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RESEARCH ARTICLE - CIVIL ENGINEERING

Numerical Modeling of Reinforced Concrete Strengthened Columns Under Cyclic Loading

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Abstract Although many numerical investigations are available on external strengthening of reinforced concrete columns using fiber-reinforced polymer jackets, there are few numerical investigations presented on rectangular/square reinforced concrete columns subjected to cyclic lateral loading. In the present work, numerical analyses on square reinforced concrete columns strengthened using fiber-reinforced polymer and steel bars were carried out. This paper is divided into two parts. In the first part, two numerical models were presented to predict the cyclic behavior of the columns strengthened using either glass fiber-reinforced polymer or steel bars. The results of the first part showed a good accordance between the numerical modeling and the experimental laboratory tests. The second part reports a numerical investigation that evaluates the comparative behavior of reinforced concrete columns strengthened with glass fiber-reinforced polymer subjected to cyclic lateral loading, in which a parametric study related to the amount of the glass fiber-reinforced polymer material wrapping with 1–3 layers was performed. The results of the parametric study revealed that although the general behaviors of the columns are slightly similar in terms of load capacity and maximal displacement, the solicitations at the glass fiber-reinforced polymer material scale and the values of the cracks corresponding to the maximum load are different. The main objectives of this numerical modeling were to provide a comparison between the numerical models and the experimental tests and to show the necessity to perform more complex numerical simulation to describe the behavior of strengthened materials in finer scale.

Keywords Numerical analyses · GFRP wrap · Columns strengthening · Experimental tests · Rehabilitation

1 Introduction

Many existing reinforced concrete (RC) frames buildings located in the seismic zones are deficient to resist moderate to severe earthquakes [1]. The need to improve the resistance performance of the existing buildings is a more and more pressing in the area of rehabilitation of the concrete structures. In order to ensure compliance with the security levels required by various applicable regulations, including requirements in deformability and in resistance, the structural members may need to be upgraded to current seismic requirements, as existing structural components inadequately reinforced may be deficient in terms of their seismic strength. Strengthening such elements is a method to increase the flexural [2], axial compressive [3] and shear strengths [4]. The methods of strengthening depend on the type of structure and loading. For structures subjected mainly to static loads, increasing flexural and axial compressive strengths is more considerable. For structures subjected mainly to dynamic load, increasing flexural and shear strengths is more considerable, and improving column ductility and rearrangement of column stiffness can be achieved by repairing techniques. Among several techniques of strengthening of structural elements, the composite materials as the most commonly used fiber-reinforced polymers (FRP) due to their high longitudinal modulus of elasticity, high ultimate strength and the weak resistance variation between them are quoted. The columns

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are more critical members in a concrete structure. Their failure may lead to a partial or even a total collapse of the whole structure. The behavior of structures under cyclic loads can be used as mean of evaluating the behavior of structures using different types of strengthening [5–8].

Significant researches have been devoted to RC columns strengthened using FRP, and numerous models were proposed. Mazzucco et al. [9] simulated the behavior of concrete columns strengthened with carbon fiber-reinforced polymer under axial loading using a damage model. The model was applied on circular and square columns under monotonic compressive loading. Manal [10] verified the performance of FRP strengthened concrete columns under combined axial–flexural loading where it has presented an optimization technique for retrofitting RC columns using FRP. The same type of loading was performed in experimental investigation on rectangular RC columns using carbon fiber-reinforced polymer (CFRP) by Alireza et al. [11]. Various parameters such as CFRP thickness, eccentricities, and fiber orientation were considered. From the study, it was found that the overall behavior of the specimens revealed an increase in the moment capacity and stiffness with an improvement of ductility causing a greater level of energy dissipation. Increasing the number of longitudinal layers would improve the CFRP stiffness decreasing the curvatures.

Najwa et al. [12] performed an investigation on different types of cross sections of concrete columns, including circular, square and rectangular strengthened with fiber-reinforced polymer composites. A modified concrete damaged plasticity (MCDP) was used in this investigation. The influence of the different input parameters of concrete damages plasticity model on the monotonic behavior was studied, and it was shown that the MCDP predicts the monotonic stress–axial strain response of the FRP-confined concrete columns. In [13], Barbato performed a simple frame finite element model in which the load-carrying capacity and ductility of reinforced concrete circular columns confined with externally bonded fiber-reinforced polymers are estimated. The collapse mechanism, reinforcement steel yielding, and FRP rupture can be simulated by this model. Jinsup et al. [14] performed experimental and numerical studies on the seismic performance of non-seismically detailed RC columns retrofitted with glass fiber-reinforced polymer (GFRP) strengthening device in order to improve the shear strength and ductility of weak-in-shear columns. It was found that the improvement in shear behavior is observed by comparing strains in the transverse re-bars, which are increasing when more amount of GFRP strip is incorporated to the column. In [15], Fatemeh et al. studied the behavior of RC columns confined by FRP and conventional lateral steel reinforcement and a constitutive stress–strain model was implemented.

