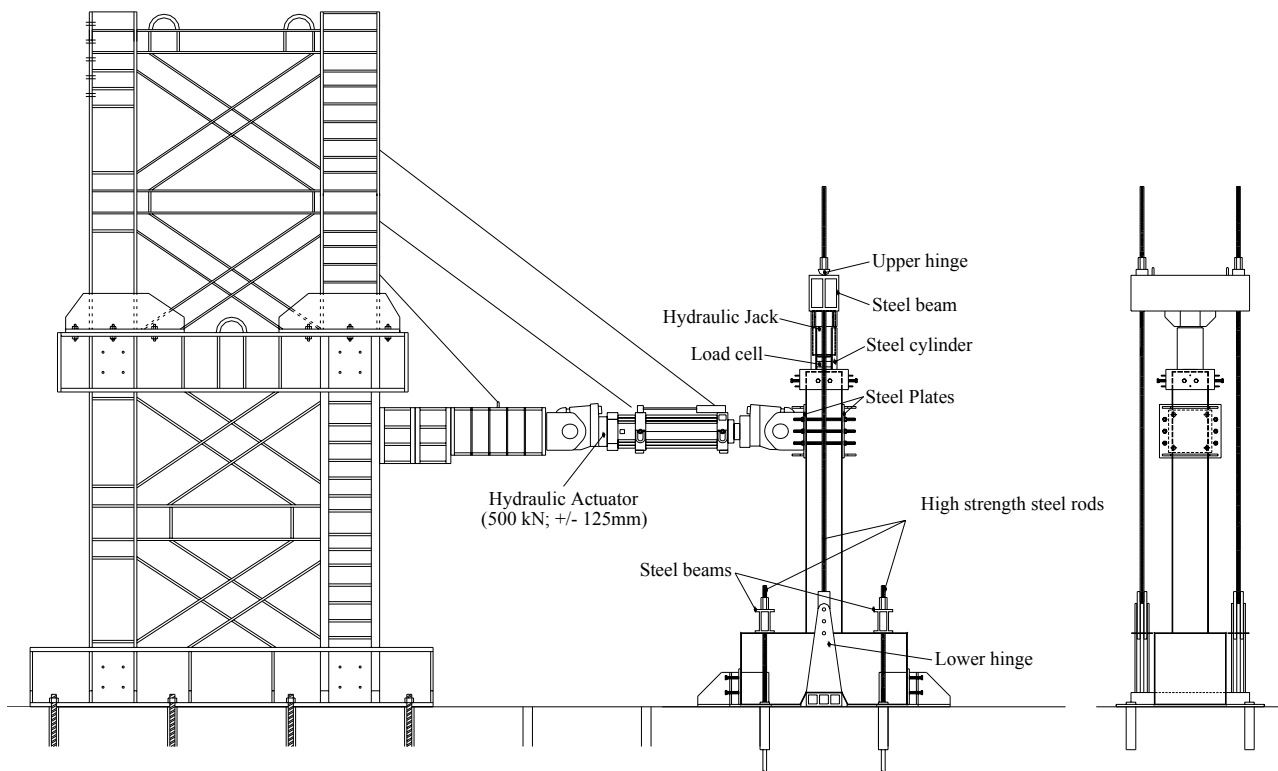


Fig. 3. Experimental test set-up.

The actuator was horizontally mounted to a reaction steel frame and was capable of moving the top of the column 125 mm in both positive and negative directions; a displacement of 125 mm corresponds to a drift of 7.35 percent.

As abovementioned the tests were stopped as the columns reached the “conventional collapse”, i.e. in correspondence of strength degradation equal to 10%.

During the tests loads, displacements, cracks, etc. have been measured; in particular:

1. the vertical load was measured using a 500 kN load cell;
2. horizontal and vertical strains were measured using several surface strain gauges placed at about 100 mm from the stub face (i.e. in the critical region);
3. four LVDTs were mounted on the columns to measure the vertical deformation of the concrete at the column-stub interface;
4. deflection along the length of the column was measured using two trasducers placed at the top of the column and in correspondence of the actuator horizontal axis.

