

Axial compressive behavior of grouted concrete block masonry columns confined by CFRP jackets



Khalid Saqer Alotaibi¹, Khaled Galal*

Department of Building, Civil & Environmental Engineering, Concordia University, Montreal, Quebec, H3G 1M8, Canada

ARTICLE INFO

Article history:

Received 3 November 2016

Received in revised form

20 January 2017

Accepted 27 January 2017

Available online 31 January 2017

Keywords:

Carbon fibre

Strength

Mechanical properties

Mechanical testing

Masonry

ABSTRACT

Confining existing concrete and masonry columns by Carbon Fiber Reinforced Polymers (CFRP) is a beneficial method for enhancing the column axial capacity and ductility. This paper presents an experimental investigation of the CFRP confinement influence on the uniaxial compression stress-strain behavior of concrete block masonry columns. Scaled fully grouted concrete block masonry columns, with a square cross section, were confined by continuous CFRP jackets and tested under concentric axial loading up to failure. The results indicate that CFRP enhances the ultimate axial strain and the axial load capacity by up to 281% and 79%, respectively compared to unreinforced columns. In this study, the effect of corner radius and the thickness of CFRP jackets are investigated. Special attention was also given to the effective tensile strain in the CFRP jackets. Finally, the CNR-DT 200 R1 confinement model, the only guide addressing strengthening masonry columns with external FRP composites, was assessed and refined equation is proposed.

© 2017 Elsevier Ltd. All rights reserved.

1. Introduction

Rehabilitation or upgrading of existing structures is often more economical and preferable solution than demolishing the entire structures or reconstructing some members. Many existing masonry buildings are structurally deficient or in need of seismic upgrade. On the other hand, many concrete masonry columns are not capable of meeting the increase in axial loading demand in case of a change in building occupancy. In addition, factors such as poor construction practices, low quality materials, and environmental deterioration may cause strength degradation. Also, reinforced concrete masonry columns that are part of the lateral force resisting system or designed to sway under lateral seismic load may need more ductility for better performance or to meet new seismic design requirements. All of the above create a need to develop effective strengthening method for increasing the ductility and bearing capacity of concrete masonry columns.

Fiber Reinforced Polymers (FRP) materials have been commonly used in recent decades to confine concrete columns. The popularity of using FRP for strengthening and upgrading concrete columns is

mainly attributed to the high strength and lightweight characteristics of the FRP materials [1]. Using FRP composites reduces additional dead load associated with traditional strengthening solutions and simplify the application in areas with limited access. Numerous researchers have taken advantage of FRP characteristics and see the potential of using FRP materials to externally confining concrete columns by aligning the fibers along the direction of hoop stresses perpendicular to the vertical axis of the columns, as will be presented in the following paragraphs.

In the last few decades, experimental, numerical and analytical works have been conducted to better understand the behavior of FRP confined concrete columns. Ozbakkaloglu and Lim [2] assembled a database of circular FRP-confined concrete tests contains 3042 results from 253 studies published between 1991 and 2013. Ozbakkaloglu et al. [3] reviewed and assessed 88 stress-strain models developed to predict the stress-strain behavior of circular FRP-confined concrete columns. The extensive researches of FRP confined concrete columns have explored many different parameters in order to determine main factors affecting the behavior of confined sections including fibers type, the volumetric ratio of FRP or thickness of jacket, tensile strength and ultimate strain of the FRP, unconfined concrete strength, the shape and dimensions of column section, the aspect ratio and corner radius for prismatic sections, and the presence of steel reinforcement in confined columns. A significant effort has been invested in field of

* Corresponding author.

E-mail address: khaled.galal@concordia.ca (K. Galal).

¹ On leave Lecturer, Department of Building Engineering, College of Architecture and Planning, Imam Abdulrahman bin Faisal University, Dammam, Saudi Arabia.

strengthening reinforced concrete columns with externally bonded FRP. This has led to developing several design codes and guidelines [4–7].

Compared to the extensive research conducted in the field of confinement concrete columns with FRP, much less effort has been invested in retrofitting masonry columns. Few researchers have investigated the application of carbon, glass and basalt FRP to confine masonry columns constructed with clay, tuff, limestone, or concrete blocks.

In the experimental studies [8–11], different types of masonry column confined with different FRP jackets were tested. Confinement models calibrated by experimental results were proposed in Refs. [12–14]. Another technique has been considered by some researchers by internally confining masonry using FRP bars inserted in holes drilled through the column with or without FRP wrap [15–17]. Shaheen and Shrive [18] explored a different technique for strengthen masonry columns by spraying the columns with glass FRP in different thicknesses. Recent studies focused on discontinuous confinement by FRP strips [19–21]. From the above it can be seen that there is a need for more research that contributes to the literature on FRP strengthening of concrete block masonry columns. Galal et al. [22] tested reinforced concrete masonry columns strengthened with carbon FRP under combined axial load and cyclic quasi-static loading. Farnia [23] proposed a confinement model for FRP wrapped concrete masonry columns calibrated based on an experimental investigation of continuous and discontinuous carbon FRP wrapped grouted concrete masonry columns.

In the failure mechanism of confined masonry columns with FRP wraps, the passive confinement provided by FRP composites is induced by increasing the lateral expansion of blocks and grout under axial force. The restraint of FRP composites to the expansion of core of column transforms into tension stress along the hoop direction of FRP composites. The FRP lateral confining pressure helps the masonry composite keep its integrity until the FRP composites reach the maximum tensile force, the system would fail due the rupture of FRP composites. Generally, confinement leads to axial strength increase and strain enhancement. Several experimental research [24] observed that FRP confinement is more effective in strengthening circular columns compared to other shapes. In the case of prismatic columns with a continuous FRP jacket, only the area contained by four second-degree parabolas is considered effectively confined. Rounding edges of the columns or increasing the corner radius plays a significant role in increasing the effectively confined area which leads to the increase of the compressive strength of the columns. The arch-effect for square cross section is shown in Fig. 1. Also it has been noticed that confined plain concrete masonry prisms show similar behavior

under compression to the behavior of confined prisms with vertical steel bars. Priestley and Elder [25] indicated that presence of vertical steel bars in the columns does not significantly influence concrete masonry stress-strain curves when the contribution of vertical steel bars was subtracted from the total axial stress of masonry columns.