Almost all the previous studies were undertaken at the full scale of the RC columns strengthened with FRP. There are few studies which attempted to model the finer scale of the structural elements in order to take into account the parameters that influence the structural behavior at coarse scale. In this paper, numerical simulations of RC columns strengthened with composite materials are presented. This paper is presented in two parts. In the first part, numerical models are developed to predict de behavior of experimental columns strengthened with GFRP and steel bars under cyclic lateral loading. The second part reports a comparative study related to the amount of the GFRP material wrapping with varying the number of layers (from 1 to 3) in order to describe its influence on the RC columns behavior and to reveal the necessity of applying a multi-level model to describe the realistic behavior of RC columns strengthened with GFRP at the full scale.

2 Research Significance

The use of externally bonded FRP composite for strengthening and repair can be a cost-effective alternative for restoring or upgrading the performance of existing concrete columns. Even though a lot of research has been directed toward circular columns, relatively less works have been performed on square and rectangular columns, to examine the effects of external confinement on the structural performance. However, the vast majority of all columns in buildings are square and rectangular columns. Therefore, their strength and rehabilitation need to be given attention to preserve the integrity of building infrastructure. This paper contributes to the understanding of the cyclic behavior of square RC columns strengthened with GFRP and steel bars using the finite element modeling.

3 Experimental Program Description

According to the experimental tests performed in the Laboratory of Civil Engineering Faculty, Department of CCIA [16] (see Figs. 1, 2), two RC columns were strengthened either using GFRP or steel bars. For the first column strengthened using GFRP (named C6C1–GW–BC), lateral winding of GFRP jackets was applied on the base of the column. The overlap length of the GFRP jackets was 600 mm. No anchorage of the GFRP jackets to the footing of the column was applied in the experimental tests (see Fig. 5c). It should be noted that the notation C6C1–GW–BC means that the concrete column (C6C1) was wrapped using glass FRP (GW) and confined in its base (BC). For the second column strengthened using steel bars (named C3C–BM–AF), two longitudinal steel bars were placed on left and right sides of the base of the column and chemically anchored to the footing. Vertical silts were cut into the cover concrete, about

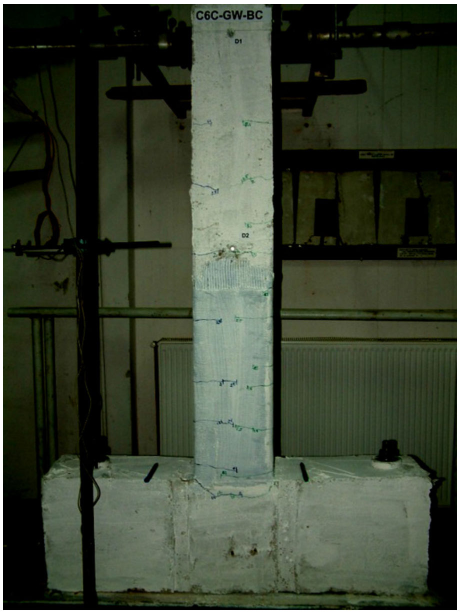

Fig. 1 General view of experimental stand element C6C1–GW–BC after retrofit with crack pattern [16]

Fig. 2 General view of experimental stand element C3C–BM–AF after retrofit with crack pattern [16]

2 cm deep. Afterward, holes were drilled in the footing of the column for anchorage of the steel bars. The steel bars were placed into the holes and chemically anchored to the footing using the resin anchor HILTI HIT-RE500. According to the technical data of the resin anchor HILTI HIT-RE500 [17], anchorage of the steel bars was performed using a length of 180 mm. The last step of the strengthening using steel bars is the filling of the vertical silts using epoxy mortar. The overlap length of the steel bars at the base of the column was 700 mm.

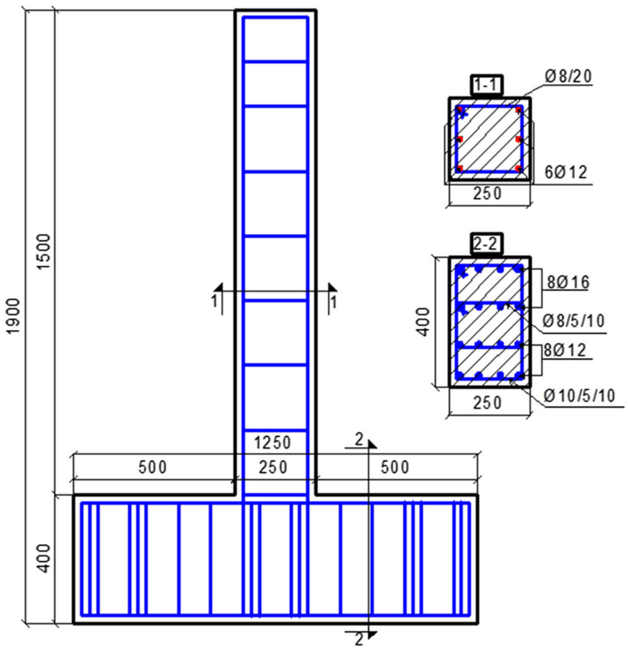

Fig. 3 Plan of reinforcement elements

It should be noted that the notation C3C–BM–AF means that the concrete column (C3C) was strengthened using steel bars (BM) and these bars were anchored to the footing (AF). Since the implementation of this concept in Japan in the 1980s, this technique has been widely used. Its great advantage is the flexibility offered in terms of adaptability to different section shapes and positions of elements.