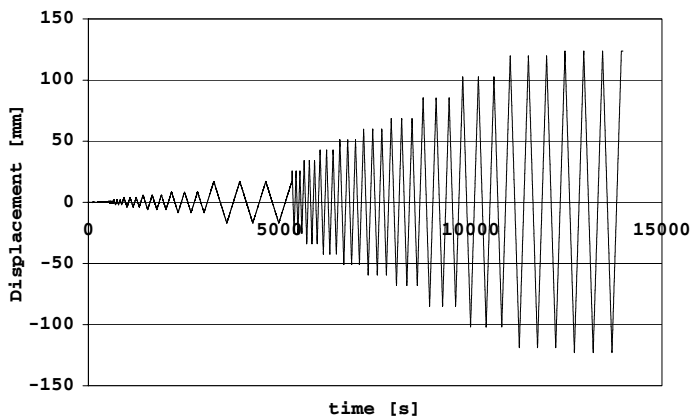


Fig. 4. Loading displacement history.

PRELIMINARY EXPERIMENTAL RESULTS

After the first four tests (the first four reported in table 2) some significant results were obtained. All the performed tests were conducted on 300x300 mm square columns reinforced using smooth steel rebars: the two unstrengthened samples “M5-S” and “C3-S” and the two glass fibre reinforced polymers confined columns “C4-S-G” and “C1-S-G”.

For what concerns the compressive concrete strength, the compression tests performed on the concrete specimens cast with the column “C1-S-G” provided a mean cylindrical strength ($f_{cm}=28.8 \text{ N/mm}^2$) larger than the one obtained for the other three columns ($f_{cm}=25.5 \text{ N/mm}^2$). Therefore, according to Eq. (1), the axial load was assumed equal to 360 kN for the specimen “C1-S-G” and equal to 320 kN for the other samples.

Even though it was planned to stop each test at the abovementioned “conventional collapse”, in the case of sample “C1-S-G” the test was stopped before this threshold, because the collapse was not yet attained when the MTS actuator reached its maximum displacement (125 mm).

Deflections of columns “C3-S” and “C4-S-G” shown during the tests are presented in Fig.5.



Fig. 5. Unconfined “C3-S” column (left) and GFRP confined “C4-S-G” column (right) during the test.

In all tests, a flexural crack perpendicular to the column’s axis developed first close to the column-foundation interface, i.e. in correspondence of the column section subjected to the highest moment; the width of this horizontal crack significantly increased during the test.

Fig. 6 shows two pictures taken just before the collapse of the column and relative to two opposite sides of sample “C4-S-G”; it can be noticed the very large crack width in tension and the buckling of the GFRP system in compression. It has to be underlined that, due to the very low bond between smooth rebars and concrete, the large crack at the bottom end of the column was observed in all the tests.

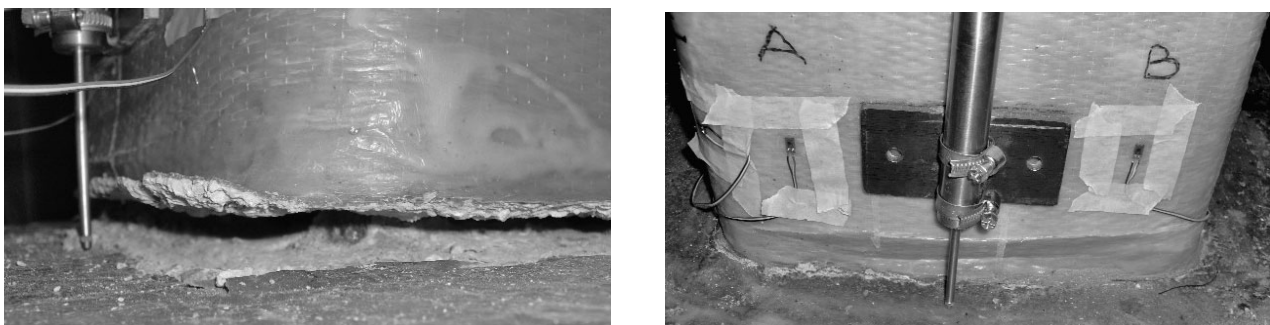


Fig. 6. Crack width and GFRP failure at the base of sample “C4-S-G”.

