

Upgrading the Seismic Performance of Reinforced Masonry Columns Using CFRP Wraps

Khaled Galal, M.ASCE¹; Nima Farnia²; and Oscar A. Pekau³

Abstract: Compared with reinforced concrete, relatively fewer experimental studies address the behavior of masonry columns under combined axial load and cyclic flexure. There exist reinforced concrete masonry (RCM) columns that are part of the moment resisting system of masonry structures that are in need of seismic upgrade. Wrapping such susceptible RCM columns with carbon fiber-reinforced polymers (CFRP) is expected to enhance the seismic behavior of reinforced masonry columns considerably. This paper focuses on assessing the seismic performance of RCM columns wrapped with CFRP. In this experimental study, six 1.4-m reinforced masonry columns were constructed and tested when subjected to constant axial force and cyclic lateral excitations. The columns had a cross-section of 390 mm × 390 mm and were constructed using bull-nosed concrete units. The first column had no CFRP wraps and was used as a control specimen whereas the other five columns were wrapped using different layers of CFRP sheets or different wrapping schemes. From the tests, it was observed that wrapping the masonry columns with CFRP wraps enhanced the seismic performance of the columns by offering more ductile behavior, increasing both strength and energy dissipation capacity. DOI: 10.1061/(ASCE)CC.1943-5614.0000252. © 2012 American Society of Civil Engineers.

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Introduction

Devastating and deadly earthquakes occur around the world constantly. In major earthquake events, losses are considerable because of buildings' collapse and, consequently, human casualties. When it comes to masonry structures, many existing structures are not able to resist (or have unsatisfactory performance) against future medium to high ground motions. Most of these buildings have common deficiencies such as poor proportioning of members causing strong beam and weak columns, soft stories, or non ductile performance resulting from the occurrence of short column mechanism.

For reinforced masonry columns that are part of the moment resisting system, recent research showed that confinement of such members, especially near the potential plastic hinging regions, is not enough (Youd et al. 2000). Because demolishing and reconstructing such deficient structural elements is not always preferable, retrofitting them to meet appropriate seismic ductility demands becomes a necessity.

For retrofitting and upgrading purposes, fiber-reinforced polymer (FRP) composite materials have become a popular material in strengthening of reinforced concrete elements in recent decades.

It offers attractive characteristics such as high strength and high stiffness-to-weight ratio, and is light weight for ease of application with minimal interruption to occupants (Seible 1997).

Most research efforts in retrofitting deficient masonry structural elements using FRP were directed to masonry walls, so less work has been conducted on retrofitting reinforced masonry columns. However, significant efforts have been made in evaluating the performance of FRP-rehabilitated plain and reinforced concrete (RC) columns (e.g., Shrive et al. 2003; Galal et al. 2005; and Paultre et al. 2008). In general, previous research showed that wrapping non ductile RC columns with carbon fiber-reinforced polymers (CFRP) sheets is an effective form of increasing the column's ductility and, hence, seismic performance. To increase confinement, FRP wraps are laid perpendicular to the column's axis. However, the wrap is not activated until the concrete is dilating substantially as it is failing (Shrive et al. 2003). For experimental studies on concrete masonry columns retrofitted with fiber-reinforced polymer (FRP) composites, bull-nosed units can be used to provide rounded corners on the tested columns (Masia and Shrive 2003). Concrete blocks with chamfered corners have been recommended to minimize stress concentration at the columns, edges and to increase the effectiveness of the wrap (Shrive et al. 2003).

In recent years, strengthening different types of masonry columns with FRP-material attracted the attention of various researchers. Micelli et al. (2004) studied the behavior of brick masonry columns with different masonry material and external FRP wrapping to quantify the compressive strength of confined masonry columns. Krevaiakas and Traintafillou (2005) proposed a strength model for FRP-wrapped rectangular columns made of clay bricks. In addition, FRP confinement of rectangular specimens made of limestone and clay bricks were studied by Aiello et al. (2009). Recently, Ludovico et al. (2010) provided more data on behavior of a FRP-wrapped tuff and clay brick masonry section. Previous research has shown that the failure in wrapped masonry columns subjected to axial load only appear to be initiated by complete crushing of the mortar joints, causing wrinkles in the wrap,

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followed by explosive rupture of the CFRP wrap. First the FRP material rips vertically, then very rapidly circumferentially, at the failure of masonry columns.

In this experimental program, wrapped columns were tested when subjected to constant axial load and increasing cyclic lateral excitations. To the authors' knowledge, the literature surveyed did not reveal a similar experimental program on the behavior of concrete masonry columns.

Experimental Program

In this research, the tests were carried out in two phases: (1) the auxiliary tests that are meant to provide the mechanical characteristics of the constituent materials and the masonry assemblage; and (2) the six $390 \times 390 \times 1,400$ -mm reinforced masonry columns that were constructed and tested when subjected to constant axial force and cyclic lateral excitations. All of the auxiliary specimens (masonry prisms) and columns were constructed by domestic professional masons representing the current method of practice in Québec, Canada.

Material Properties and Auxiliary Tests

The constituent materials used in the construction of reinforced concrete masonry (RCM) columns consist of mortar, grout, concrete masonry blocks, steel rebars/wires, and CFRP sheets. Mechanical properties of carbon fiber, CFRP sheets (with unidirectional fabric), and epoxy resin are summarized in Tables 1–3, as it was provided by supplier.

Mortar

In this study, Type S mortar that is a mix of 0.5 volumetric unit Portland cement, 1 unit masonry cement, 2.9 units sand, and 0.7 unit water was chosen, after several trial mixtures, to conform with the requirements stated in ASTM (2002b) and CSA (2004a). A series of six 50-mm mortar cubes were sampled, and after curing in a humid environment, tested according to ASTM (2002e) at the age of 7 and 28 days. The average 7- and 28-day compressive strengths were 8.6 and 20.7 MPa (with coefficient of variation of 9.6 and 4.9%), respectively. The former should not be less than 7.5 MPa and the latter should not be less than 12.5 MPa according to CSA A179-04 (2004) and ASTM (2002b) for laboratory-made mortar cubes.

Grout

The grout used in the study, categorized as “coarse grout” in accordance with CSA (2004a) and ASTM (2002a), consisted of a series of six grout cylinders with a mixture of 1.0 volumetric unit Portland cement, 2.8 units fine aggregate (sand), 2.0 units coarse aggregates with the maximum size of 7 mm, and 0.9 units water were sampled and tested. The average compressive strength at the age of 7 and

Table 1. Properties of the Dry Carbon (As Provided by the Supplier, [Fyfe Co. 2009](#))

	Dry carbon
Tensile strength	3.79 GPa
Tensile modulus	230 GPa
Ultimate Elongation	1.60%
Density	1.8 gr/cm ³
Weight	315 gr/m ²

Table 2. Properties of the Tyfo SCH-11UP CFRP Sheets (As Provided by the Supplier, [Fyfe Co. 2009](#))

	CFRP sheets
Tensile strength	1062 MPa
Tensile modulus	102 GPa
Elongation at break	1.05%
Thickness	0.27 mm

Table 3. Properties of the Tyfo S epoxy (As Provided by the Supplier, [Fyfe Co. 2009](#))

	Epoxy
Tensile strength	72.4 MPa
Tensile modulus	3.18 GPa
Elongation percent	5%
Flexural strength	123.4 MPa
Flexural modulus	3.12 GPa

28 days were 11.7 and 21.6 MPa (with coefficient of variation of 10.3 and 5.8%), respectively. A slump test of fluid grout was done every day during the construction period ranging from 260- to 280-mm slump that passes the lower limit stated in the Canadian code (CSA 2004b).