The results of 19 half-scale unreinforced concrete block masonry prisms confined with Carbon Fiber Reinforced Polymers (CFRP) jackets are presented in this study. The main objective of this paper is to quantify the compressive strength and the ultimate axial strain capacity of concrete masonry columns under various CFRP confinement levels with different corner radius towards the development of the complete stress-strain curves. The thickness of CFRP jacket and the corner radius of section have been mainly chosen for their influence in changing the shape of stress-strain curve. Strain gages mounted along the faces of the prisms were used to obtain a precise measurement of the effective tensile strain in the CFRP jackets. Finally, the analytical confinement model recommended by CNR-DT 200 R1 [26], the only guide addressing strengthening masonry columns with external FRP composites, was assessed and refined.

2. Experimental tests

2.1. Prism construction and preparation

A total of 19 half-scale fully grouted concrete masonry prisms were constructed and tested under axial compression. All prisms were two blocks thick and five blocks high, bonded together with 5 mm mortar. Professional masons built prisms by placing blocks in running bond pattern. Prisms were grouted after a week of curing in laboratory environment. All half-scale masonry prisms had a cross section of 185×185 mm and height of 470 mm before rounding the corners, as shown in Fig. 2. The primary variables in experimental tests were the number of composite layers and the corner radius of the cross section. The properties of the tested prisms are shown in Table 1. Seven sets of prisms consisted of three or two replicate prisms each, were tested in order to obtain average data. The sets varied with a corner radius of 0, 10 and 30 mm. Prisms were confined with one, two and three layers of CFRP composite jackets. Each prism is given a notation as RX-LN-#, where R stands for a corner radius, X is the corner radius, L stands for a composite layer, N is the number of composite layers, and the final number represents the number of replicate prism in the set. The prisms without wrapping and no rounded corners were used as control specimens and were denoted by R0-L0.

The surface of all prisms was prepared before wrapping CFRP sheets. The flanged ends of the concrete block units were filled with a repair mortar, as shown in Fig. 3. The recommended repair mortar according to the manufacturer's specifications is Sikadur® 30 epoxy extended with dried silica sand at a 1:1 volume mix ratio. Before jacketing the prisms, the corners were rounded with a radius of 10 or 30 mm by using an electric grinder. After patching the repair mortar, a grinder was used to sand and remove any irregularity or protrusion that comes from the bed joint or the repair mortar. After that, the dust and loose particles coming from grinding process were removed by a vacuum, and then the surface was cleaned by water. The prepared masonry prisms before wrapping CFRP sheets are shown in Fig. 4.

The steps taken to wrap the masonry prisms are summarized in Fig. 5. In order to ensure the effectiveness of the wrapping, the overlapping of the sheet in the direction of the fibers was 150 and 125 mm for prisms with corner radius of 10 and 30 mm, respectively. The overlapping is more than the recommended length of 100 mm by the manufacturer's specifications, as shown in Fig. 6.

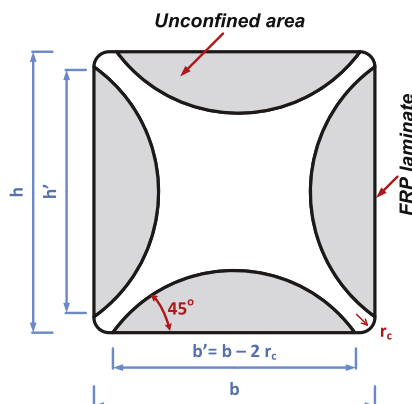


Fig. 1. The arch-effect for square cross section.

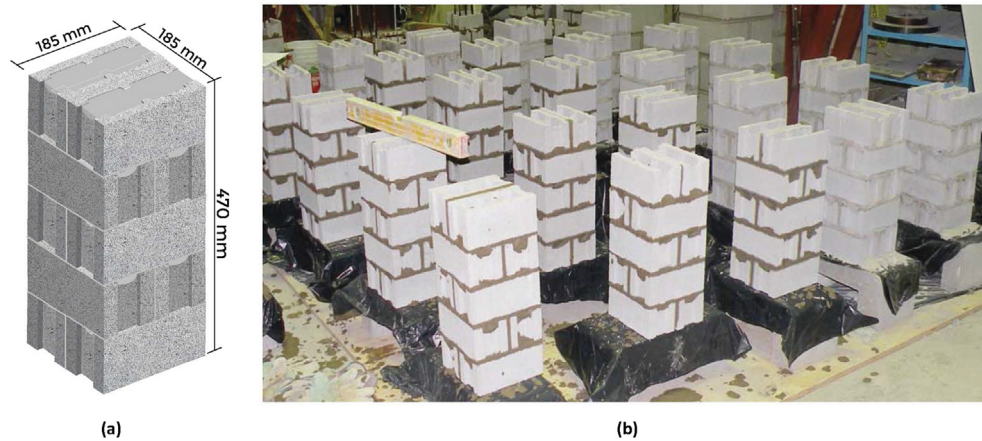


Fig. 2. (a) Dimensions of half-scale fully grouted prism; (b) prisms construction before grouting.

Table 1

Properties of the prisms.

Set	Corner radius (mm)	Number of CFRP layers	Number of specimens	Cross-sectional area (mm ²)
R0-L0	—	—	3	34225
R10-L1	10	1	3	34139
R10-L2	10	2	3	
R10-L3	10	3	2	
R30-L1	30	1	3	33452
R30-L2	30	2	3	
R30-L3	30	3	2	

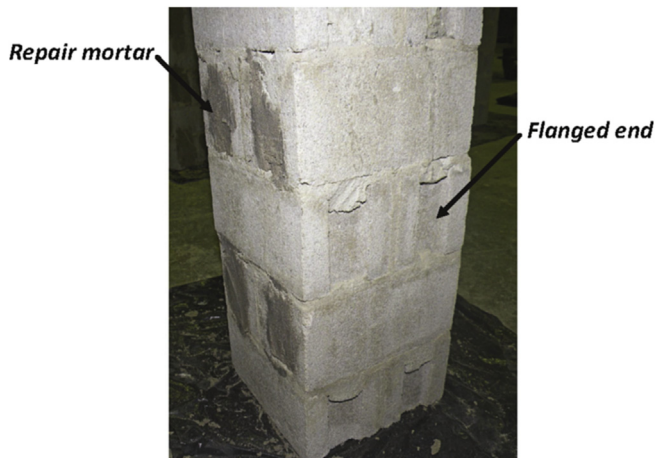


Fig. 3. Filling the flanged ends of the concrete block units with a repair mortar.