In case of the three cyclic tests a second sign of damage appeared in the form of a further sub-horizontal crack formed at a distance of 600 mm from the face of the stub, i.e. exactly at the end of the rebars lap splice length. For the specimen without FRP sheets (specimen “C3-S”) this new flexural crack fastly developed becoming inclined; after the maximum lateral strength was attained, this crack connected to another vertical one due to the incipient buckling of a compressed steel rebar; then the concrete cover spalled (see Fig.7). In the case of GFRP confined samples (“C1-S-G” and “C4-S-G”), the presence of the FRP layers wrapped at the column base for the first 500 mm prevented the spread of these cracks and - as can be observed in Fig.7 - the concrete cover did not spall.

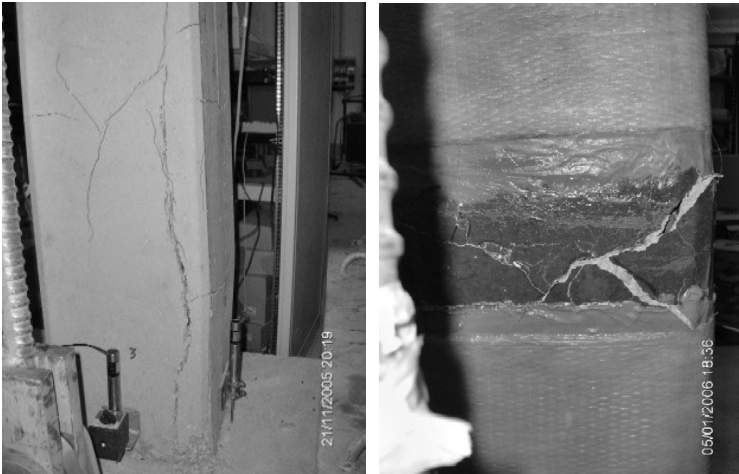


Fig. 7. Crack patterns for GFRP confined column (right) and the unconfined one.

In Fig. 8 the force-displacement curves for the first four tests are depicted; in particular, the last diagram shows both the force-displacement curve of the monotonic tests “M5-S” and the envelopes of the three cyclic tests. Observing the Figure it can be recognised that the cyclic behaviour of the GFRP confined columns is significantly more ductile than that relative to the unconfined column; on the contrary, due to the low value of the axial load, the confinement system produced only a slight improvement in terms of strength.