The details of the geometry and reinforcement of the columns are shown in Fig. 3. Using presses application of hydraulic force, the attempts were subjected to cyclic lateral loading without axial force. To generate lateral load, hydraulic pump was used in experimental tests (see Fig. 4). The steps of the preparation of the columns strengthening are presented in Fig. 5.

The compression tests on concrete and tensile tests on steel bars were carried out in order to determine the strength characteristics of the columns. The mechanical characteristics of the reinforced concrete and the diameters of the steel bars are shown in Tables 1 and 2, respectively. A test on a reference column was made to determine the limit of elastic behavior of the column [16]. The results showed that the elastic limit is achieved when the loading reaches 25 kN corresponding to the displacement of 24 mm.

4 Finite Element Modeling

The aim of the numerical analysis is the description of the behavior of the RC columns as were done experimentally. Two RC columns strengthened with either steel bars (Fig. 6) or GFRP materials (Fig. 7) were modeled. 2D finite element

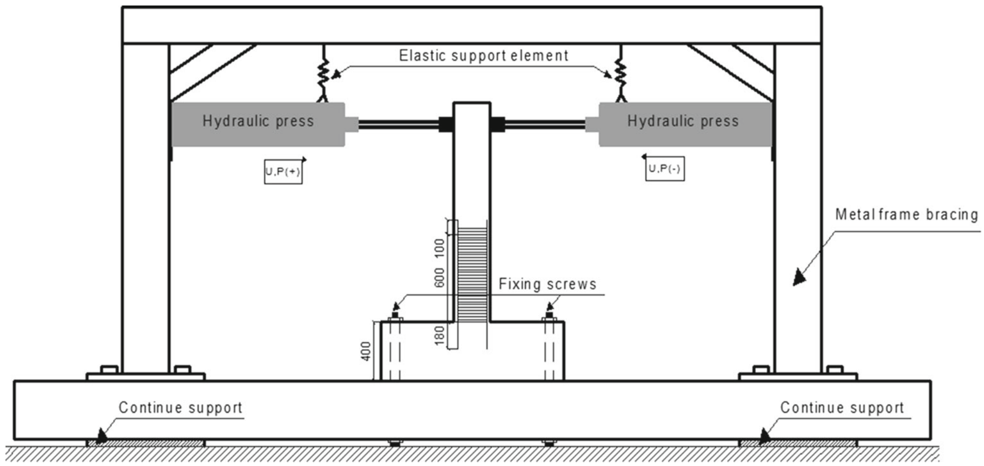

Fig. 4 Elements of the test bench

model was created using the software of analysis of concrete and reinforced concrete structures ATENA [18]. From the element library of ATENA software, 2D four-node solid structural element (SBETA) was used to model the concrete of the columns. The solid element has four nodes with two degrees of freedom in each node, namely translations in x and y directions. Steel bars and GFRP bars were represented by two-node structural bar elements.

The application of loading on the studied columns was performed similar to the experimental tests [16] and was divided into two phases. In the first phase, several load steps of imposed force (each having a value of 1 kN) were applied until the model reaches the elastic limit. In the second phase, several load steps of imposed displacement (1 mm each) were applied until failure. Five monitoring points were defined in order to control force and displacement, namely one monitoring point for control of displacement (D1) (see Fig. 8) and four monitoring point for control of force at the load application points (right and left of the column). The five monitoring points were situated at a same point of load application. In Fig. 8, U and P represent the imposed displacement and force, respectively. The signs '+' and '−' represent the right and left direction, respectively. The column strengthened with GFRP was subjected to the same values of the imposed step forces and step displacements (1 kN for force and 1 mm for displacement) similar to the column strengthened with steel bars. The numerical analysis was terminated when the 75% of the ultimate load was reached. The geometry of the columns and the load application were similar to the experimental ones. The mechanical characteristics of GFRP and steel bars used in this study are shown in Table 3.

5 Constitutive Model Law

Nonlinear behavior for concrete was used. For the model of strain softening of the concrete in compression, strain softening law based on strain was used. In this case, the softening law is defined by means of the softening modulus. The model of tensile softening was used with the exponential crack opening law based on the crack band theory [19]. This formulation is suitable for modeling of crack propagation in concrete.

Based on the smeared crack concept, the rotated crack model [20] for cracks was used in order to take into account the rotation of the cracks directions when the principal strain axes rotate during the loading. The stress–strain behavior of reinforcing steel was represented by the nonlinear stress–strain relationship and cyclic rules proposed by Menegotto and Pinto (Fig. 9) [21]. As no debonding was observed during the experimental tests, perfect bond between reinforcing materials (GFRP and steel bars) and concrete was considered in the numerical modeling.