The CFRP sheet was wrapped around the prisms by the dry lay-up procedure approximately six months after the prisms were constructed. First, the fiber sheet was cut to the desired length. Then, Sikadur[®] 330 epoxy resin was combined by adding contents of component A to component B and mixed thoroughly for 3 min with a mixing drill on a low speed until resin and hardener were uniform in color. The mixed epoxy resin was applied directly onto the prepared prisms using a paint roller. The fiber sheet was carefully placed onto the resin coating and smoothed out. The resin was squeezed out between the fibers and any irregularities or air pockets were worked out with a plastic laminating roller. If more than one layer of fiber sheets was needed, additional epoxy resin

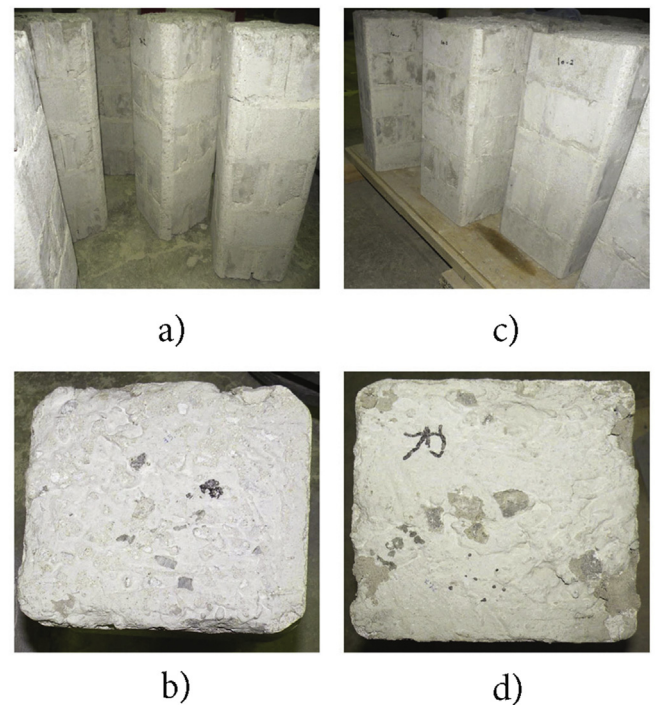


Fig. 4. (a) Prisms with corner radius of 30 mm; (b) cross section of prism rounded with 30 mm; (c) prisms with corner radius of 10 mm; and (d) cross section of prism rounded with 10 mm.

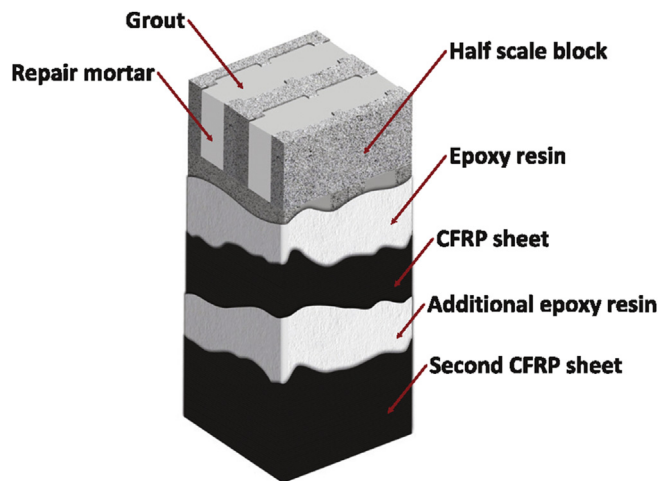


Fig. 5. Steps taken to wrap prisms.

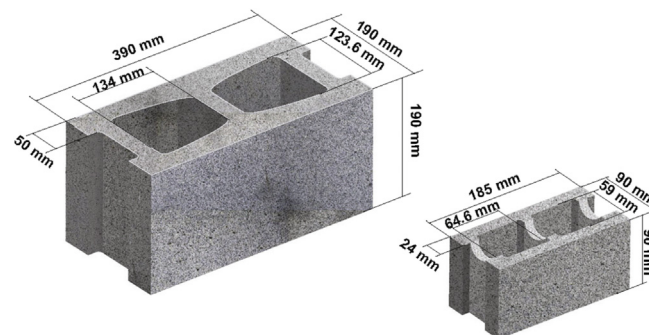


Fig. 7. Typical dimensions of the half-scale block comparing to the full-scale block.

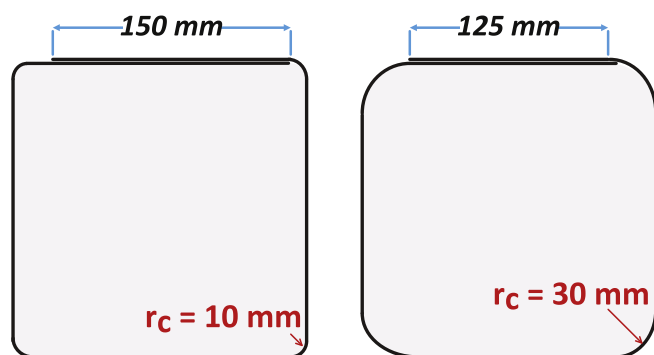


Fig. 6. The overlapping for prisms with corner radius of 10 and 30 mm.

was applied on the previous layer without stopping the lay-up process. The overlapping area is always placed at one face of the prism. The wrapped prisms were cured in the laboratory environment for at least one week before testing.