Vertical and Transverse Reinforcement

Steel reinforcement used in the tested columns was conventional 15 M steel rebars (grade 400) with 412 MPa yield strength. Stirrups were made from steel wires with 4.65-mm diameter with a yield strength of 240 MPa; they were used as transverse reinforcement.

Concrete Masonry Units

In this study, masonry units were hollow concrete blocks with nominal dimensions of $390 \times 190 \times 190$ mm, and two bull-nose corners with a radius of 25 mm. The average tested compressive strength of the unit was 15.3 MPa (coefficient of variation of 8.2%) with the average net-to-gross area ratio of 0.7.

Auxiliary Tests

After obtaining mechanical properties of constituent materials, two series of auxiliary tests were conducted to obtain the compressive and tensile strength of the masonry assemblage.

To obtain the compressive strength of the masonry assemblage, f'_m , a series of five unreinforced grouted prisms were tested according to ASTM (2002c) as shown in Fig. 1. Five-block-high and one-block-wide prisms were built to better represent the real changes in the masonry columns. From the tested prisms, compressive strength



Fig. 1. Tests for obtaining compressive (left) and tensile (right) strength of masonry assemblage

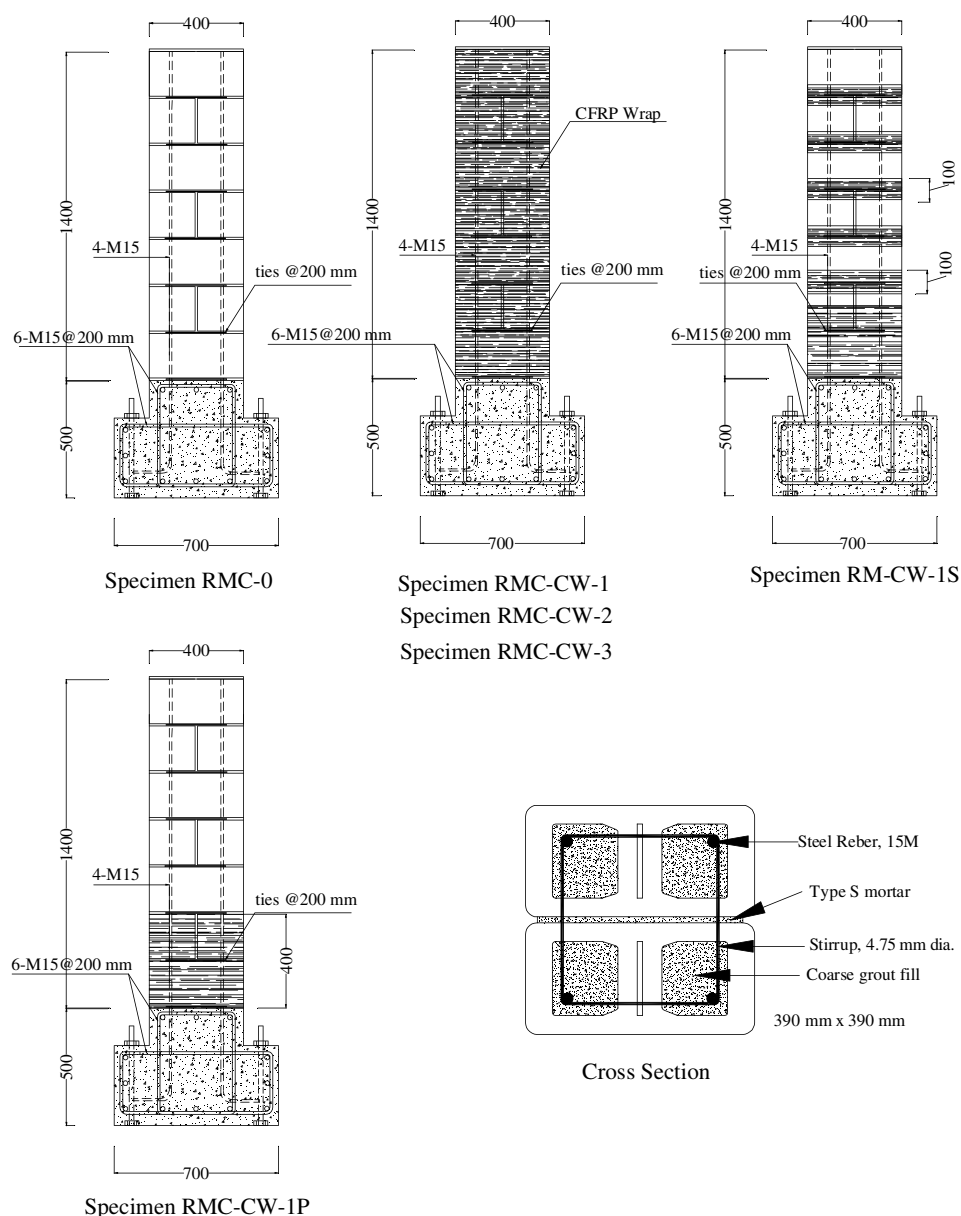
Table 4. Designation of Specimens

Specimen	Designation
Reinforced masonry column: Control specimen	RMC-0
Reinforced masonry column with 1 layer of CFRP wraps	RMC-CW-1
Reinforced masonry column with 2 layers of CFRP wraps	RMC-CW-2
Reinforced masonry column with 3 layers of CFRP wraps	RMC-CW-3
Reinforced masonry column with 1 layer of CFRP strips	RMC-CW-1S
Reinforced masonry column with 1 layer of CFRP wrap at plastic hinge	RMC-CW-1P

of masonry assemblage was obtained equal to 11.5 MPa with a coefficient of variation of 14.8%.