6 Comparison of the Numerical Analyses with the Experimental Results

6.1 Column Strengthened with Steel Bars

The experimental and numerical lateral force–displacement relationships of the column strengthened using steel bars are presented in Fig. 10. From the figure, it was observed that the maximum displacement in the numerical analysis and experimental test was 54.04 and 55.00 mm, respectively, and the maximum force supported by the column in the numerical analysis and experimental test was 49.24 and 38.15 kN, respectively. The results showed that the model overestimates the force supported by the column. However, it gives a good estimate of the ultimate displacement. The overestimation of the force can be observed because of some micro-cracks appeared in the concrete before starting the loading, these micro-cracks may occur because of the non-hydration of

Fig. 5 Samples preparation: **a** cleaning of surface, **b** uniform application of the epoxy resin correction, **c** application of wrapping tissue of GFRP, **d** application of *side bars* [16]

Table 1 Mechanical characteristics of concrete

Concrete C16/20		
Compressive strength (MPa)	Elastic modulus E (MPa)	Tensile strength (MPa)
27.5	34,500	2.187

Table 2 Mechanical characteristics of steel bars

Steel bars Ø8, Ø10, Ø12 and Ø16		
Tensile strength (MPa)	Elastic modulus E (MPa)	Density (kg/m^3)
560	200,000	7850

some particles of cement. It can be seen that the numerical model presents a similar rigidity comparing to the experimental one.

Figures 11 and 12 show the failure of the column strengthened using steel bars in experimental test and numerical modeling, respectively. Since the steel bars were anchored

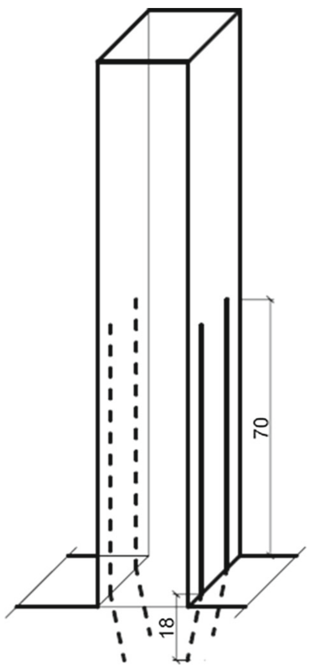

Fig. 6 Layout of *steel bars* [16]

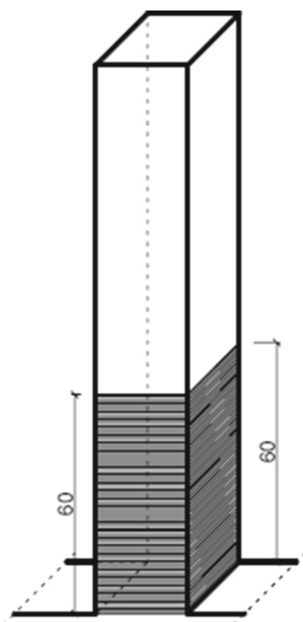

Fig. 7 Layout of GFRP sheets [16]

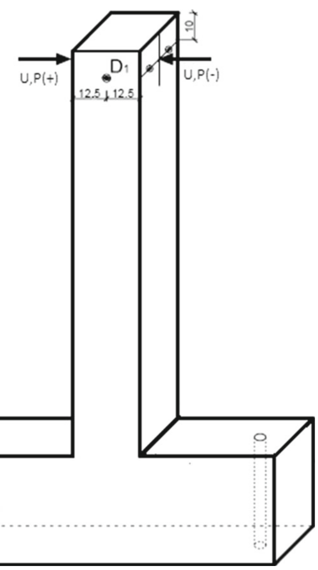

Fig. 8 Displacement and force application points [16]

Table 3 Mechanic characteristics of GFRP and steel bars

Nr.	Material	Elastic modulus E (MPa)	Tensile strength (MPa)	Density (kg/m^3)	Thickness (mm)
1	GFRP	70,000	2250	2530	0.17
2	BM	200,000	560	7850	Ø = 12 mm

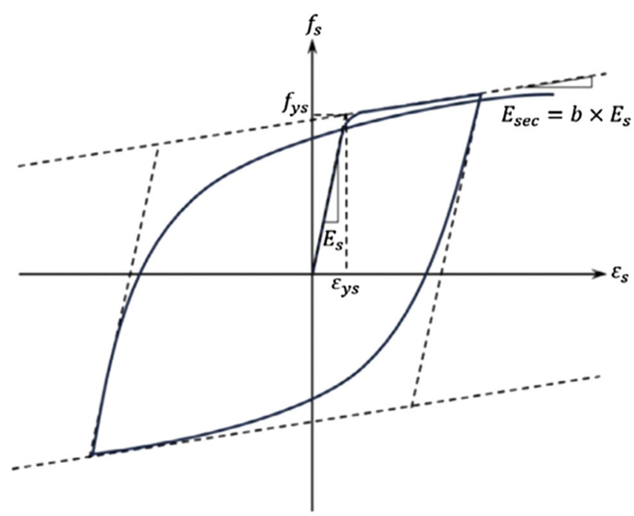

Fig. 9 Menegotto–Pinto stress–strain model [20]

to the support of the column, the base of the column was not fractured while vertical cracks appeared at the contact zone of the concrete with the longitudinal steel bars when the cyclic lateral loading is increased. The observed failure illustrated in Fig. 12 obtained with the numerical model reflects the failure mode of the column in experimental test.

