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SEISMIC REHABILITATION OF REINFORCED MASONRY COLUMNS USING CFRP WRAPS

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ABSTRACT

Compared to reinforced concrete, relatively few experimental studies have been conducted to document the behaviour of masonry columns under combined axial load and flexure. Furthermore, by introducing new techniques such as using carbon fibre-reinforced polymers (CFRP) wraps, it is possible to enhance the behaviour of reinforced masonry columns considerably. Therefore, this paper focuses on improving the seismic performance of reinforced concrete masonry (RCM) columns using CFRP wraps. In current experimental study, five 1.4m reinforced masonry columns were constructed and tested when subjected to constant axial force and cyclic lateral excitations. The columns have a cross-section of 390mm×390mm and were constructed using bull-nosed concrete units. The first column, which had no CFRP wraps, was used as a control specimen while the other four columns were wrapped using different layers of CFRP sheets or different wrapping schemes. From the tests, it was observed that CFRP wraps improves confinement of masonry column, which leads to more ductile behaviour and improvement in lateral load capacity. In analytical part of this paper, a load-displacement model was proposed and validated by comparing the analytical results with the response of tested confined RCM columns.

Introduction

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

For reinforced masonry columns that are part of the moment resisting system, recent research showed that confinement in such members especially near the potential plastic hinging regions is not enough (Youd et al. 2000). Since demolishing and reconstructing such deficient elements is

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not an option, retrofitting them to meet appropriate seismic ductility demands is a necessity.

For retrofitting and upgrading purposes, fibre-reinforced polymer (FRP) composite materials have become popular material in strengthening reinforced concrete elements in recent years. It offers attractive characteristics such as high strength and high stiffness–to-weight ratio, as well as light weight for ease of application with minimal interruption to occupants (Seible1997).

Most research efforts in retrofitting deficient masonry structural elements using FRP were directed to masonry walls, and less work has been conducted in the past on retrofitting reinforced masonry columns. On the other hand, there has been significant effort in evaluating the performance of FRP-rehabilitated plain and reinforced concrete (RC) columns (e.g. Shrive et al. 2003, and Galal et al. 2005). In general, previous researches showed that wrapping non-ductile RC columns with CFRP sheets is an effective form of increasing the column's ductility and, hence, seismic performance. In order to increase confinement, FRP wraps are laid perpendicular to the column axis. However, the wrap is not activated until the concrete is dilating substantially as it is failing (Shrive et al. 2003). Early research work on FRP-strengthening of RC columns concluded that FRP wrapping is more efficient in circular columns compared to rectangular ones due to stress concentration at the columns edges. Chamfering round corners for concrete columns have been recommended to avoid such problem, where for concrete masonry; bull-nosed units can be used for the corners of the columns (Masia and Shrive 2003). It is observed in previous researches that in wrapped masonry columns under only axial load, failure appeared to be initiated by complete crushing of the mortar joints, causing wrinkles in the wrap, followed by explosive disruption of the CFRP wrap. The material first rips vertically, then very rapidly circumferentially at failure of masonry columns.

In this experimental program, wrapped column were tested when subjected to constant axial load and increasing lateral excitations. It is noteworthy to say that, up to authors' knowledge, literature survey did not reveal similar experimental program on the behaviour of concrete masonry columns under combined axial and lateral loads.

Experimental Program

In this research, the tests were carried out in two phases: a) the auxiliary tests that are meant to provide the mechanical characteristics of the constituent materials and the masonry assemblage, and b) Five 390×390×1400mm reinforced masonry columns were constructed and tested when subjected to constant axial force and cyclic lateral excitations. All the auxiliary specimens (masonry prisms) and columns were constructed by domestic professional masons representing the current method of practice in Québec.

Before starting the construction of RCM (reinforced concrete masonry) columns it was necessary to obtain material properties of concrete masonry blocks, mortar, grout, compressive and tensile strength of masonry assemblage, and CFRP sheets. CFRP material properties are summarized in Tables 1. The masonry unit that is used in this study is hollow concrete block with two bull-nose corners with nominal dimensions of 390mm×190mm×190mm. The average tested compressive strength of the unit is 15 MPa and the average net-to-gross area ratio is 0.7. Type S mortar with

28 days compressive strength of 20.7 MPa that is a mix of 0.5 volumetric unit Portland cement, one unit masonry cement, 2.9 units sand, and 0.7 unit water was chosen after several trial mixtures to be conforming with the requirements stated in ASTM C270-02 and CSA A179-04. The grout used in the program, categorized as "coarse grout" with 28 days compressive strength of 21.6 MPa and prepared in accordance with CSA A179-04 and ASTM C476-02. Coarse grout prepared by mixing one volumetric unit Portland cement, 2.8 units fine aggregate (sand), two units coarse aggregates with the maximum size of 7mm (1/4″), and 0.9 unit of water.