2.2. The mechanical properties of materials

The half-scale concrete blocks used in prisms construction were similar to the blocks proposed by Long [27]. The model units resemble the hollow stretcher type of 200 mm nominal size concrete masonry blocks. Typical dimensions of the half-scale block compared to the full-scale block are shown in Fig. 7. Three half-scale masonry units were tested according to ASTM C140 [28] to determine the compressive strength of block. After cutting the top part of the block to remove the depressed webs, high strength gypsum capping compound was used to distribute the load uniformly to the blocks. The average compressive strength for half-scale concrete blocks was 22.88 MPa with coefficient of variation

(COV) equals to 4.4% considering average net area equals to 8424 mm² (COV = 1.5%). The average density of half-scale blocks was 2181 kg/m³ (COV = 0.5%) and absorption was 108.5 kg/m³ (COV = 2.1%). The moisture content of blocks averaged around 7.9% (COV = 4.6%). A coarse grout, according to ASTM C476 [29], was designed for grouting the prisms. The grout was cast in 100 (diameter) × 200 (height) mm plastic molds. The cylinder specimens of grout were cured for 28 days and tested after six months from construction in the same period of testing prisms. The average compressive strength of 3 cylinders of grout was 31.39 MPa (COV = 8.5%). Ready-to-use mortar mix, blend of sand and cements, commercially available used throughout the construction process in order to achieve more consistency. The mortar meets Type S requirements as specified in ASTM C270 [30]. Compression tests were conducted according to ASTM C780 [31] on 51 mm mortar cubes resulted in average 13.12 MPa (COV = 5.0%).

The unidirectional carbon fiber sheet SikaWrap® Hex 230C with Sikadur® 330 epoxy resin [32] were used for wrapping the concrete block masonry prisms. The mechanical properties of the CFRP laminate were obtained experimentally from tensile testing of flat coupons and compared to the manufacturer's datasheet in Table 2. Five coupons of CFRP from the same roll used for wrapping the prisms were fabricated to perform tension tests according to the ASTM D3039 [33] and ASTM D7565 [34] standards. Large laminates were prepared with two layers by the regular dry layup process and then allowed to cure in the laboratory environment for at least one week. The cured laminates were cut to 15 mm nominal wide strips by a diamond saw. The nominal thickness of the carbon fiber cured laminate is 0.381 mm. The flat coupons were fabricated with thick GFRP tabs at the ends to avoid premature failure caused by the jaws of the testing machine. Two strain gages with 5 mm gauge length were attached to each coupon to measure the ultimate tensile strain of the laminate. The test was conducted at a constant cross-head movement rate of 2 mm/min. The ultimate tensile strength and the modulus of elasticity were calculated according to the ASTM D3039 [33] based on the actual composite laminate thickness and actual width rather than the nominal dimensions of cured laminate. The nominal dimensions of flat coupon specimen are shown in Fig. 8.

Table 2
Comparison of CFRP coupon test and manufacturer's datasheet.

Mechanical properties of CFRP	Manufacturer	Average coupon test (COV)	Coupon test/Manufacturer
Tensile strength (MPa)	894	984 (3.4%)	1.10
Elastic modulus (GPa)	65.40	69.36 (8.3%)	1.06
Ultimate tensile strain (%)	1.33	1.34 (2.2%)	1.01

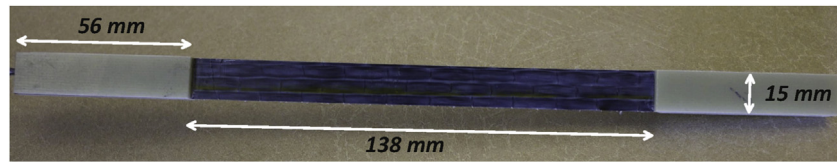


Fig. 8. Nominal dimensions of flat coupon specimen.

2.3. Test procedure and instrumentation

All prisms were instrumented to capture the complete stress-strain behavior. A hydraulic loading cylinder connected to a manually controlled motor pump was used to apply the axial compression force on tested prisms. The applied compression load was measured by a load cell attached to the loading cylinder and recorded by a data acquisition system with high recording speed to be able to capture the descending branch of stress-strain curves. Before testing, the prism was placed inside the testing frame and both ends of prism were capped with a layer of high strength gypsum to level the steel bearing plates. A steel spherical head was placed above the upper bearing plate and centered with respect to the center of the prism to maintain uniform distribution of stress and avoid any eccentricity. Four draw wire displacement sensors, with 0.05% linearity, were attached between the lower and upper bearing plates at the centers of the four sides to measure the vertical displacement of tested prism. A total of 16 strain gages with 5 mm gauge length, four on each side of the prism, were installed

on the outer layer of CFRP composite jacket to measure the tensile hoop strains within the fiber laminates. The strain gages were placed near the rounded corners at mid height of the second and the fourth courses of the prism. There were no strain gages on unwrapped control specimens. Test setup and instrumentation are shown in Fig. 9. A monotonic uniaxial compressive load was applied up to prism failure by rupture of the entire CFRP jacket. The test was conducted at an approximate rate of 15 kN/min.

3. Experimental results and discussions

3.1. Failure modes

The three unwrapped control specimens showed vertical cracks at maximum load followed by separation between the faces of the masonry blocks, leading to spalling of the face shells and end webs. Fig. 10 shows failure modes of unwrapped prisms at the end of the compression tests. The failure of CFRP wrapped prisms was always by tensile rupture of the CFRP jacket. The damage of CFRP confined

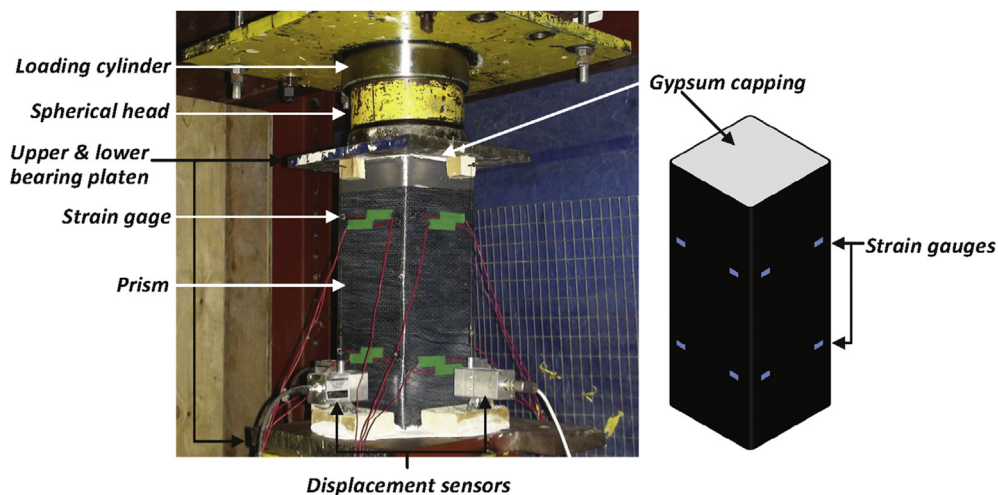


Fig. 9. Typical test setup and instrumentation for wrapped prisms.