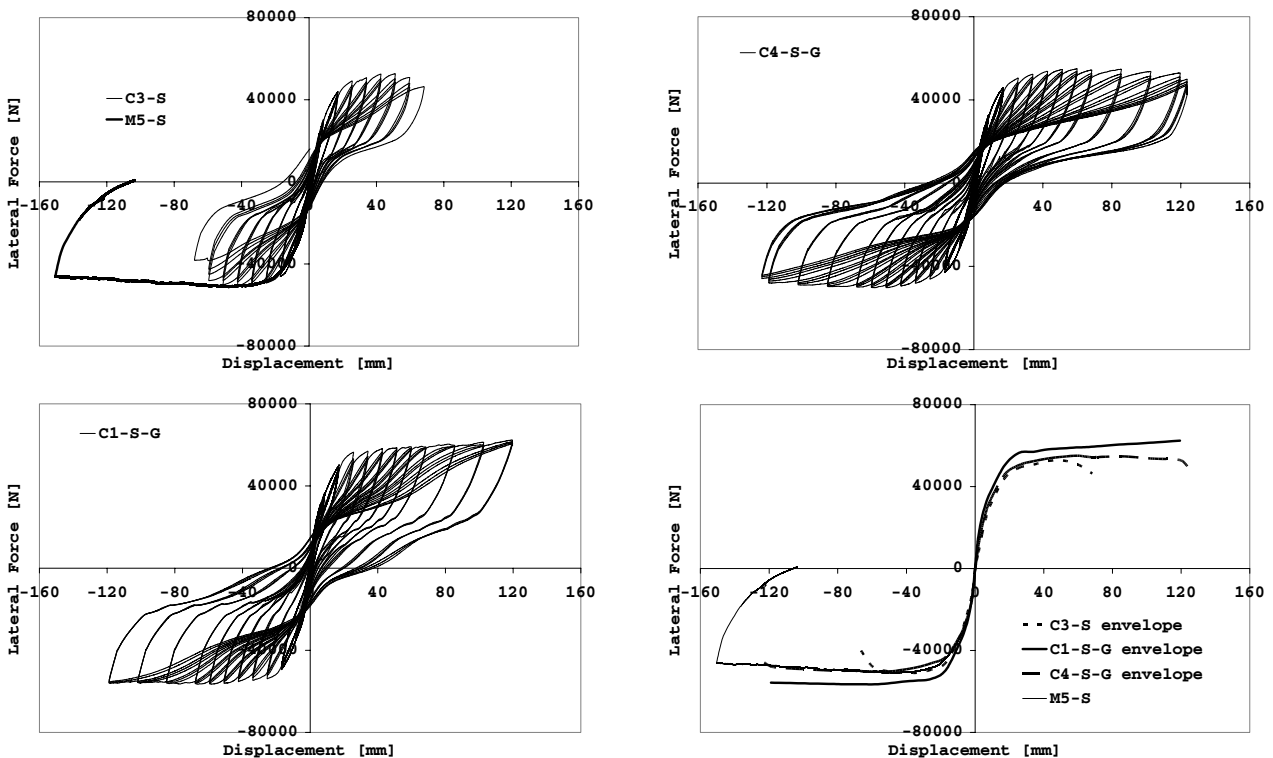


Fig. 8. Hysteresis loops and load-displacement envelopes.

In the monotonic test conducted on the unconfined column, the conventional collapse was attained for a column drift equal to about 8%; during the cyclic tests on an analogous unstrengthened sample (“C3-S”) the collapse happen with a 4% column drift; in case of GFRP confined columns the “ultimate” drift was instead greater than 7%.

In all the cases the hysteresis loops are characterised by a slight dissymmetrical behaviour; furthermore, a very low strength degradation is observed and a significant pinching phenomenon - due to the low bond between smooth rebars and concrete – is evidenced by the loops.

According to previous observations made by other Authors [3], Fig. 8 shows that the pinching phenomenon decreases as the axial load increases (low pinching in the “C1-S-G” test).

The column base rotation-drift diagrams for all the performed tests are shown in Fig.9; the base rotations were obtained by the data measured with two LVDTs located at the bottom end of the column on the two sides perpendicular to the horizontal load direction.

Observing the experimental curves, the very large contribute of the fixed-end rotation on the overall rotational capacity of the column can be recognised for all the tested samples. Therefore, the large flexural ductility exhibited by the tested columns is essentially due to the large width of the crack developed at the column-foundation interface: during the tests a crack width of about 15 mm was measured.

Analogous results were obtained by other Authors [3].