Table 1. Properties of the Tyfo SCH-11UP (as provided by supplier, Fyfo Co.2008)

Tensile strength (MPa)	**Young's modulus (MPa)**	**Elongation at break (%)**
903	86.9	1.05

In order to obtain compressive strength of masonry assemblage (f'_m) a series of five unreinforced grouted prisms were tested according to ASTM C1314-02a and as shown in Figure 1. It was decided to build five-block high and one-block wide prisms for a better representation of the real changes in the masonry columns. From tested prisms compressive strength of masonry assemblage equal to 11.5MPa was obtained.

Figure 1. Tests for evaluating (left) compressive and (right) tensile strength of masonry assemblage

For the purpose of estimating the tensile strength of masonry assemblage, a series of five prisms were tested under four-point loading method according to ASTM E518-02 guideline. As it is shown in Figure 1, Prisms with the height of seven blocks and width of one block were constructed in order to properly locate the two point loads and supports and also to provide sufficient span-to-depth ratio. From tested prisms tensile strength (f_t) equal to 1.3 MPa was obtained.

Table 2. Designation of tested specimens

Specimen	**Designation**
Reinforced **M**asonry **C**olumn : Control Specimen	RMC-0
Reinforced **M**asonry **C**olumn with **1** layer of **C**FRP **W**raps	RMC-CW-1
Reinforced **M**asonry **C**olumn with **2** layers of **C**FRP **W**raps	RMC-CW-2
Reinforced **M**asonry **C**olumn with **3** layers of **C**FRP **W**raps	RMC-CW-3
Reinforced **M**asonry **C**olumn with **1** layer of **C**FRP **S**trips	RMC-CW-1S

In the second phase of this experimental study, five 390×390×1400mm reinforced masonry columns were constructed on reinforced concrete footings which were securely fixed to a strong floor to simulate fixed support condition. Each column was reinforced with four 15M vertical steel rebars with yield strength of 450 MPa and 4.75mm steel ties with yield strength of 240 MPa on every row. The first column was tested as control specimen without any CFRP wraps and the rest of columns wrapped using different layers of CFRP sheets or different wrapping schemes as it is shown in Figure 2. Table 2 shows the designation of the tested columns. The tested columns were subjected to a constant axial force of 200kN ($\sigma = 0.11 f'_m A_g$) and cyclic lateral excitations. Cyclic load applied in the deformation control mode and cycles were repeated twice at each displacement level. Figure 3 illustrates the test set-up.

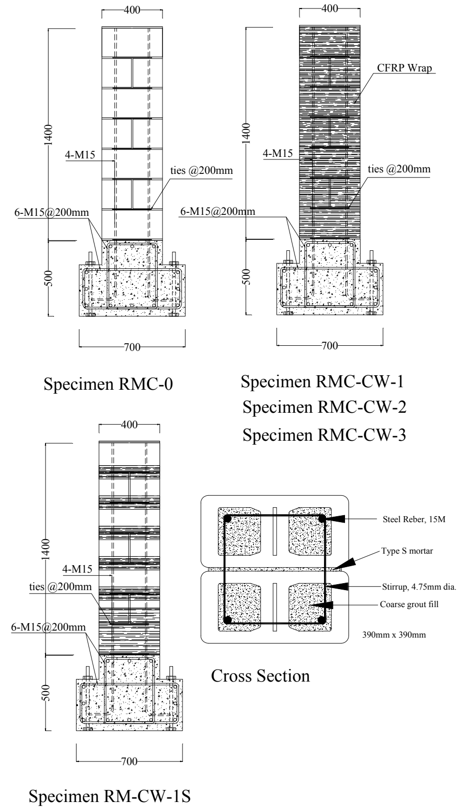


Figure 2. Dimensions and details of reinforcement of the five tested RCM columns

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

Observed Behaviour

Figure 4 shows experimentally recorded adjusted lateral load (F_{adj}) - lateral drift hysteretic relations of tested columns. Lateral loads are adjusted due to contribution of the horizontal component of vertical load to the applied lateral load (especially at high lateral displacements). Prior to the application of lateral load, columns were first loaded with constant axial load of 200 kN. In testing of control column (RMC-0), observations during the test showed that the first crack was formed at 0.7% drift. Cracks were widened during the subsequent cycles at the same displacement. The yielding of vertical rebars