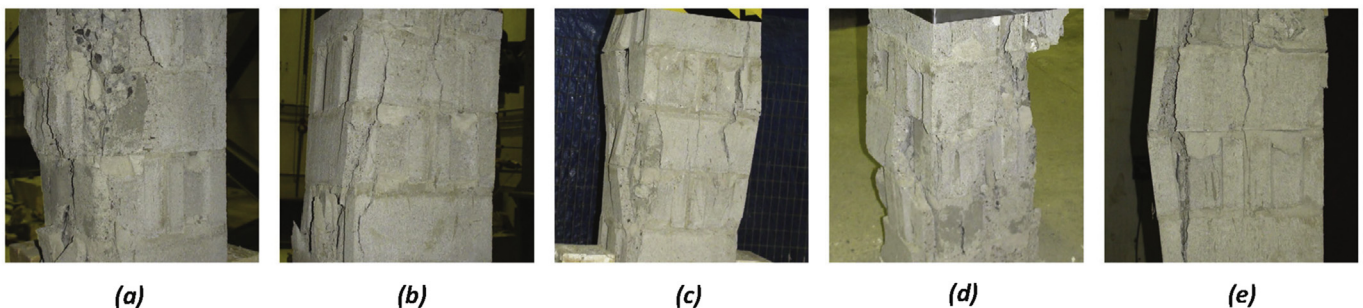


Fig. 10. Failure modes of unwrapped prisms at the end of the compression tests: (a) R0-L0-1; (b), (c) R0-L0-2; (d), (e) R0-L0-3.

prisms at the end of the tests can be seen in Fig. 11. Axial stress-strain curves in Fig. 12 show that when the load reached the peak and the descending branch of stress-strain curves started, the failure was initiated by local CFRP snapping near the prism corners due to local stress concentration in the jacket. The local rupture of CFRP straps subsequently decreased the load. The prism continued carrying load until a sudden rupture of the entire CFRP jacket with a loud sound followed by an abrupt load drop. The failure of composite jackets occurred near the corners in the medium portion of the confined prisms. After the test, a close visual examination of the prism revealed a strong bond between the CFRP jacket and the external masonry substrate (Fig. 11g). The separation was mainly between the face shell of block and the grouted core or was in the mid of the face shell. No debonding between CFRP layers at the overlapping zone was noticed. Mainly this premature failure was avoided by providing the sufficient overlap length. Also, it was observed that the cores of CFRP wrapped prisms were severely crushed more than unconfined concrete masonry prisms (Fig. 11h).

3.2. Axial stress-strain curve

The axial stress-strain curves of all test prisms are shown in Fig. 12. The axial stress is calculated as the force measured by the load cell divided by the area of the cross section considering the respective corner radius of the prism. The axial strain is the average reading of the four displacement sensors divided by gauge length equal to the prism height. The experimental results from axial compression tests with the average and (COV) are summarized in Table 3 and Table 4, where f_{md} is unconfined masonry strength; ϵ_m and ϵ_{mu} are axial strains at peak and ultimate axial strain of unconfined masonry corresponding to $0.85 f_{md}$ along the descending branch of the stress-strain curve, respectively. Also f_{mcd} is peak strength of confined masonry. ϵ_{mc} and ϵ_{mcd} are axial strains at peak

and the axial strain at 15% strength degradation for confined masonry, respectively. The definition of the usable ultimate axial strain of confined or unconfined masonry at 15% strength degradation is arbitrary, although being consistent with ACI 440.2R guideline [4].

All tested prisms exhibited a parabolic ascending portion of stress-strain curves followed by a descending branch, instead of featuring bi-linear stress-strain curves reported for FRP confined concrete prisms. However, the average peak stresses and ultimate axial strains at 15% strength degradation in confined prisms are much higher than the corresponding average stress and ultimate axial strain in unconfined prisms. It can be concluded that confining by CFRP jackets is an effective method for strengthening concrete block masonry columns. The unreinforced prisms possess the steeper descending branch and the R30-L2 and R30-L3 sets showed almost a plateau region near the peak strength followed by a sudden drop. It is observed that the increase in CFRP jacket thickness makes the slope of the descending branch less steep.

The axial stress-strain curves for one prism from each set of tested prisms are shown in Fig. 13 for a convenient comparison between sets. The unconfined control specimens exhibited a brittle behavior with a steep drop in strength after reaching the peak load. It was observed from stress-strain curves for the one-layer specimens that all six prisms exhibited enhancement in the post peak behavior by softening the descending branches of the stress-strain relationships compared to unreinforced prisms. For the two-layer prisms, the descending branches of the stress-strain curves showed more ductile behavior than the one-layer sets. Also, the slope of the descending branches of the stress-strain curve of R30-L2 set is flatter than R10-L2 set which is related to the corner radius increasing. For the three-layer prisms, the higher confinement increased the ductility of prisms. The R30-L3 set almost demonstrates a plateau region near the peak strength. On the other hand, the R10-L3 showed more ductile behavior after a slight drop in strength.