The Young's modulus, E , of the prisms was also determined to be 8.1 GPa on the basis of a secant line between 5 and 33% of the average ultimate load. CSA (2004a) specifies modulus of elasticity, E , using the expression $850 \times f'_m$ for concrete masonry. Using this expression, E is calculated equal to 9.8 GPa, which overestimates

the reported value. Typical splitting of the concrete blocks was observed as the dominant mode of failure initiated by cone and shear cracks. Previous studies have attested that the grout properties are the major factor affecting the failure of grouted concrete masonry, which may occur even at lower stress levels compared with ungrouted concrete masonry (Dhanasekar and Shrive 2002). The relatively lower strength of the prisms of this program, compared

**Fig. 2.** Dimensions and details of reinforcement of the six tested RCM columns

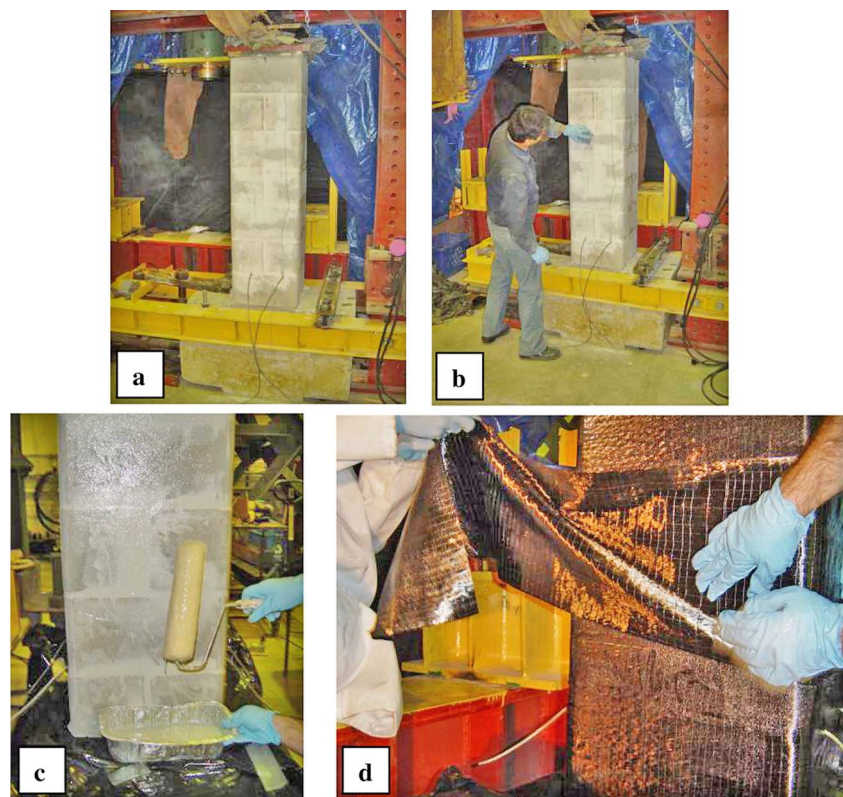


Fig. 3. (a) Covering all mortar joints and grooves; (b) removing any unevenness and bumps from the surface; (c) saturating test column with epoxy; (d) overlaying CFRP sheets for column RMC-CW-2