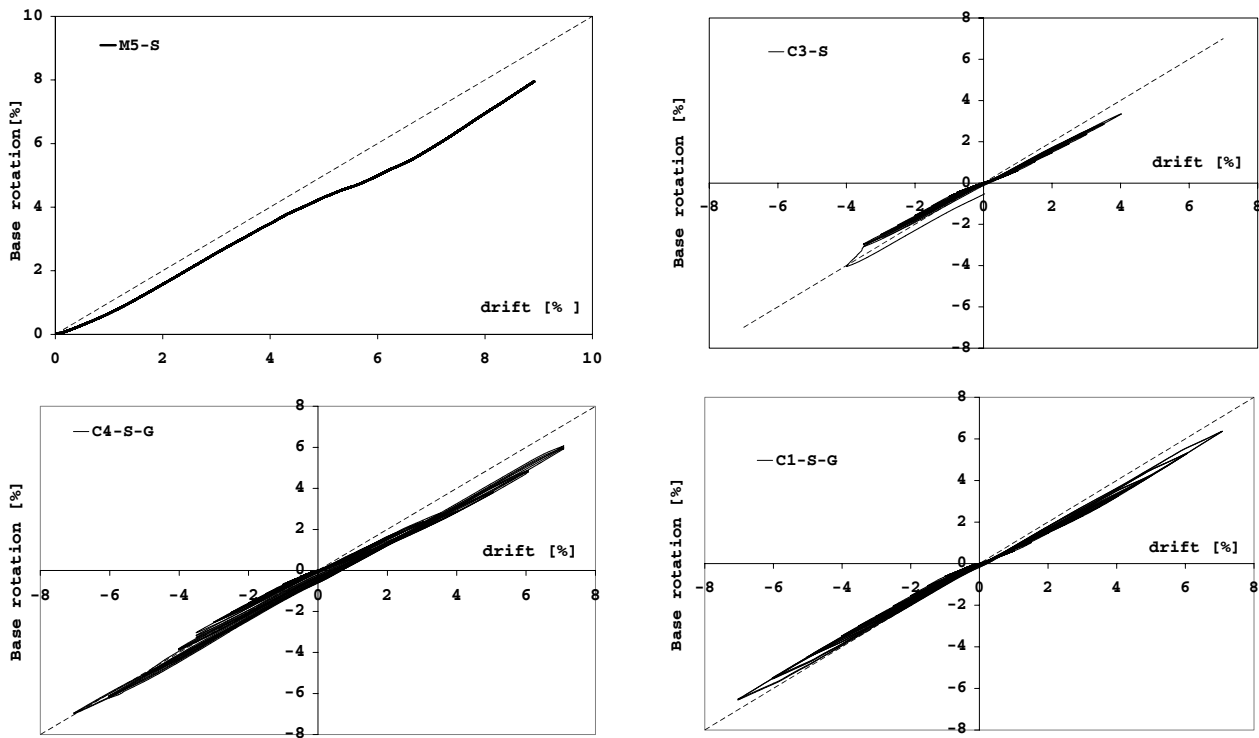


Fig. 9. Base rotation versus drift curves of the tested samples.

Stiffness degradation

The tested columns under cyclic shear loads have shown progressive stiffness degradation and a significant reduction of strength for constant deformation cycling (see Fig.8).

Usually it is complex to calculate the stiffness degradation because of the difficulty in defining a sufficiently significant parameter. In fact, the nonlinear behaviour obliges to choose a conventional index to estimate such reduction. Then a mean value of stiffness for the *i*-th cycle has been herein evaluated by the following ratio [4]:

$$k_{m,i} = \frac{|H_{max,i}^+| + |H_{max,i}^-|}{|S_{max,i}^+| + |S_{max,i}^-|} \quad (2)$$

where $H_{max,i}^+$ and $H_{max,i}^-$ are the i -th positive and negative maximum force respectively, and $s_{max,i}^+$ and $s_{max,i}^-$ are the corresponding displacement.

The relationships between $k_{m,i}$ and the displacement and between $k_{m,i}$ and the number of cycle are depicted in Fig.10. Such Figure shows that the law of variation of stiffness with displacement is practically not dependent on the presence of the FRP confinement system.

Furthermore, considering constant deformation cycles the stiffness decreases, showing a fatigue phenomenon.