6.2 Column Strengthened with GFRP

Figure 13 shows the behavior of the column strengthened using GFRP in numerical analysis and experimental test. The maximum displacement and load of the numerical model were 77.82 mm and 28.01 KN, respectively, while for the experimental test were 78.34 mm and 31.85 kN, respectively.

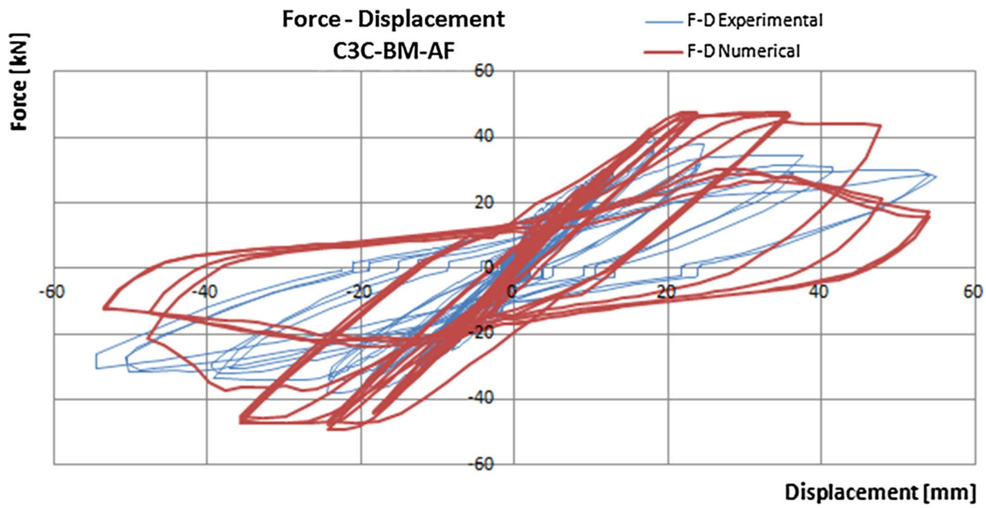

Fig. 10 Curve of comparison between the numerical and experimental test for the column C3C–BM–AF

Fig. 11 Observed failure mechanism of the column C3C–BM–AF in experimental [16]

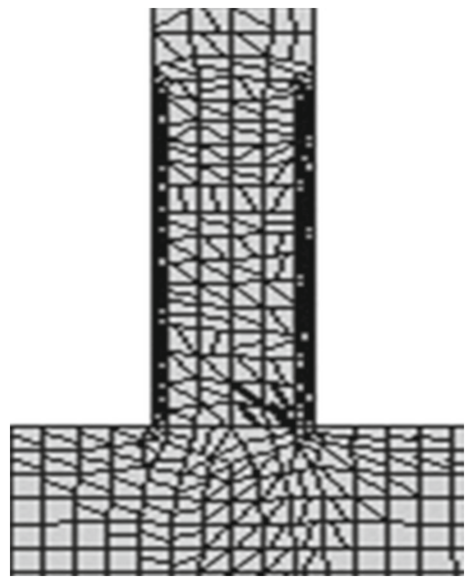

Fig. 12 Observed failure mechanism of the column C3C–BM–AF in numerical modeling

These results showed a small difference in the displacement with 0.67% and a difference with 12.83% of force over in experimental test comparing to the numerical modeling. The slight difference for the numerical behavior is observed in the little smaller areas enclosed by the hysteretic behavior model and a slightly higher peak load capacity. This may be attributed to the numerical model in which perfect bond between concrete and GFRP and negligence of GFRP creep for the short-term cyclic loading were assumed. However, it is seen from the figure that the ductility of the column is predicted close to the ductility achieved by the experimental results. The ultimate displacement is well predicted by the numerical model. In general, the numerical model reflects the behavior of the column experimentally studied.

Although modeling cyclical pattern of material was refined and calibrated to reflect properly the behavior of the column, it should be noted that these types of modeling are sensitive to material changes in microscale that may be encountered in the real elements (aggregate uniform in shape and size, compaction slightly different element on its height, contact FRP-concrete, etc).