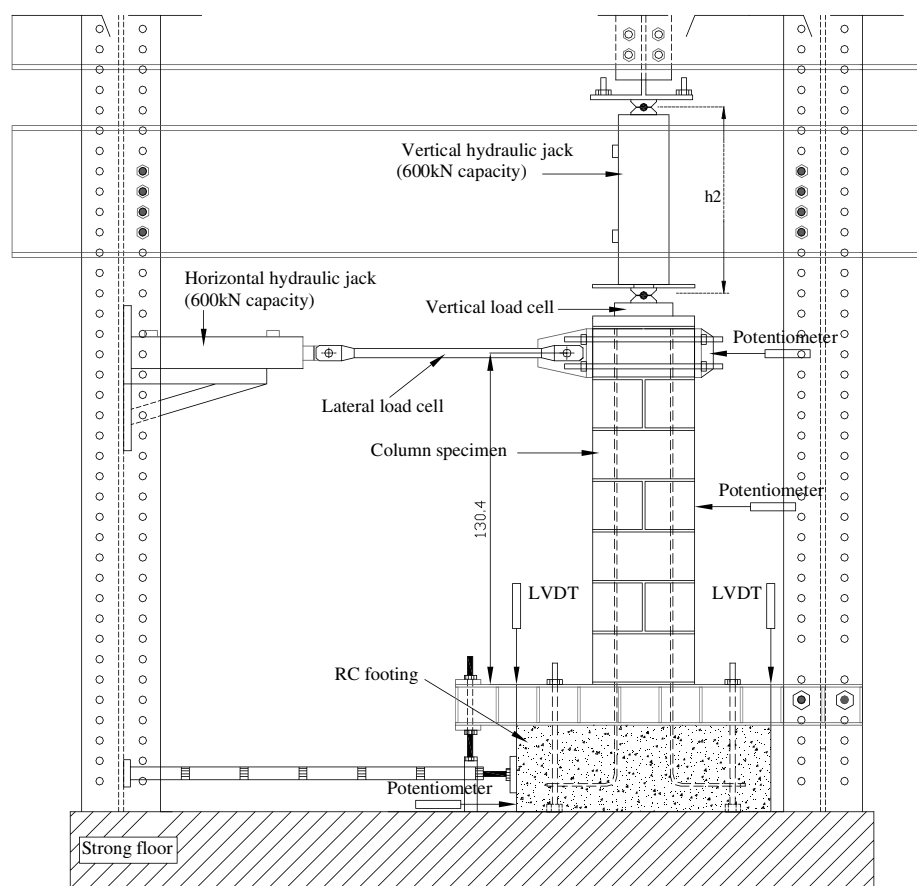


Fig. 4. Test setup and instrumentation

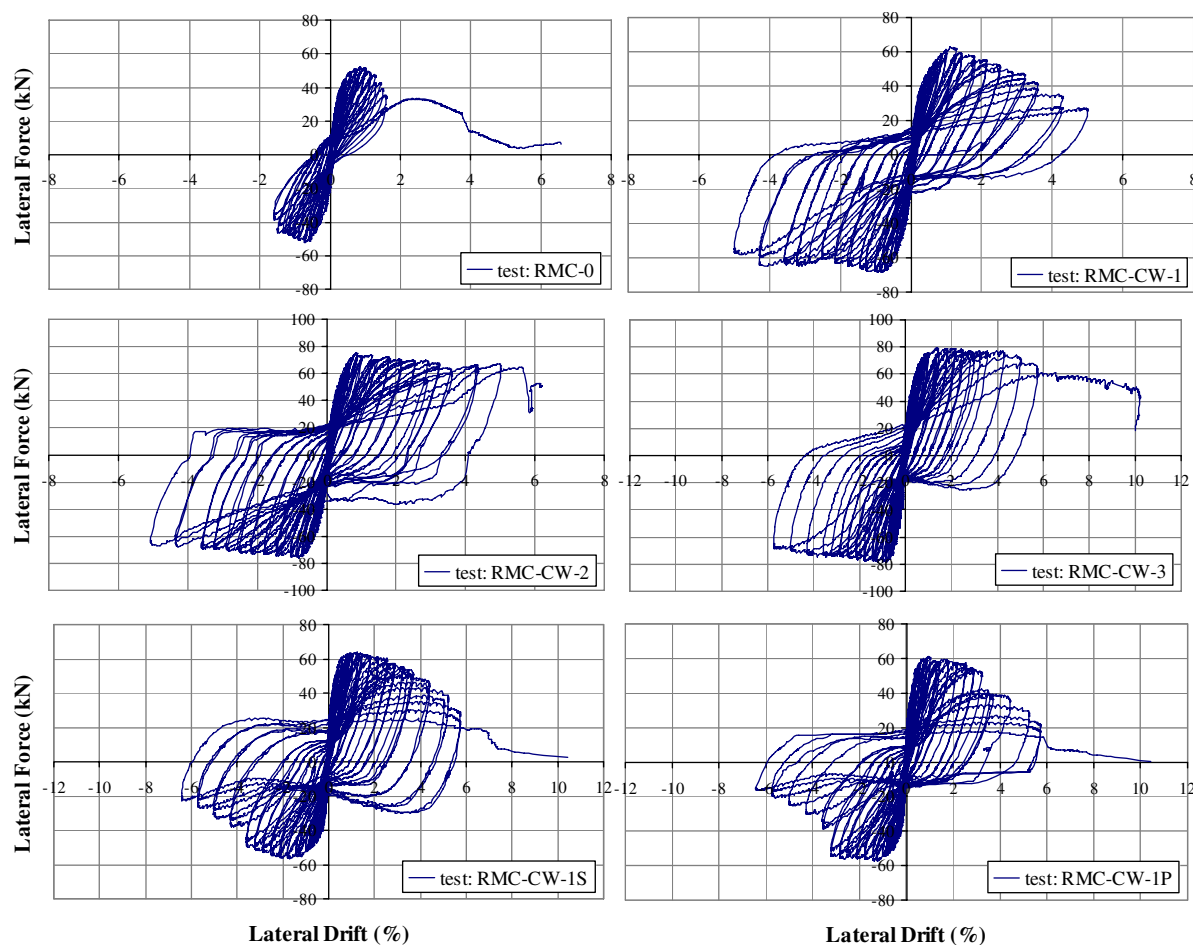


Fig. 5. Lateral loads–lateral drift relationships of the tested columns

with their constituents, can be attributed to the excess of grout expanding inside the cells that resulted in splitting the face shells. Similar behavior was observed in the concrete masonry prisms that were tested by Priestley and Elder (1983). The grout cores that were found intact at the end of all five tests also account for this observable fact.

For the purpose of estimating the tensile strength of masonry assemblage, a series of five prisms were tested under a four-point loading method according to ASTM (2002d) guidelines. As shown in Fig. 1, prisms with a height of seven blocks and a width of one block were constructed to properly locate the two point loads and supports, and to provide a sufficient span-to-depth ratio. From the tested prisms, a tensile strength, f_t , of 1.3 MPa with a coefficient of variation of 12.4% was obtained. The tensile strength of masonry assemblage, f_t , is used for calculating the moment of cracking on tested columns.

Tests Specimen

In the second phase of this experimental study, six $390 \times 390 \times 1,400$ -mm reinforced masonry columns were constructed. The tested columns represented the lower half of a 2.8-m-long column with a fixed base condition by means of a reinforced concrete footing that was fixed to the strong floor and a free top to represent the inflection point of the original column. Table 4 shows the designation of the tested columns. Each column was reinforced with four 15-M vertical steel bars and 4.75-mm steel stirrups on every row as shown in Fig. 2. The first column was tested as a control specimen

without any CFRP wraps and the rest of the columns were wrapped using different layers of CFRP sheets or different wrapping schemes as shown in Fig. 2. In the wrapping process, all grooves and unevenness on the surface of the masonry columns, like mortar joints, were filled with high strength cementitious-based plaster to provide a flat and even surface as shown in Fig. 3. To ensure the effectiveness of the wrapping, 150-mm overlap was used as recommended by the ISIS Canada (2008) design manual.

Test Setup, Loading Program, and Instrumentation

The tested columns were subjected to a constant axial force of 200 kN (providing a stress of 0.11 of the axial compressive capacity of the gross-section area of the tested columns) and cyclic lateral excitations. Tests were conducted in a displacement-control mode in which two cycles were applied at each displacement level having 2-mm intervals (i.e., 2 mm, 4 mm, 6 mm, and so on) until 3% lateral drift (40 mm), after which larger displacement intervals of 10 mm were applied. The columns were considered to have failed when one of the following two limit conditions were reached: (1) the lateral load at the positive or negative direction was reduced to 0.80 times the maximum recorded lateral load during each test; or (2) the column could not sustain an axial load and lateral load resulting from FRP rupture and brittle crushing of the concrete masonry (primarily at the base of the column or plastic hinge zone). Fig. 4 shows a schematic of the test setup and the location of potentiometers used for displacement measurement.

The tested columns were instrumented to monitor the displacement and strain measurements using a data acquisition system. Four strain gauges were installed on vertical steel rebars at the location of the column and footing interface. Two horizontal linear potentiometers were attached to each side of the column along the point of application of the lateral load, and two horizontal potentiometers were used to measure the top and middle height displacements. Three potentiometers were installed on the RC footing to ensure there was no rotation relative to the strong floor. Furthermore, three strain gauges were installed on the bottom, middle, and top of the column on the surface of the CFRP wrap. Also, strain gauges were installed on the bottom, middle, and top ties.

Test Results

Fig. 5 shows the lateral load, F , versus lateral drift hysteretic relations of tested columns. Lateral loads are adjusted as a result of contribution of the horizontal component of vertical load to the applied lateral load (especially at high lateral displacements). Therefore, the adjusted lateral load is defined as

$$F = V + P \cdot \frac{\Delta}{h_2} \quad (1)$$

where V and P = applied lateral and vertical applied loads; Δ = lateral displacement of the tip of the column; and h_2 = distance between two hinges on the top of the column. Because displacements are relatively small, contribution of vertical component

V would not have a significant effect on the level of applied axial load.

Before the application of the lateral load, columns were first loaded with a constant axial load of 200 kN. As shown in Figs. 6(a) and 6(b), the first crack in the control column RMC-0 was observed at 0.7% drift. Cracks widened during subsequent cycles at the same displacement level. The yielding of vertical rebars started after 1% lateral drift. The new diagonal cracks that were formed after cyclic loading resulted in reduction in the lateral-load bearing capacity. The most extensively damaged region shifted above the first mortar bed joint (i.e., approximately 200 mm away from the column-footing interface) because of the additional confining effect of the footing. Similar results have been observed in tests of reinforced concrete columns tested under seismic and axial loads (Sheikh and Yau 2002). After reaching a 1.6% lateral drift, Column RMC-0 experienced a significant reduction in the lateral-load bearing capacity; thus, a monotonic push load was applied to the column up to failure, which occurred at a 6.6% lateral drift [Fig. 6(c)].