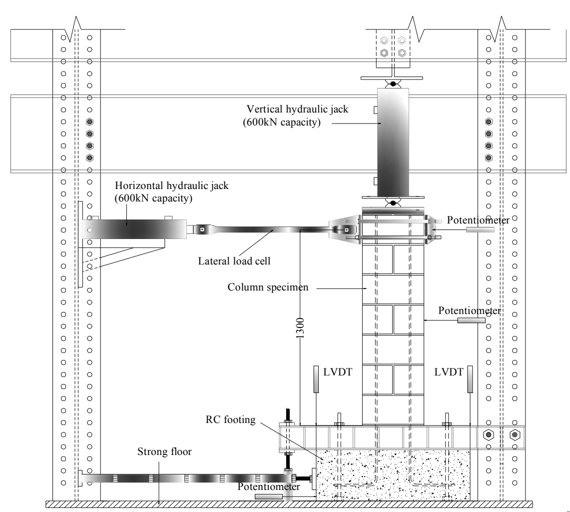


Figure 3. Test setup

started after 1% lateral drift. The new diagonal cracks caused decrease in the load bearing capacity. The maximum lateral load reached during the test was 52 kN at the 1% drift level (see Figure 4). Column RMC-0 was considered failed when lateral load capacity reduced to less than 80% of maximum lateral load. Therefore, failure occurred at 1.5% drift when lateral load capacity decreased to less than 40 kN. In order to achieve ultimate failure of the column, test was continued, between 1.0% and 5% drift levels new cracks appeared and continued to widen (see Figure 5a).and spalling of concrete blocks was observed at the base of the column at 4.0% drift level (see Figure 5b). After completion of 1.6% lateral drift level, column was only pushed until the end of the test at 5% drift level.