Failures of the column studied in experimental test and numerical modeling are illustrated in Figs. 14 and 15, respectively. The crack increases in number and width at the base of the column when the load is increased. Because of the no anchorage of the GFRP material to the support, it is seen

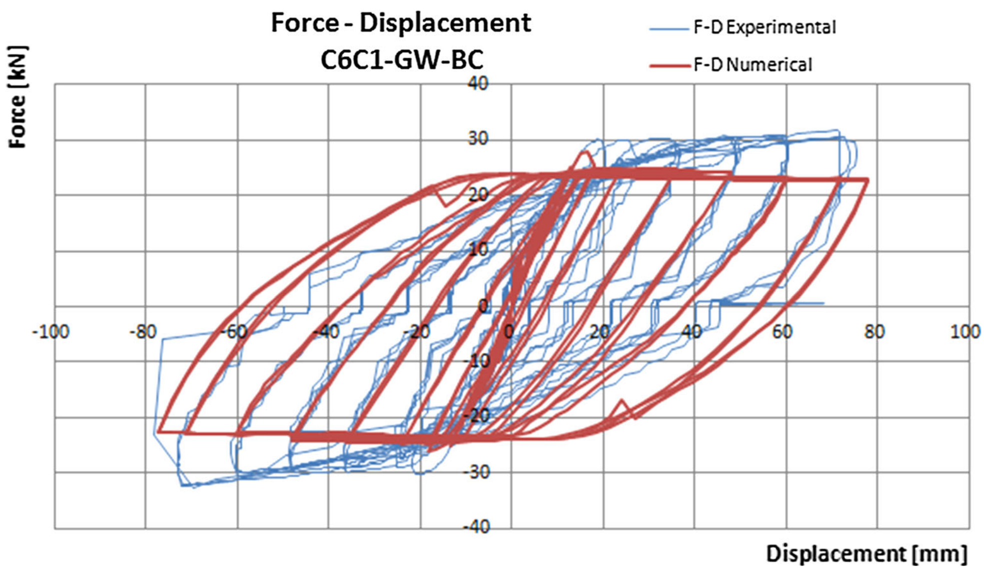

Fig. 13 Curve of comparison between the numerical and experimental test for the column C6C1–GW–BC

Fig. 14 Observed failure mechanism of the column C6C1–GW–AF in experimental [16]

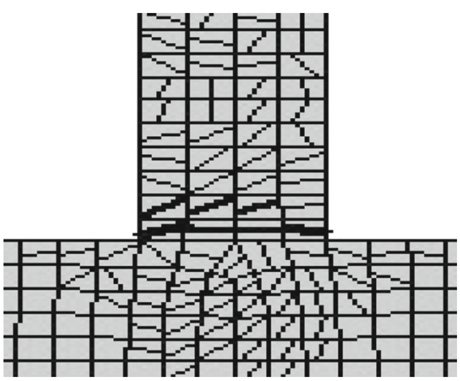

Fig. 15 Observed failure mechanism of the column C6C1–GW–AF in numerical modeling

that the GFRP wrap and the base of the column were fractured when cyclic lateral loading was increased. It is shown from the figures that the numerical analysis reflects the failure mode of the column experimentally tested.

7 Comparative Study on the Behavior of the RC Column Strengthened with GFRP

7.1 Numerical Modeling and Characteristics of the Column

In this part, numerical investigation that evaluates the comparative behavior of RC column strengthened using GFRP, under cyclic lateral loading without axial force, was reported. Parametric study related to the amount of the GFRP material wrapping with varying the number of layers was performed, in order to describe the influence of number of GFRP layers on the RC columns behavior. Three models of square cross sectional columns were modeled under cyclic lateral loading without axial force. The number of GFRP layers was varying from 1 to 3 layers. The computation of these models was performed using the software of analysis of concrete and reinforced concrete structures ATENA [18].

The analysis study was performed on the same column presented in Fig. 3 of the first part, with the same characteristics of the concrete and reinforcement presented in Tables 1 and 2 (see above). The characteristics of the GFRP composite material are shown in Table 4.

The loading application of the column used in this study was performed with the same loading application noted in Sect. 4 for the column C6C1–GW–BC (see above Fig. 7). Perfect elastoplastic model for steel bars was used. For the GFRP material, bilinear model was used.

7.2 Results and Comparison

The numerical results of the column strengthened using GFRP with varying the number of layers from 1 to 3 layers

Table 4 Characteristics of GFRP composites

Material	Elastic modulus E (MPa)	Tensile strength (MPa)	Density (kg/m^3)	Thickness (mm)	Limit strain (%)
GFRP	70,000	2170	2530	0.17	3.1