Because of the confinement provided by one layer of CFRP wrap on column RMC-CW-1, it was not possible to monitor cracking of the original masonry column; however, a minor rupture of CFRP wrap was observed at the column-footing interface (see Fig. 6(d)). The lateral-load-carrying capacity did not increase after the drift level of 1.5% and it decreased gradually after increasing the drift. The maximum lateral load reached during the test was 68 kN at a 1% drift level (see Fig. 5). The column was considered failed at a 2.8% drift level when the lateral-load resistance

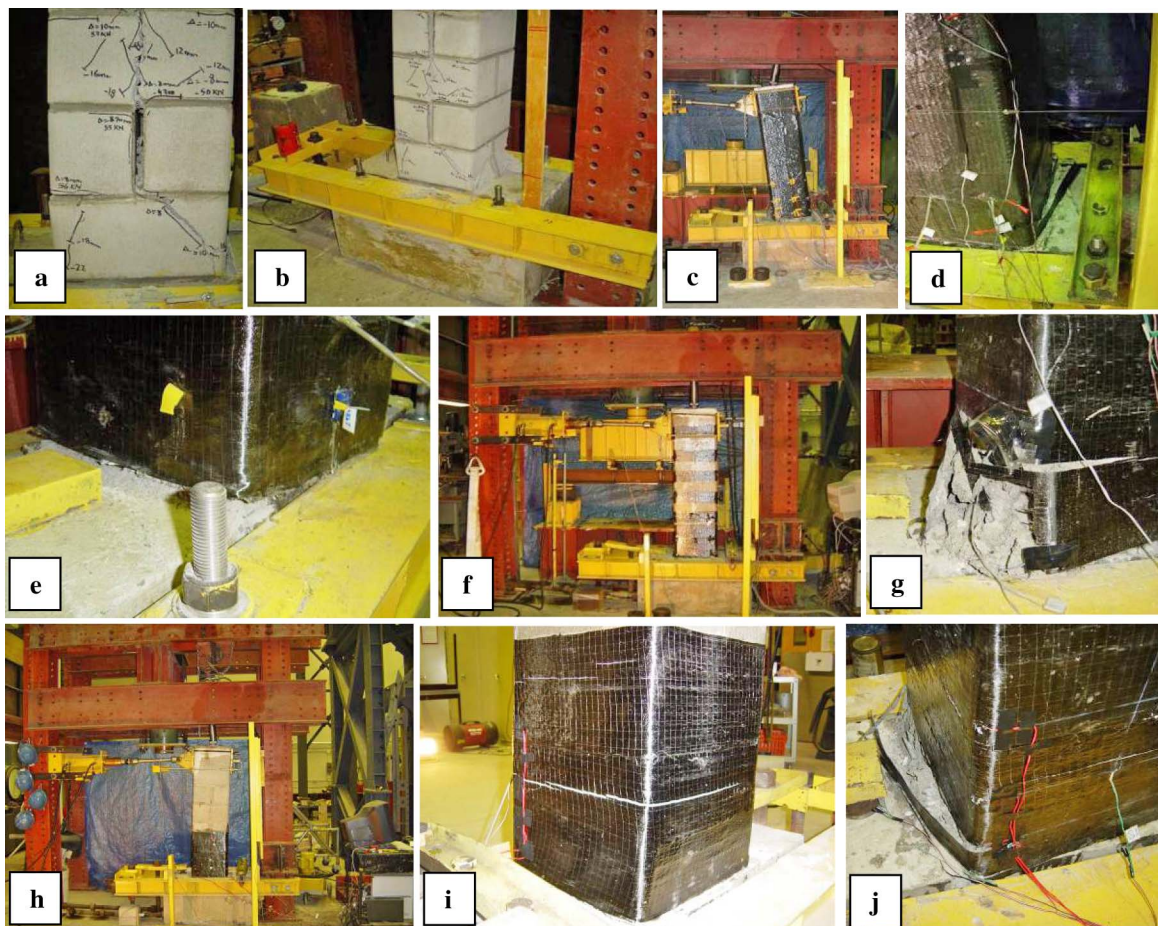


Fig. 6. Columns' performance at different stages of tests including observed cracking pattern and behavior at failure

decreased to less than 80% of the maximum recorded lateral-load capacity. The test was stopped at a 5% drift level.

As for column RMC-CW-2, similar to the previous column, the lateral load did not increase after a drift of 1.5%, after which it started to decrease gradually. The maximum lateral load reached during the test was 76 kN at the 0.9% drift level (see Fig. 5). Because of the limitation in stroke of the horizontal hydraulic jack used in this test, it was not possible to continue the lateral drift beyond a 6% drift, and up to that lateral-displacement level no rupture or damage had occurred in the CFRP wraps [as shown in Fig. 6(e)]; however, minimal local debonding of the wrap was observed. After the 5% drift level, the specimen was monotonically pushed and the test was stopped at a 6% drift level.

In column RMC-CW-3, the lateral load did not increase after a drift of 1.5% and it started decreasing gradually after a 4% drift (see Fig. 5). As shown in Fig. 5, the maximum lateral load reached during the test was 79.7 kN at a 1.5% drift level. The test continued until it reached a 10% lateral drift without rupture or damage to the CFRP wraps; however, local debonding of the wrap was observed. In this test, the limitation in the stroke of lateral hydraulic jack was corrected, and after a 5% drift level the specimen was pushed only until the end of the test. After removing the CFRP sheets on columns RMC-CW-1, 2, and 3 after being tested, the crushed pieces of masonry could easily be removed from the bottom 200 mm of the column.

In an attempt to optimize the CFRP material used in wrapping the columns, another rehabilitation scheme was proposed instead of wrapping the masonry column along its full height. RMC-CW-1S was wrapped with one layer of CFRP strips at the plastic hinge zone (bottom 300 mm) and at the bed joints, as shown in Fig. 6(f). After applying cyclic excitations, the lateral load did not increase after a drift level of 1.2%, after which it started decreasing gradually (see Fig. 5). The maximum lateral load reached during the test was 63 kN, which is approximately 8% less than the maximum recorded lateral load for column RMC-CW-1. Column RMC-CW-1S failed at a 3.5% drift when the lateral load decreased to less than 80% of the column's maximum recorded lateral load. After a 5% drift, the column was only monotonically pushed, and the ultimate failure occurred at a 10% drift because of a sudden rupture of CFRP wrap that occurred simultaneously with an explosive crushing of concrete masonry [see Fig. 6(g)]. The most extensive damage occurred at the bottom 300 mm of the column-footing interface.

Because no cracks were observed above the plastic hinge zone while testing column RMC-CW-1S, column RMC-CW-1P was wrapped only at the base of the column (i.e., at the plastic hinge location), as shown in Fig. 6(h). In general, as shown in Fig. 5, this column behaved very similarly to column RMC-CW-1S, and the maximum recorded lateral-load capacity during the test was 60 kN, which is slightly less than the maximum lateral load obtained from testing column RMC-CW-1S. As in Fig. 6(i), the CFRP jacket opened up at several locations along the height at a 5.7% lateral drift. No cracks were observed above the wrapped zone. Similar to the RMC-CW-1S column RMC-CW-1P failed at a 10% drift when the CFRP wrap ruptured and the masonry concrete crushed at the base of the column [see Fig. 6(j)].