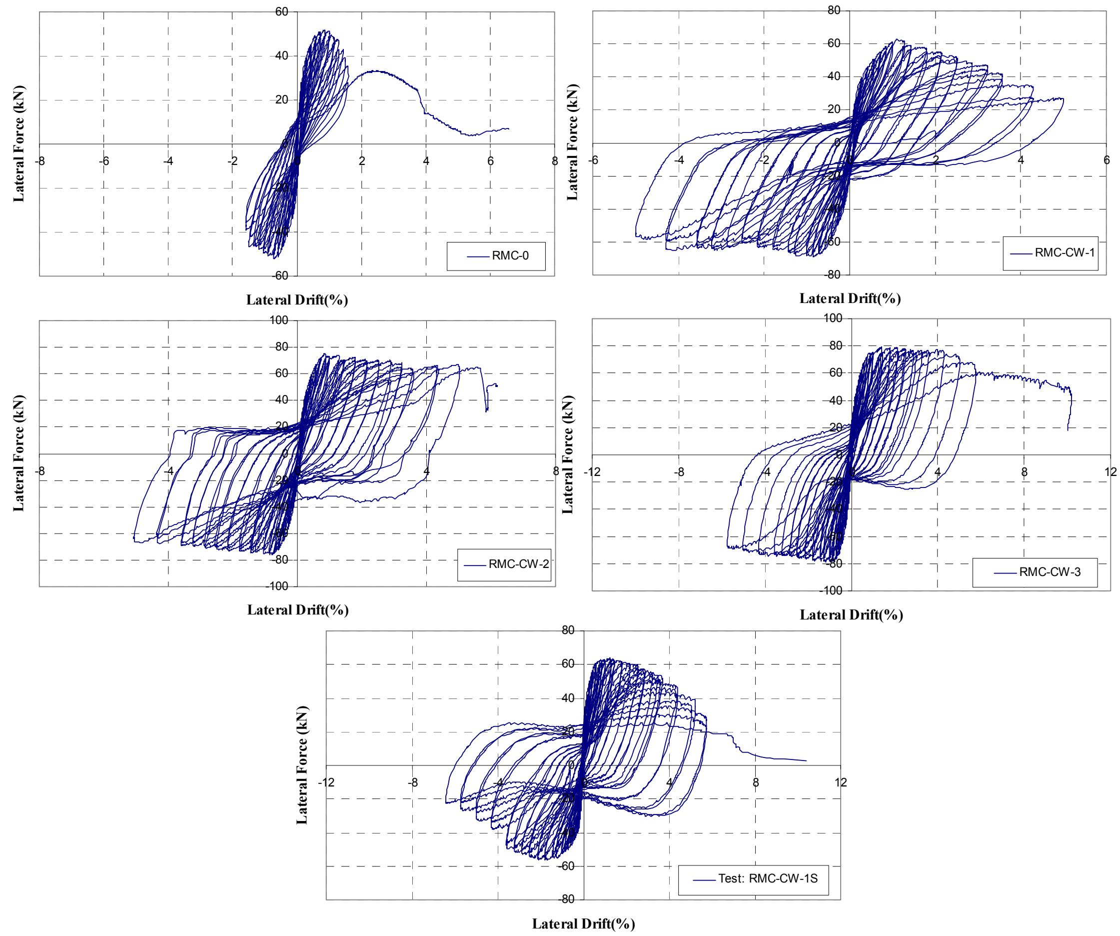


Figure 4. Lateral loads-lateral drift relationships of tested columns

In column RMC-CW-1, due to the confinement provided by one layer of CFRP wrap it was not possible to monitor cracking of the original masonry column; however, in column and footing interface minor rupture of CFRP wrap was observed (see Figure 5d). The lateral load did not increase after the drift level of 1.5% and reduced gradually after. The maximum lateral load reached during the test was 68 kN at the 1% drift level (see Figure 4). Column considered failed at 2.8% drift level when lateral load reduced to less than 80% of the maximum recorded lateral load. Test was stopped at 5% drift level, due to technical problems.

Column RMC-CW-2, lateral load did not increase after the drift of 1% where it reduced gradually. The maximum lateral load reached during the test was 76 kN at the 0.9% drift level (see Figure 4). Due to limitation of stroke of the horizontal actuator used in the test, it wasn't possible to continue the lateral drift beyond 6% drift, and up to this point no rupture or damage were occurred on CFRP wraps; however, local debonding of the wrap were observed. Note that after 5% drifts level, the specimen was only pushed and test was stopped at 6% drift level.