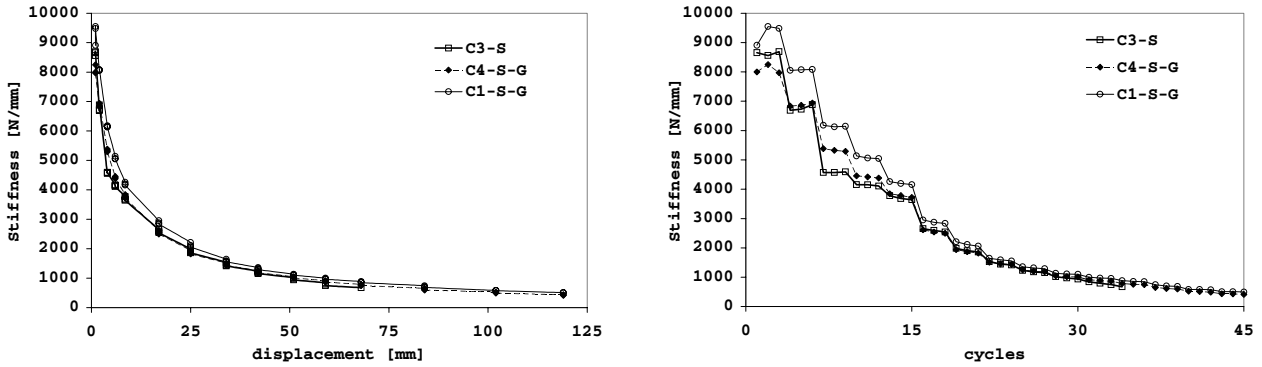


Fig. 10. Stiffness degradation.

Energy dissipation

A good behaviour of structural members under cyclic action is always related to the energy dissipation capacity, in particular if such capacity is not coupled with strong degradation of the strength and/or of the stiffness.

Fig.11 shows the relationship between the energy dissipated in the i -th cycle (E_i) and the number of the cycle. Observing the trend reported in this figure it can be noticed that no particular difference have been evidenced between the three cyclic tests at least up to the last cycle reached by the unconfined column “C3-S”; then, only a slight increment of energy dissipation has been evaluated for the last cycles of test “C1-S-G”.

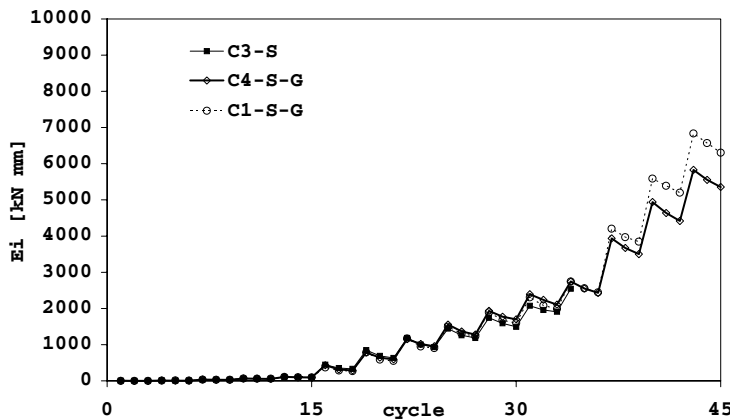


Fig. 11. Energy dissipated in the i -th cycle vs number of cycle.

The relationships between the dissipated energy and the displacement shown in Fig.12 confirm the abovementioned observations; as obvious, the total dissipated energy ($E_{h,tot}$) of the unconfined column is the lowest one, since such sample reached the conventional collapse before than the others.

Finally, the dimensionless ratio of the stiffness $k_{m,i}$ and the stiffness $k_{m,1}$ - computed at the first cycle with Eq.(2) - is reported in Fig.13 as a function of the ratio between the energy E_h dissipated up to the i -th cycle and the total dissipated energy $E_{h,tot}$.

The figure shows experimental hyperbolic trends that are independent on the type of column (confined or unconfined); this means that in all the cases significant energy dissipation always corresponds to large stiffness degradation.

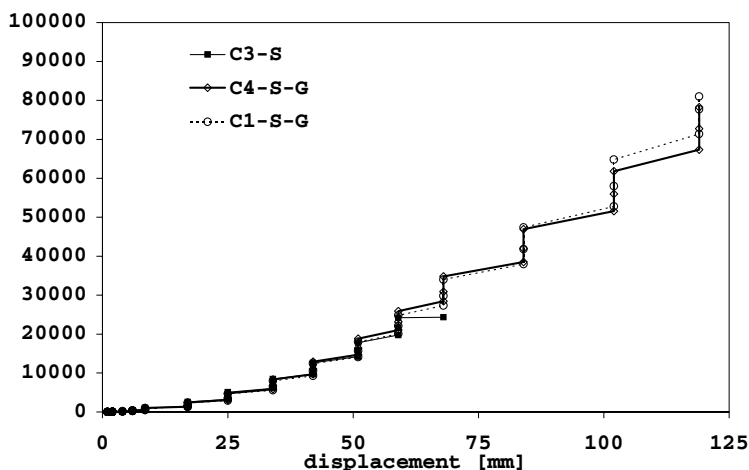


Fig. 12. Dissipated energy vs displacement.