Fig. 7 shows the variation of transverse strains on the CFRP wrap along the column height for columns RMC-CW-1S and RMC-CW-2. Strain values are obtained from the strain gauges installed on the rounded corners of the wrapped sections. In column RMC-CW-1S, the rupture of the CFRP wrap occurred at a recorded strain of 0.82% where it initiated from one of the column's corners. Conversely, the maximum transverse strains of RMC-CW-2 did not exceed 0.31% when the test was stopped at a 6% lateral drift. These

strain readings reflect a combined effect of local expansion resulting from concrete confinement and diagonal tension caused by shear. The maximum strains in the CFRP wraps occurred at the location of first mortar bed joint. A similar pattern of CFRP strain readings were previously reported for reinforced concrete columns confined with CFRP tubes (Ozbakkaloglu and Saatcioglu 2006). The distribution of the CFRP tensile strain along the height of the tested rehabilitated columns can be used to determine the plastic hinge length in RCM columns, which generally falls within a one-column dimension from the base of the column. Moreover, as previously observed, a larger amount of confining FRP results in a smaller damaged region (Iacobucci 2001), and consequently a shorter plastic hinge length. The strain readings above the plastic hinge zone, where shear stresses are dominant in comparison to flexure stresses, are very small and almost negligible.

Rupturing strains of 0.82–0.85% were recorded for columns RMC-CW-1S and RMC-CW-1P, which is less than rupturing strains obtained from the coupon test. This could be attributed to the combined effect of flexural and shear stresses on the CFRP sheets or the presence of a curvature at the corners, which made it rupture at lower tensile stress.

Performance of CFRP-Wrapped RCM Columns

Improvement in Lateral-Load Bearing Capacity

Fig. 8 shows the percentage gain of lateral-load carrying capacity versus volumetric ratio of FRP material, ρ_{FRP} , for all tested columns. In this figure, the maximum lateral load is defined as the average of the maximum lateral load in the push and pull directions, and volumetric ratio of CFRP reinforcement (ρ_{FRP}) is calculated as follows (Roca 2007):

$$\rho_{FRP} = \frac{2 \times n \times t_f (b + h) b_f}{b \times h} \frac{1}{S_f} \quad (2)$$

where n = number of CFRP layers; t_f = thickness of CFRP sheet; b and h = section dimensions; b_f = width of CFRP strips in partial wrapping; and S_f = pitch in partial wrapping. Fig. 9 shows the envelope of hysteretic loops for all tested columns. The general trend shows that lateral-load bearing capacity and the displacement ductility of the rehabilitated columns increases by increasing the number of CFRP layers. It is observed that increasing the CFRP wraps from 1 to 2 layers results in an increase in the maximum lateral-load capacity by approximately 10.5%. The optimum number of CFRP layers depends on the section dimension and radius of the rounded corners (Aiello et al. 2007). Therefore, for the tested columns in this study, increasing the CFRP sheets beyond three layers would not have a significant additional effect on increasing the lateral-load bearing capacity.

Improvement in Displacement Ductility

An important indicator of seismic performance is the displacement ductility factor, μ_Δ , defined as Δ_u/Δ_y where Δ_y is yield displacement and Δ_u is the ultimate displacement. To determine the yield displacement, an equivalent elasto-plastic relationship is defined. The elastic branch of this system is the secant to the real force–displacement relationship at 75% of the maximum lateral load and is extended to reach the maximum lateral-load level to estimate the displacement at yield. The failure of the column was defined at the post-peak displacement, Δ_u , where the remaining capacity dropped to 80% of the peak load (Park 1989). Table 5 shows the ductility factors for each column in both directions: push and pull. Unlike the slight increase in the load-carrying capacity

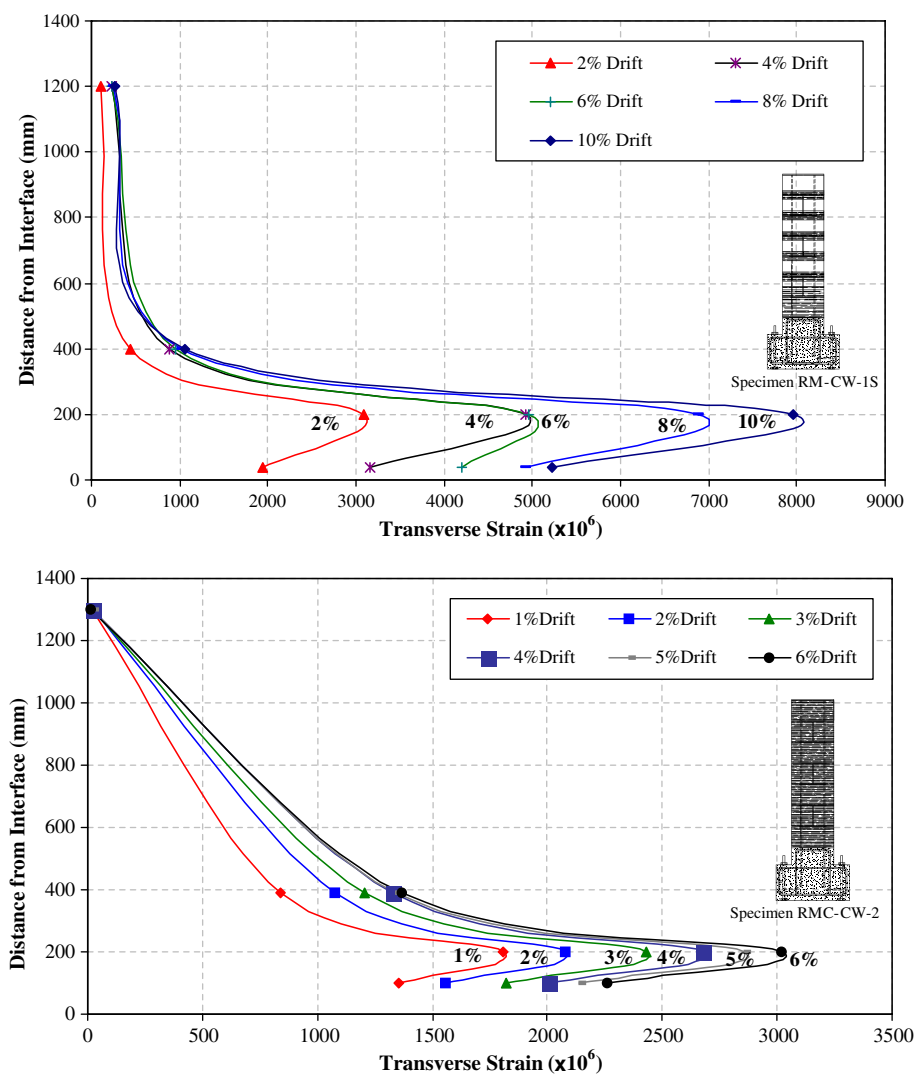


Fig. 7. Variation of transverse strains on CFRP wrap along column height for columns RMC-CW-1S and RMC-CW-2