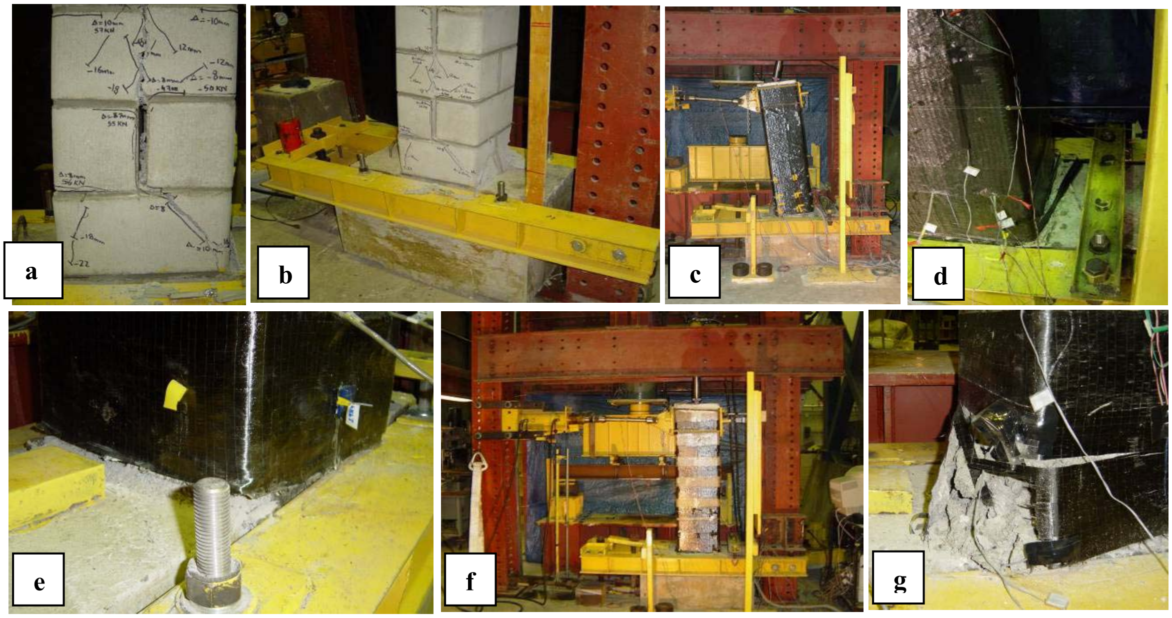


Figure 5. a) Crack development in column RMC-0, b) Spalling of concrete masonry unit at base of column RMC-0, c) Column RMC-CW-1 at 5% lateral drift, d) Minor rupture of CFRP wrap at the base of column RMC-CW-1, e) Column RMC-CW-2 and footing interface at 5% lateral drift , f) Column RMC-CW-1S before starting the test, g) CFRP wrap rupture and explosive crushing of concrete masonry at 10% lateral drift in column RMC-CW-1S

In column RMC-CW-3, lateral load did not increase after drift of 1.5% and started decreasing gradually after 4% drift (see Figure 4). As it is shown in Figure 4, the maximum lateral load reached during the test was 79.7 kN at 1.5% drift level. Test continued until 10% lateral drift and up to this point no rupture or damage were occurred on CFRP wraps; however, local debonding of wrap were observed. Note that after 5% drifts level, the specimen was only pushed until the end of the test. In columns RMC-CW-1, 2, and 3, it is observed after removing CFRP sheets that the crushed pieces of masonry can be easily removed from the bottom 200 mm of the column.

In the last test, it is tried to optimize used CFRP material; therefore, instead of wrapping masonry column continuously along the height, only plastic hinge zone (bottom 300 mm) and mortar joints were wrapped with one layer of CFRP strips, as it is shown in Figure 5f. Lateral load did not increase after drift level of 1.2% and reduced gradually after (see Figure 4). The maximum lateral load reached during the test was 63 kN which is 8% less than maximum recorded lateral load for column RMC-CW-1. Column RMC-CW-1S was considered failed at 3.5% drift when lateral load reduced to less then 80% of the maximum recorded lateral load. After 5% drift, column was only pushed, and ultimate failure was occurred at 10% drift with sudden rupture of CFRP wrap and explosive crushing of concrete masonry (see Figure 5g).

Improvement in lateral load bearing capacity and displacement ductility

Lateral load bearing capacity: As it is shown in Figure 6, maximum lateral loads are illustrated versus volumetric ratio of CFRP reinforcement. In general, by increasing the number of layers, lateral load bearing capacity of the columns increased. It is observed that by increasing CFRP layers from 1 to 2 maximum measured lateral loads increased 10.5%. The optimum number of CFRP layers depends on the section dimension and radius of rounded corners (Aiello et al., 2007), therefore; as it shown in Figure 6, for the tested columns in this study, more than 3 layers of CFRP sheets has almost no additional effect in increasing lateral load bearing capacity.

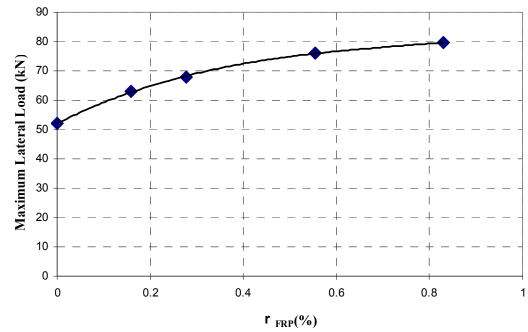


Figure 6. Maximum recorded lateral load versus volumetric ratio of CFRP reinforcement

Table 3. Displacement ductility factors for tested specimens

	Push			Pull		
Specimen	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

Displacement ductility: Displacement ductility factor (μ_Δ) can be written as Δ_y / Δ_u where Δ_y is the yield displacement and Δ_u is the ultimate displacement. In order to determine yield displacement, an equivalent elasto-plastic system was defined. The elastic branch of this system was secant to the real curve at 75% of the maximum lateral load and reached the maximum lateral load to find the yield displacement. The failure of the column was defined at the post-peak displacement, Δ_u, where the remaining capacity has dropped to 80% of the peak load (Park 1989). Table 3 shows the ductility factors for each column in both directions.