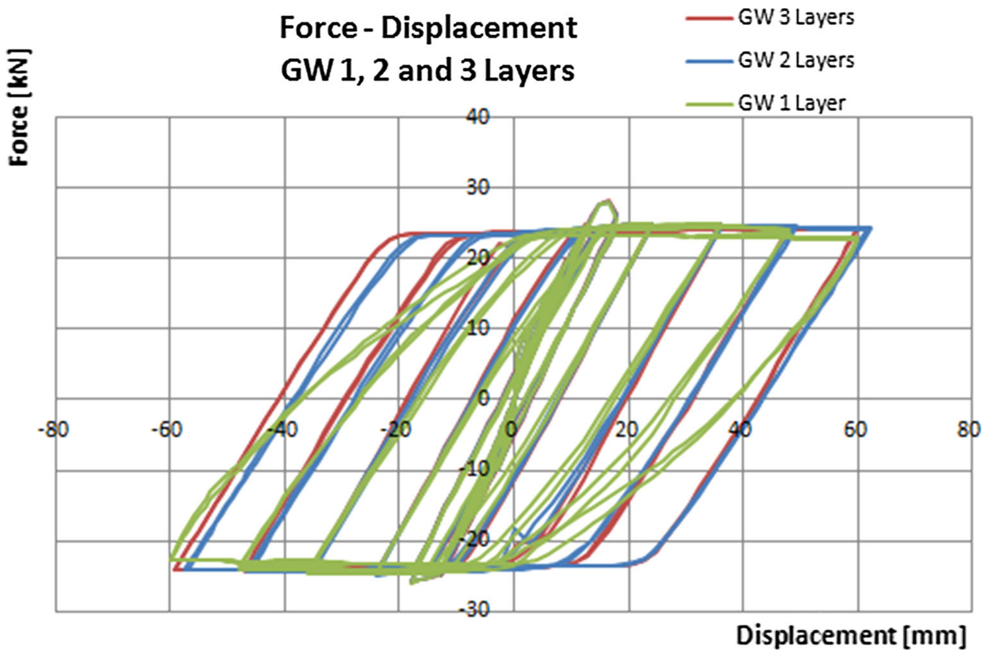

Fig. 16 Behaviors of the column strengthened with 1–3 GFRP layers

are illustrated in Fig. 16. Due to the rupture of GFRP layers, it is seen from the figure that all the columns strengthened using 1, 2 and 3 layers reach their maximal load capacities of 29 kN, while around 60 mm of maximal displacement was estimated. It is illustrated from the figure that adding the number of GFRP layers to the RC columns did not enhance their capacity to support more than 29 kN. This may be because of two reasons. First, the applied load considered in this study was only cyclic lateral loading without an axial force. Second, GFRP material was applied laterally at the base of the column without anchorage to the support. However, it is shown in the figure that after the columns reach 20 mm of displacement, the residual displacement is decreased in the case of the columns strengthened with 2 and 3 GFRP layers comparing to the column strengthened with only 1 GFRP layer. Therefore, it can be seen that adding the number of GFRP layers can be advantageous to limit the residual displacement, and consequently the damage for such a case of strengthening and loading type. This can be improved by comparing the crack width of each column. The obtained values of the crack width corresponding to the maximum load of the columns strengthened with 1, 2 and 3 GFRP layers were 16.9, 11.9 and 10.9 mm, respectively. It can be seen that the values of the crack decrease with increasing the number of layers. It proves that adding the number of GFRP layers limits the apparition of the damage.

In order to describe with more details the behavior of the columns strengthened using different number of GFRP layers (1–3 layers), it was necessary to highlight the different parameters that characterize the behavior of the strengthening material at GFRP material scale. Using ATENA code, the maximum values of partial internal forces (internal force in each node of bar element of GFRP), stress and strain in each bar element of GFRP were obtained. These values are illustrated in Table 5. The first column of the table (Nr) represents the number of GFRP bars in the critical zone starting from the base of the column (see above Fig. 7).

It can be seen from Table 5 that the values of partial internal force, stress and strain change when the number of GFRP layer is increased. It was observed that although the general hysteretic behavior of the columns strengthened with different number of layers is similar in terms of load capacity and maximal displacement, the solicitations at the GFRP materials scale are different. Figures 17, 18 and 19 show the ultimate values of partial internal force, stress and strain at the GFRP material scale of the column strengthened with 1–3 GFRP layers.

From these curves, the difference of the solicitations was observed as follows:

Table 5 Results at GFRP wrap 1, 2 and 3 layers

Nr	GW 1 layer			GW 2 layers			GW 3 layers		
	Partial internal force (KN)	Stress (MPa)	Strain	Partial internal force (KN)	Stress (MPa)	Strain	Partial internal force (KN)	Stress (MPa)	Strain
1	0.10	18.1	0.0002	0.55	52.49	0.0003	1.09	36	0.0005
2	1.52	666	0.0095	2.21	263	0.0038	3.06	189	0.0027
3	3.12	1350	0.0193	3.88	492	0.0070	6.09	357	0.0051
4	3.89	1630	0.0294	5.77	728	0.0104	8.53	531	0.0076
5	3.27	1450	0.0397	7.81	973	0.0139	10.9	713	0.0010
6	4.10	1470	0.0504	10	1230	0.0176	14.30	903	0.0129
7	3.43	1560	0.0454	9.60	1170	0.0167	14.50	848	0.0121
8	4.29	1860	0.0405	9.27	1120	0.0161	14.90	794	0.0113
9	4.94	2060	0.0355	9.64	1070	0.0153	15.10	739	0.0105
10	5.69	2100	0.0306	10.10	1020	0.0146	15.40	685	0.0098
11	5.06	1790	0.0256	10.40	970	0.0138	15.60	629	0.0090
12	4.17	1560	0.0223	9.26	862	0.0123	13.60	557	0.0080
13	3.40	1330	0.0189	8.11	753	0.0107	11.60	486	0.0069
14	2.78	1090	0.0156	6.96	645	0.0092	9.71	415	0.0059
15	2.15	861	0.0123	5.80	535	0.0076	7.73	343	0.0049
16	1.54	627	0.0090	4.63	425	0.0061	5.83	272	0.0039
17	1.33	550	0.0079	3.71	372	0.0053	5.05	238	0.0034
18	1.12	474	0.0067	2.78	319	0.0045	4.26	204	0.0029
19	0.91	397	0.0057	2.16	265	0.0038	3.48	171	0.0024