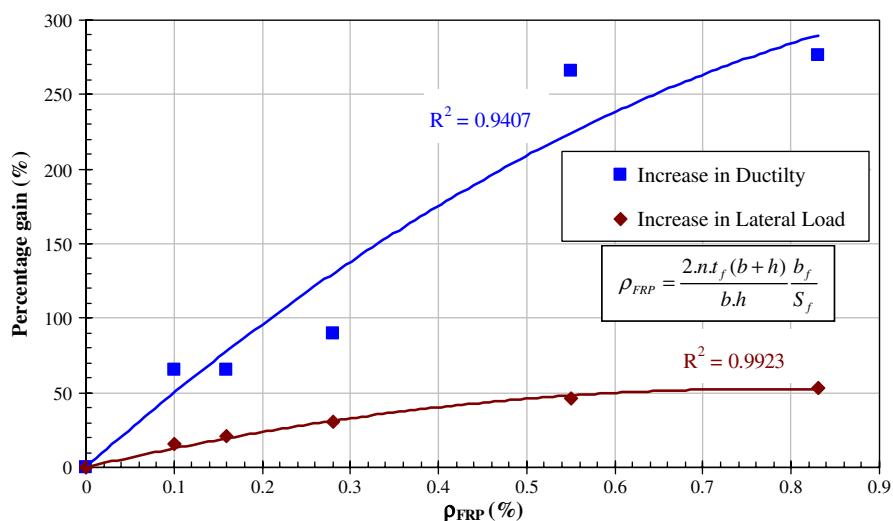


Fig. 8. Percentage gain in lateral-load carrying capacity and displacement ductility versus volumetric ratio of CFRP material

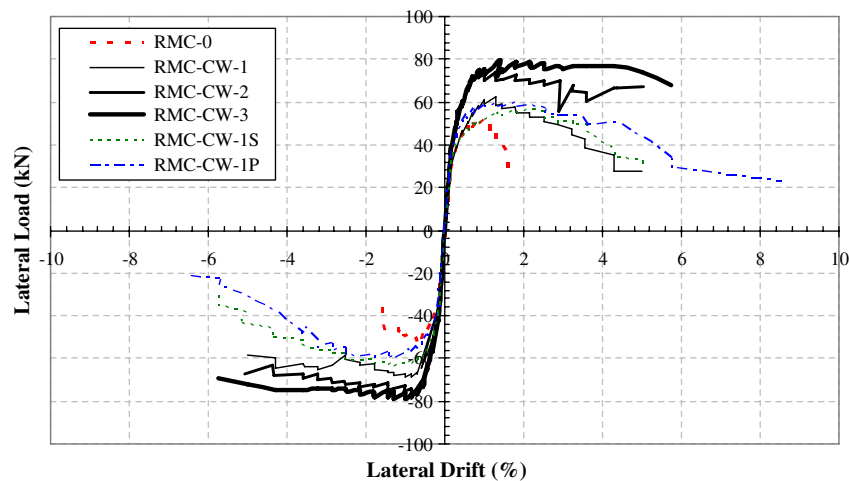


Fig. 9. Envelope of hysteretic loops of tested columns

Table 5. Displacement Ductility Factors for Tested Specimens

Specimen	Push			Pull		
	Δ_{yield} (mm)	Δ_{ult} (mm)	μ_{Δ}	Δ_{yield} (mm)	Δ_{ult} (mm)	μ_{Δ}
RMC-0	13	22	1.7	10	21	2.1
RMC-CW-1	14	44	3.1	15	62	4.1
RMC-CW-2	12	80	6.7	10	72	7.2
RMC-CW-3	15	102	6.8	13.6	100	7.5
RMC-CW-1S	13.1	50	3.8	21	52	2.5
RMC-CW-1P	13	49	3.8	20	50	2.5

that the control column experienced, the displacement ductility capacities of the retrofitted columns showed significant increase; the columns with two and three layers of CFRP wrap exhibited displacement ductilities approximately 3.5 times higher than that of the control column. As shown in Fig. 5 and Table 5, the confinement from CFRP did not significantly affect the load at yield of the rehabilitated columns, and all columns' reinforcement yielded at approximately the same level of lateral drift. Similar results were also observed by Binici (2008).

In addition to the percentage gain of lateral-load carrying capacity of all tested columns versus the volumetric ratio of FRP material, ρ_{FRP} , the percentage gain of displacement ductility shown in Fig. 8 can also be useful for design purposes.

Lateral Stiffness

Stiffness values were determined by dividing the maximum lateral load reached within a cycle by the corresponding displacement in the positive (push) or negative (pull) direction (see Fig. 10). The stiffness of a cycle is then idealized by taking the average of stiffness values in positive and negative directions (Park 1989).

The stiffness degradation diagram for the six tested columns is provided in Fig. 11. Retrofitted columns exhibited higher stiffness values than nonretrofitted ones. Retrofitted columns with two and three layers of CFRP sheets exhibited very close stiffness values, whereas the stiffness values of columns RMC-CW-1, RMC-CW-1S, and RMC-CW-1P are situated in between non-retrofitted and columns RMC-CW-2 and RMC-CW-3. At each level of lateral drift, lower stiffness represents the expansion of section cracks and damages resulting from applied lateral and axial loads. On the basis of the obtained stiffness degradation curves, the following general

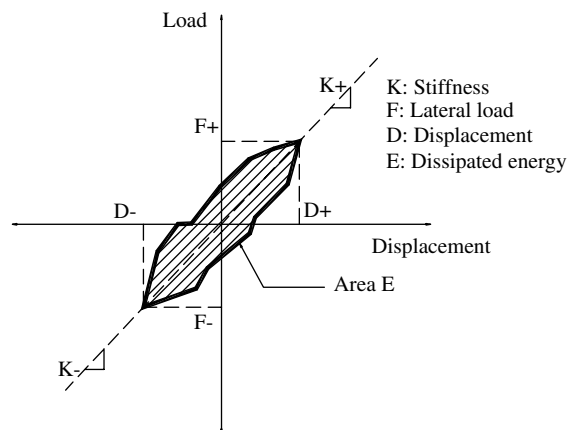


Fig. 10. Determination of stiffness and energy dissipation for each cycle

equation can be proposed for estimating the stiffness values of control and CFRP-strengthened square RCM columns:

$$k = (2.1\rho_{FRP} + 3.2)\delta^{(0.23\rho_{FRP} - 1.04)} \quad (3)$$

where k = column stiffness (kN/mm); δ = lateral drift (%); and ρ_{FRP} = volumetric ratio of CFRP reinforcement. To generate Eq. (3), first, separate equations were obtained for stiffness degradation versus lateral drift of all specimens, then the changes in the coefficients of each equation were linked to changes in the volumetric ratio of CFRP reinforcement (ρ_{FRP}) of the specimens. Although not shown in Fig. 11 for conciseness, the values of the proposed analytical equation for stiffness have very good correlation with the experimental results with a coefficient of determination value, R^2 , of 0.936. These results, combined with a strength degradation model, can be used for modeling the hysteretic rules of CFRP-strengthened rectangular RCM columns.