Analytical prediction of lateral load-lateral drift response of RCM columns

To predict lateral load-displacement of wrapped RCM columns subjected to axial and cyclic loads, first a moment-curvature (M-Φ) analysis was conducted, and then load-displacement responses of RCM columns were calculated based on M-Φ analysis and using the concept of plastic hinge as it is shown in Figure 7. For section analysis it is assumed that plane sections remain plane after deformations, tensile strength of masonry is neglected, and composite RCM section is analyzed assuming that the square section is divided to unconfined and confined zones with compressive strength. Unconfined and confined zones of masonry section are defined by four second-degree parabolas with an initial slope of the diagonals of the core with dimensions $b \times a$, as it is shown in Figure 8 (Lam and Teng, 2003). The methodology used for M-Φ and Lateral load-displacement analysis is further explained by Priestley et al. (2007).

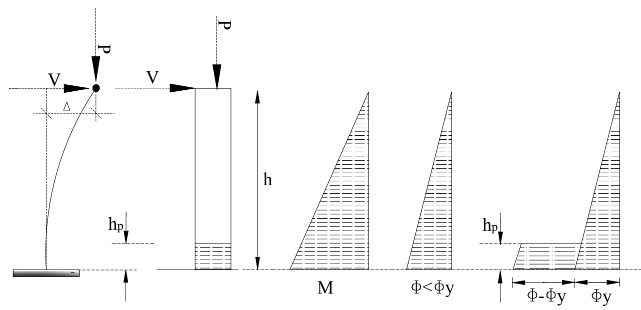


Figure 7. Obtaining displacement from idealization of curvature distribution before and after yield

Before starting the M-Φ analysis, it is important to define stress-strain models for unconfined and confined concrete masonry. Figure 9a represents stress-strain relation of unconfined concrete masonry obtained from Kent-Park model modified by Priestly and Elder (1983) (note that it is assumed that maximum compressive strength of masonry assemblage is equal to $f_m^{'} = 11.5MPa$, similar to tested masonry assemblage in this study). To obtain stress-strain relation of confined concrete masonry, a combination of lateral dilatancy based model introduced by Pantazopoulou and Mills (1996) and a confined strength model was used. Pantazopoulou and Mills model was originally developed by testing concrete cylinders, and in this study it was calibrated for concrete masonry by using modified Kent-Park model, as it is shown in Figure 9a. Figure 9b shows the stress-strain relation in confined zone of concrete masonry section. Different strength models were considered and finally strength model introduced by Lee (2006) was chosen:

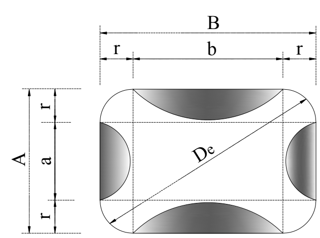
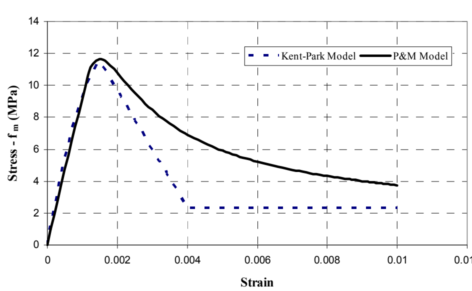


Figure 8. Model of effective confined zones

(a)



(b)

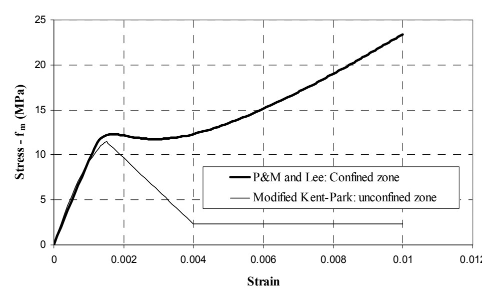


Figure 9. (a) Modified Kent-Park model and modified Pantazopoulou and Mills for unconfined concrete masonry (b)Stress-strain relation of confined and unconfined zones of a square concrete masonry section wrapped with one layer of CFRP wrap (similar to column RMC-CW-1)

$$\frac{f_{mc}}{f_m} = 1 + 3.75\left(\frac{f_l}{f_m}\right)^{0.83} \qquad \text{(Lee, 2006)} \qquad (1)$$

where f_m = compressive strength of unconfined concrete masonry, f_{mc} = compressive strength of confined concrete masonry, and f_l = confining pressure. As it is shown in Figure 10, a good agreement between experimental and proposed analytical model was observed.