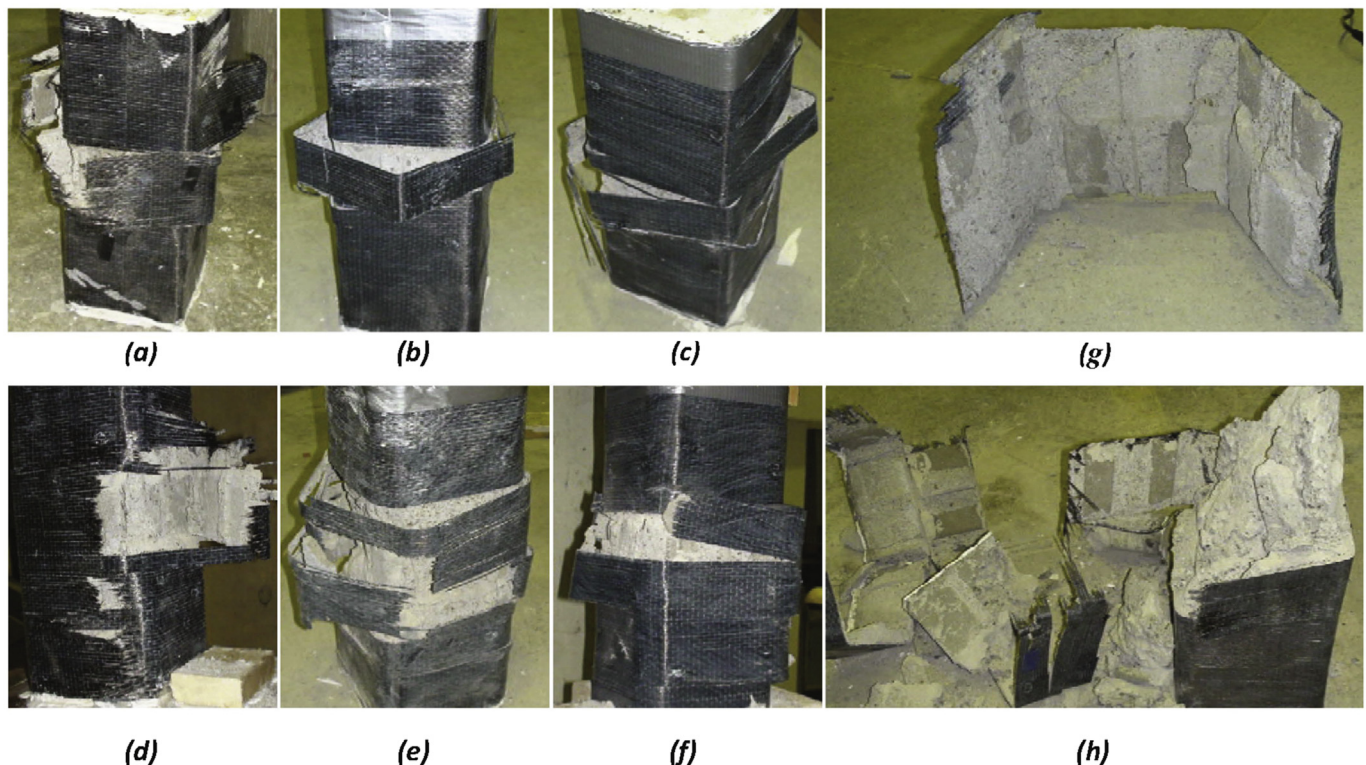


Fig. 11. Failure modes of CFRP confined prisms at the end of the compression tests: (a) R10-L1; (b) R10-L2; (c) R10-L3; (d) R30-L1; (e) R30-L2; (f) R30-L3; (g) strong bond between CFRP and masonry; (h) severely crushed core.