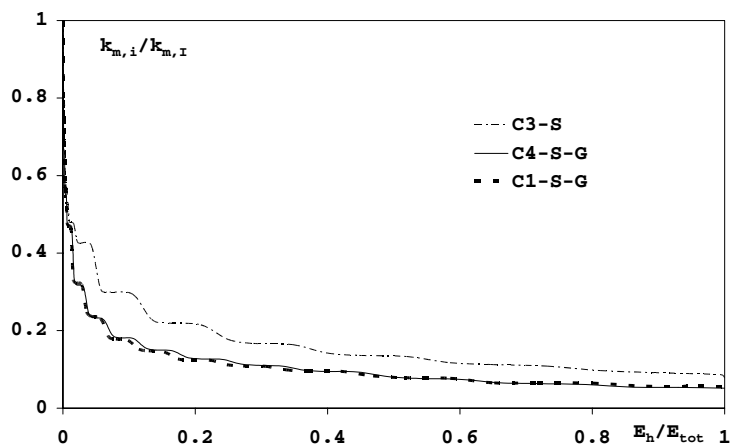


Fig. 13. Dimensionless stiffness ratio vs hysteretic energy ratio

CONCLUSIONS

A wide experimental programme is still in progress at the Laboratory of Structures of the University of Salerno on full scale square and rectangular R/C columns externally confined using a FRP system or strengthened by using both the external FRP confinement system and longitudinal steel angles placed on the corners. Some tests on unstrengthened columns were also performed in order to evaluate the benefits of the abovementioned retrofitting techniques.

Tests, performed under a constant axial load and horizontal loads applied by controlling the horizontal displacement, are conducted up to a “conventional collapse” assumed in correspondence of the strength degradation equal to 10%.

The preliminary results of such programme are presented herein; in particular results from tests on unstrengthened square columns are compared to those obtained by testing two GFRP confined columns. The longitudinal reinforcement of all the tested columns was provided by smooth steel rebars.

The test results have shown a large rotational capacity of both unconfined and confined samples due to a significant fixed end rotation; the last one was caused by a very low bond between smooth rebars and concrete.

Nevertheless, due to the FRP action, a clear improvement in terms of ductility – accompanied from a small increment of the maximum lateral load - was also observed during the tests. In fact, the FRP system was able to contrast the spread of cracks in the critical region thus reducing the strength degradation and increasing the displacement at collapse.

Finally, from the analysis of the cyclic tests data it comes out that: the law of variation of stiffness with displacement is resulted to be not dependent on the type of column (confined or unconfined); the relationship between stiffness degradation and dissipated energy in all the cases is characterised by an hyperbolic trend so that a significant energy dissipation always corresponds to a large stiffness degradation.

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REFERENCES

1. Italian National Research Council (CNR). Guidelines for Design, Execution and Control of Strengthening Interventions by Means of Fibre-Reinforced Composites (Materials, reinforced concrete and prestressed concrete structures, masonry structures). *CNR-DT 200/2004*, 2004, Advisory Committee on Technical Regulations for Constructions.
2. Teng JG, Chen JF, Smith ST, Lam L. FRP-strengthened RC structures. *Edited by John Wiley & Sons Ltd.*, 2002.
3. Fabbrocino G, Verderame GM. Rotational capacity of old type r/c columns. *Fib Symposium "Keep Concrete Attractive"*, 2005, Budapest, Hungary.
4. Mayes RL, Clough RW. State of the Art in Seismic Shear Strength of Masonry – An Evaluation and Review, *EERC Report, Berkeley, No.21*, October 1975, California, USA.