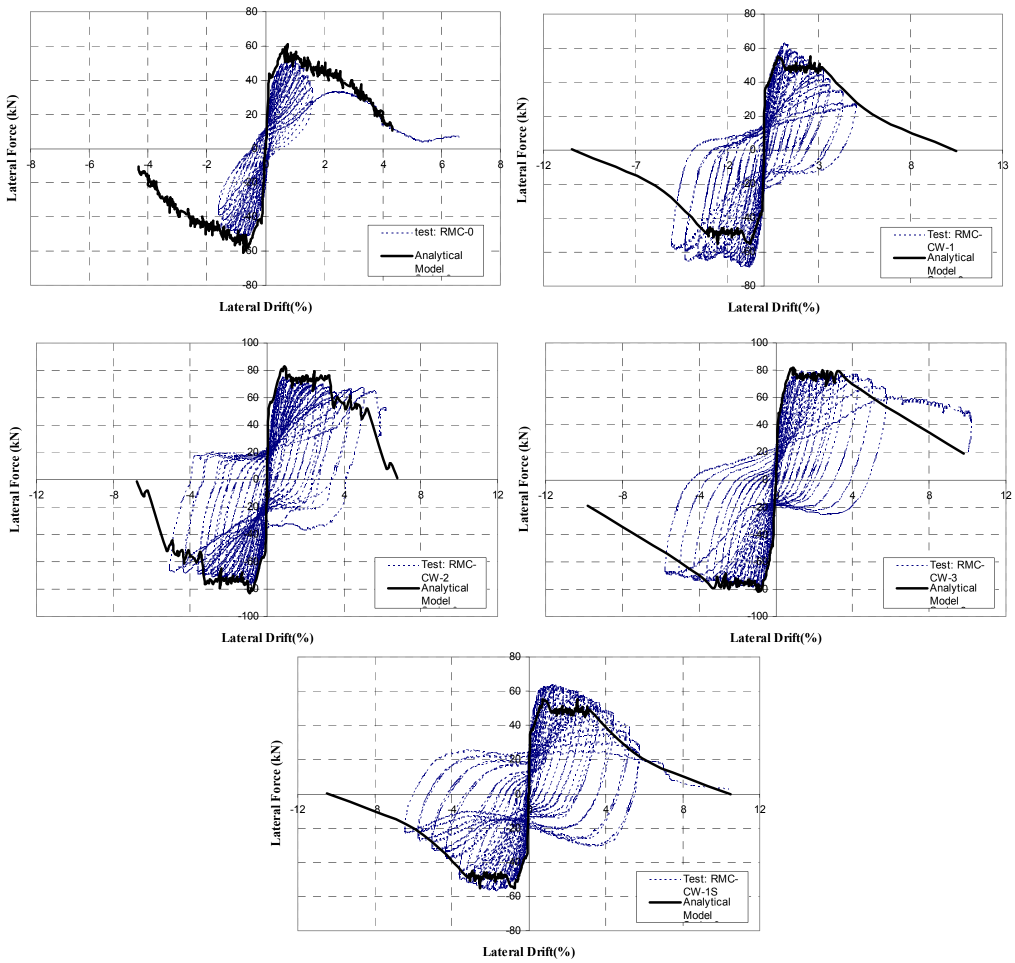


Figure 10. Load-displacement analysis for tested RCM columns

Conclusions

In this study, five full-scale reinforced masonry columns were tested under axial load and cyclic lateral loads. Columns were strengthened with different layers or patterns of CFRP sheets. This experimental research led to the following findings:

- Wrapping the reinforced masonry control column with full-height 1, 2 or 3 CFRP sheets, increased the lateral load bearing capacity of the column by about 30%, 46%, and 53% respectively.

- Wrapping the reinforced masonry control column with full-height 1, 2 or 3 CFRP sheets, increased the displacement ductility of the column by about 115% and 260%, and 273% respectively.

Furthermore in analytical part of this paper, moments-curvature and lateral load-displacement analysis of wrapped RCM columns subjected to axial load and cyclic lateral loads were conducted. In this analysis, strain-stress relations of concrete masonry are obtained from modified Kent-Park for unconfined zones, and combination of modified Pantazopoulou and Mills' model with strength model introduced by Lee (2006) for confined zone. In general, good agreement between introduced analytical model and obtained experimental results was observed.

Acknowledgments

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