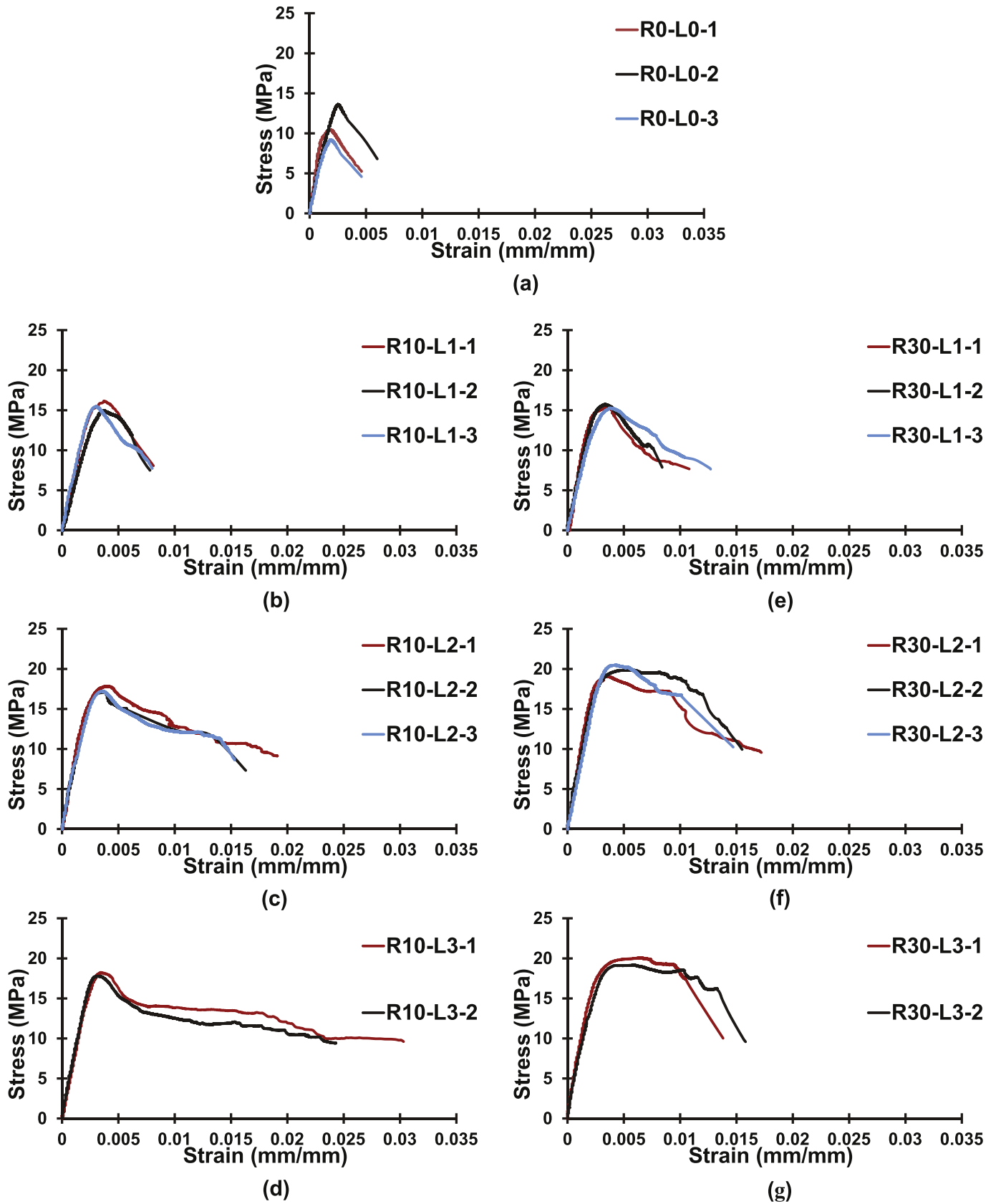


Fig. 12. Axial stress-strain curves of all test sets: (a) R0-L0; (b) R10-L1; (c) R10-L2; (d) R10-L3; (e) R30-L1; (f) R30-L2; (g) R30-L3.

Table 3

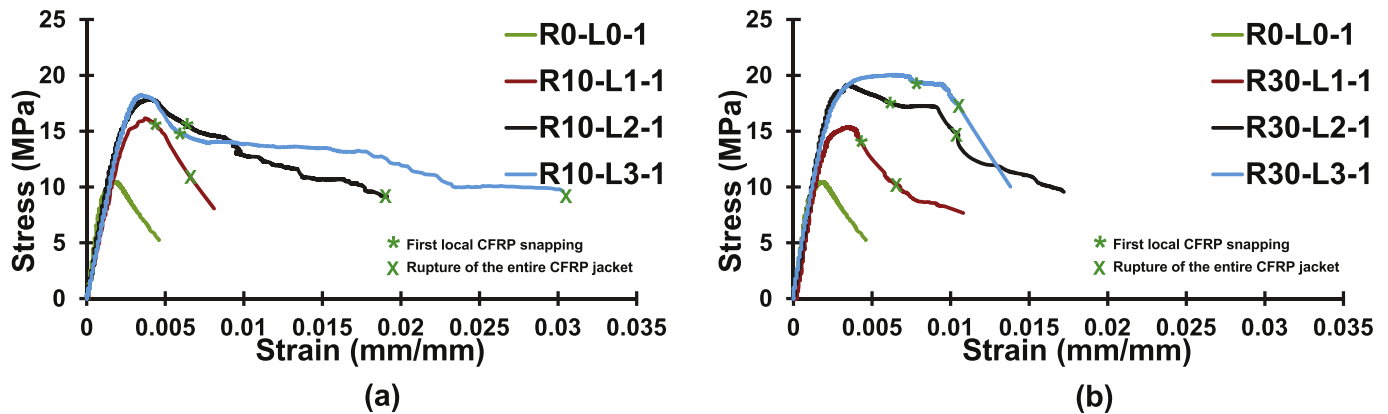
Compressive strengths and ultimate axial strains of the unconfined prisms.

Prism	f_{md} (MPa)	Average and COV (%)	ϵ_m (mm/mm)	Average and COV (%)	ϵ_{mu} (mm/mm)	Average and COV (%)
R0-L0-1	10.47	11.10	0.0018	0.0021	0.0027	0.0030
R0-L0-2	13.63	20.6	0.0026	20.8	0.0035	15.6
R0-L0-3	9.19		0.0019		0.0027	

Table 4

Compressive strengths and ultimate axial strains of the CFRP confined prisms.

Prism	f_{mcd} (MPa)	Average and COV (%)	ϵ_{mc} (mm/mm)	Average and COV (%)	ϵ_{mcu} (mm/mm)	Average and COV (%)
R10-L1-1	16.15	15.54	0.0038	0.0035	0.0054	0.0052
R10-L1-2	14.98	3.8	0.0037	12.5	0.0059	13.5
R10-L1-3	15.48		0.0030		0.0045	
R10-L2-1	17.85	17.39	0.0040	0.0038	0.0068	0.0063
R10-L2-2	17.05	2.4	0.0039	5.4	0.0063	7.9
R10-L2-3	17.25		0.0036		0.0058	
R10-L3-1	18.26	18.03	0.0035	0.0033	0.0054	0.0053
R10-L3-2	17.80	1.8	0.0031	8.6	0.0052	2.7
R30-L1-1	15.35	15.46	0.0034	0.0035	0.0048	0.0055
R30-L1-2	15.75	1.6	0.0033	7.6	0.0052	16.2
R30-L1-3	15.29		0.0038		0.0065	
R30-L2-1	19.13	19.84	0.0034	0.0043	0.0095	0.0098
R30-L2-2	19.90	3.4	0.0052	20.9	0.0118	19.6
R30-L2-3	20.49		0.0043		0.0080	
R30-L3-1	20.07	19.63	0.0063	0.0061	0.0107	0.0114
R30-L3-2	19.19	3.2	0.0058	5.8	0.0122	9.3

**Fig. 13.** Axial stress-strain curves for one specimen from each set: (a) prisms rounded with 10 mm; (b) prisms rounded with 30 mm.**Table 5**

Strength and strain gains of CFRP confined concrete prisms.

10 mm corner radius					30 mm corner radius					Average R30/ R10	
Prism	f_{mcd}/f_{md}	Average	$\epsilon_{mcd}/\epsilon_{mu}$	Average	Prism	f_{mcd}/f_{md}	Average	$\epsilon_{mcd}/\epsilon_{mu}$	Average	f_{mcd}	ϵ_{mcd}
R10-L1-1	1.46	1.40	1.80	1.74	R30-L1-1	1.38	1.39	1.60	1.82	0.99	1.05
R10-L1-2	1.35		1.95		R30-L1-2	1.42		1.72			
R10-L1-3	1.39		1.49		R30-L1-3	1.38		2.16			
R10-L2-1	1.61	1.57	2.28	2.10	R30-L2-1	1.72	1.79	3.18	3.26	1.14	1.55
R10-L2-2	1.54		2.10		R30-L2-2	1.79		3.94			
R10-L2-3	1.55		1.92		R30-L2-3	1.85		2.67			
R10-L3-1	1.64	1.62	1.80	1.77	R30-L3-1	1.81	1.77	3.56	3.81	1.09	2.15
R10-L3-2	1.60		1.74		R30-L3-2	1.73		4.07			

To assess the effect of confinement provided by CFRP jackets, strength and strain gains of confined masonry are presented in Table 5. The strength gain is defined as the peak stress of confined prisms divided by the average compressive strength of the unconfined control specimens and the strain gain is defined as the

ultimate axial strain of confined masonry divided by the average ultimate axial strain of unconfined masonry at 15% strength degradation.