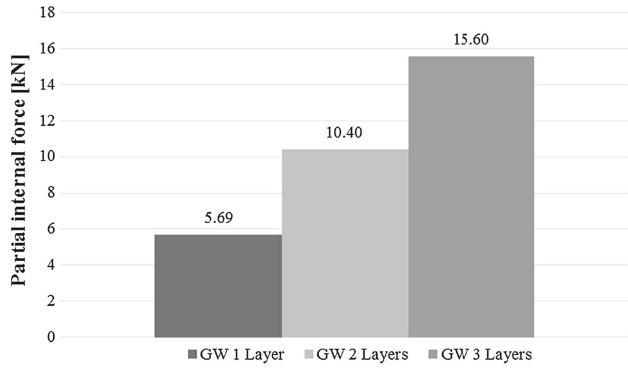

Fig. 17 Ultimate values of partial internal force of the column with 1–3 GFRP layers

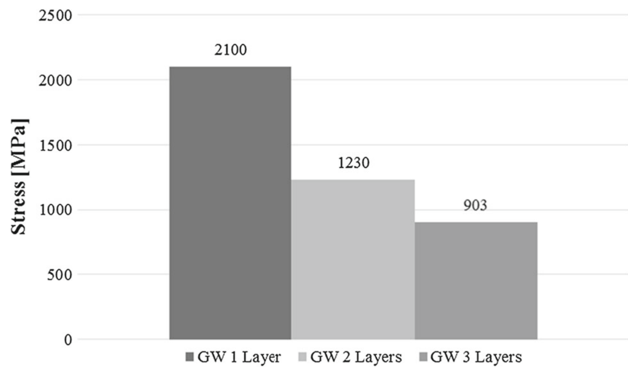

Fig. 18 Ultimate values of stress of the column with 1–3 GFRP layers

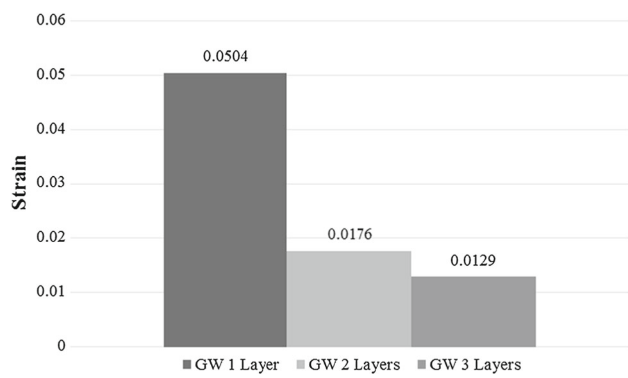

Fig. 19 Ultimate values of strain of the column with 1–3 GFRP layers

– The internal fore at the GFRP material increased with 82.78% when the second layer was added and passed to 174.16% when the third layer was added.
– The stress at the GFRP material decreased with 70.73% when the second layer was added and passed to 132.55% when the third layer was added.
– The strain at the GFRP material decreased with 186.36% when the second layer was added and passed to 290.69% when the third layer was added.

8 Conclusions

Numerical investigations were performed on square RC columns strengthened with composite materials. These numerical investigations were presented in two parts. In the first part, comparative study between numerical modeling and experimental tests of RC columns strengthened with GFRP and steel bars was presented. It was concluded that both numerical models for the columns strengthened with GFRP and steel bars reflect behavior of the experimentally tested ones. This gives confidence to the design engineers and researchers in using finite element modeling for evaluating the cyclic behavior of RC columns strengthened with different types of composite materials.

In the second part, comparative study on the RC column with varying the number of GFRP layers (from 1 to 3) was performed. The results showed similarity of the hysteretic behavior in terms of load capacity and maximal displacement. However, decreasing of residual displacement and limiting the crack width (damage) were observed when additional GFRP layers were added. This proves the efficacy of adding the number of layers in such a case of strengthening square RC columns subjected to cyclic lateral loading. Furthermore, it was showed that although the general hysteretic behavior of the columns strengthened with different number of GFRP layers is similar in terms of load capacity and maximal displacement, the solicitations at the GFRP materials scale are different, namely partial internal force, stress and strain. It was concluded that varying the number of GFRP layers influence the results at the local scale. For this purpose, it will be necessary to study the FRP composites behavior at the material scale in order to describe the realistic behavior of the RC columns strengthened with composite materials. It will concern a multi-level modeling of the columns strengthened with composite materials from mesoscale to macroscale models. The influence of some parameters (damage, stress, strain, cracks, etc) at the finer scale on the behavior of the macroscale model has to be determined. It may be appropriate to include these parameters in the future works.

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