Energy Dissipation

The endurance of a RCM structure is significantly dependent on the energy dissipation capacity of its columns and walls. The energy dissipation in each cycle is estimated by calculating the area enclosed by the corresponding load-displacement hysteretic loop

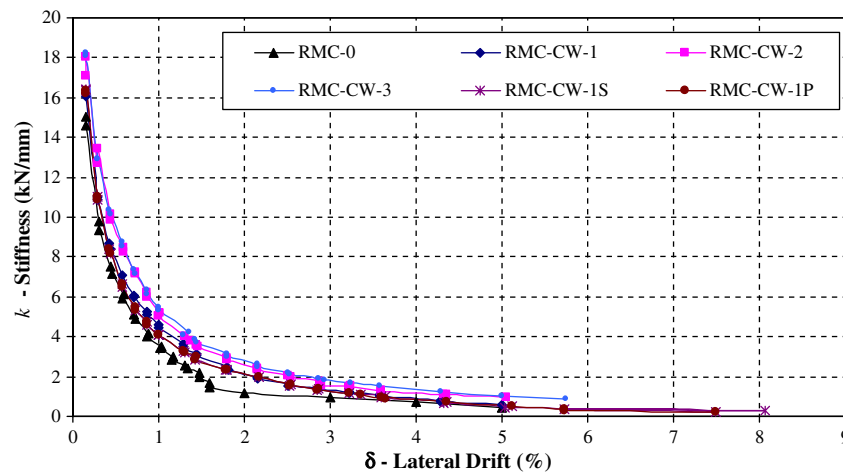


Fig. 11. Stiffness degradation of the six tested RMC columns

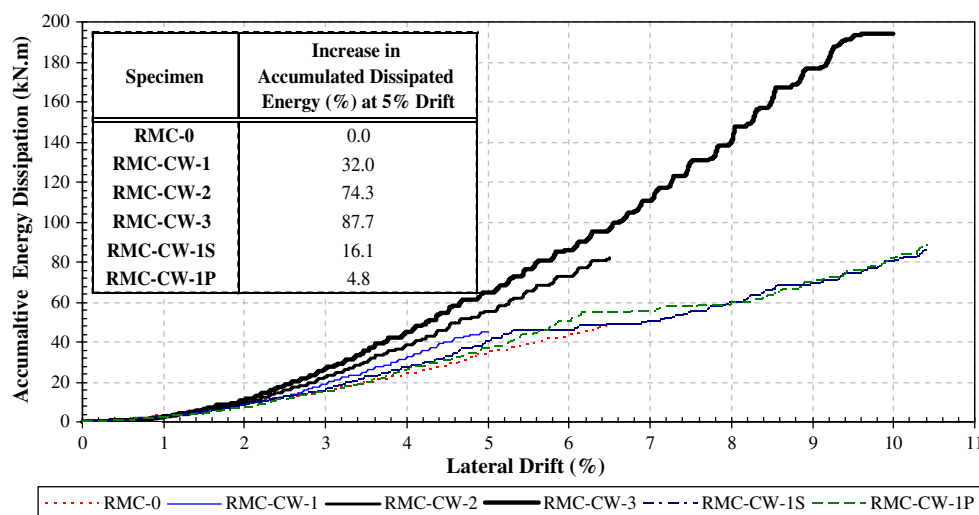


Fig. 12. Lateral drift versus accumulated dissipated energy for the six tested RMC columns

as shown in Fig. 10. The cumulative dissipated energy (CDE) was obtained by adding the energy dissipated in consecutive loops, and can be expressed as follows (Parvin et al. 2010):

$$CDE = \left[\sum_{j=1}^N \oint_{cyclej} F \cdot du \right] \quad (4)$$

where N = number of response cycles and F = lateral load obtained from Eq. (1) and corresponding to the top displacement u .

Fig. 12 shows the drift ratios versus cumulative dissipated energy. The figure also tabulates the increase in accumulated dissipated energy for all tested columns at a 5% lateral drift level. The retrofitted columns provided higher energy dissipation as compared with column RMC-0. The wobbles in the energy dissipation curves are caused by elastic energy release at the unloading paths of each cycle (two wobbles per cycle). Furthermore, whereas increasing the number of CFRP layers had a marginal effect in increasing the column's lateral-load carrying capacity, a significant increase in energy dissipation capacity is obtained. For instance, wrapping the test column with two layers of CFRP sheet provided an increase of

74% in energy-dissipation capacity as compared with the control specimen.

Conclusions

The effectiveness of CFRP confinement in increasing the lateral-load carrying capacity, displacement ductility, stiffness, and energy dissipation of RCM columns with square cross sections was assessed. This experimental research led to the following findings:

- Wrapping the reinforced masonry control column along its full height with one, two, or three layers of CFRP sheets increased the lateral-load bearing capacity of the column by approximately 30, 46, and 53%, respectively.
- Wrapping the reinforced masonry control column along its full height with one, two, or three layers of CFRP sheets increased the displacement ductility of the column by approximately 115, 260, and 273%, respectively.
- Wrapping the reinforced masonry control column along its full height with one or two layers of CFRP sheets increased the cumulative energy-dissipation capacity of the column at a 5% drift by approximately 32 and 74%, respectively.

- By wrapping the RCM columns at the plastic hinge zone, no cracks occurred above the wrapped zone when the columns were subjected to cyclic loads, and the failure occurred through CFRP rupture and the crushing of concrete masonry units, which took place at the column's base at very high lateral drifts (10% drift).
- The failure of columns wrapped at the plastic hinge only (RMC-CW-1P) or at the plastic hinge zone along with strips at the mortar bed joints (RMC-CW-1S) occurred because of premature rupture of the CFRP wrap. In these columns, CFRP failure occurred at a very high lateral drift of 10%.

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Notation

The following symbols are used in this paper:

- b and h = section dimensions;
 b_f = width of CFRP strips in partial wrapping;
 h_2 = distance between two hinges on the top of the column;
 N = number of response cycles;
 n = number of CFRP layers;
 P = applied vertical load;
 S_f = pitch in partial wrapping;
 t_f = thickness of CFRP sheet;
 u = top displacement of the column;
 V = applied lateral load;
 Δ = lateral displacement of the tip of the column;
 Δ_u = ultimate displacement;
 Δ_y = yield displacement;
 δ = lateral drift (%);
 μ_Δ = displacement ductility; and
 ρ_{FRP} = volumetric ratio of CFRP reinforcement.

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