The average strength gain for 10 mm rounded corner prisms with one, two, three layers were 40%, 57%, and 62% higher than the

strength of control specimens, respectively. On the other hand, the strength increased by 39%, 79%, and 77% for 30 mm rounded corner prisms with one, two, three layers, compared to control specimens, respectively. All prisms showed a significant increase in term of peak strength if compared to control specimens. Therefore, CFRP jacketing is clearly an effective method to confine concrete block masonry columns. In general, providing CFRP jackets reduced the scatter of test results, which usually found in the axial stress of concrete block masonry prisms. The axial stresses of confined prisms showed low COV compared to control specimens in Tables 3 and 4. The average strength gains for sets R10-L1 and R30-L1 are almost identical. This is mainly due to the low confinement effect eliminates the positive effect of rounding the corner radius. The average strength gain were 12% and 29% for 10 mm and 30 mm rounded corner prisms with two-layer compared to the one-layer prisms, respectively. It can be observed that increasing the thickness of jackets by a second layer of CFRP provided a noticeable gain in the axial strength compared to the one-layer prisms. The average change in strength gain was about 3% between R10-L3 and R10-L2. In case of R30-L3, the average axial strength gain dropped 1% compared to R30-L2. It can be concluded that adding a third layer of CFRP did not lead to a significant gain in the axial strength compared to the two-layer prisms. However, adding a third layer of CFRP improved significantly the ultimate strain compared to two-layer prisms. This observation indicates a polynomial relationship between confinement levels and the strength gain of concrete masonry prisms. This is in agreement with the model proposed by Farnia [23]. However, considering the natural variability typically found in testing masonry and considering that only two prisms with three layers have been tested and compared against three prisms with two layers which expected to produce less reliable average and COV data. The authors suggest further experimental work by testing concrete masonry prism confined with much higher levels of confinement to validate this observation. In fact, the high level of confinement would eliminate the uncertainties due to the natural variability of masonry. Based on experimental data obtained by this work and Farnia [23], assuming a polynomial relationship between confinement levels and the strength gain of concrete masonry columns seems to be reasonable.

On the other hand, the R30-L2 reached, on average, 14% higher strength compared to the R10-L2. In the case of three layers, the difference decreased to 9% between R30-L3 and R10-L3. That is attributed to the increasing in corner radius of cross section, which increased the effectively confined area.

For one layer prisms with 10 mm corner radius, R10-L1, the average increase of ultimate axial strain were 74% higher than unreinforced prisms. The R10-L2 and R10-L3 prism sets gave an average ultimate axial strain increase of 110% and 77%, compared to control specimens, respectively. The reduction in ultimate axial strain at 15% degradation between R10-L3 and R10-L2 are mainly caused by the sharp drop of stress-strain curve after the peak in the R10-L3 set. However, R10-L3 prisms showed 67% increase in average ultimate axial strain at 50% degradation compared to R10-L2. Large increment of ultimate axial strain compared to control specimens have been observed for prisms rounded with 30 mm. The prisms gave an average ultimate axial strain increase of 82%, 226%, and 281% for R30-L1, R30-L2, and R30-L3, respectively. It can be observed that increasing the jacket thickness increased the strain ductility of prisms. The ultimate axial strain of R30-L1 reached 5% higher than R10-L1 due the deference in corner radius. In case of two layers, the difference in the ultimate axial strain between R30-L2 and R10-L2 were 55%. R30-L3 and R10-L3 prisms exhibited higher difference equal to 115%.

3.3. Effective tensile strain in CFRP jackets

For accurate estimation of the confinement pressure due to CFRP jacketing, a precise measurement for the effective tensile strain in CFRP jackets is required. The average readings of the effective tensile strain of CFRP jackets at the peak strength along the faces of the prism are shown in Fig. 14. Each value represents average of six or four measurements on CFRP jacket. Two longitudinal strain gages, one at the top and one at the bottom of prism, multiplied by three, or two replicate specimens. The erroneous readings are excluded from the average in case the strain gauge debonded during the preparation or the test. Fig. 14 demonstrates that the strains at CFRP laminate at maximum loads are very low. R30-L2 set reached maximum average value at one face equal to 2838 μ strain. Fig. 15 shows average effective tensile strain in CFRP jackets at 15% strength degradation. It can be clearly seen that the CFRP strain significantly increased up to 5541 μ strain in R30-L2 set.

Table 6 presents the normalized effective tensile strain in CFRP jackets at peak strength and at 15% strength degradation to the ultimate strain of CFRP laminate obtained by tensile flat coupon tests. Each value represents the average readings of all strain gages applied on the set of prisms. Average 48 strain gages reading in case of the set consists three replicate prisms and average readings of 32 strain gages when the set has only two replicate prisms. The strain ϵ_p is the effective tensile strains within the CFRP jackets at peak strength, whereas ϵ_u is the effective tensile strains of CFRP jackets at 15% strength degradation, and ϵ_{fu} is average of the maximum ultimate strain of CFRP laminates provided by tensile flat coupon tests.

From Table 6, it is clearly seen that the strain profile in CFRP jackets at peak load and at 15% strength degradation along the perimeter of the prisms was highly non-uniform. This is confirmed by the high coefficient of variation values. The phenomenon of non-uniform strain distribution in CFRP jacket is believed to be due that the heterogeneous combination of grout, concrete masonry blocks, and mortar promotes the formation of localized deformation and cracks since the deformation compatibility between the composite

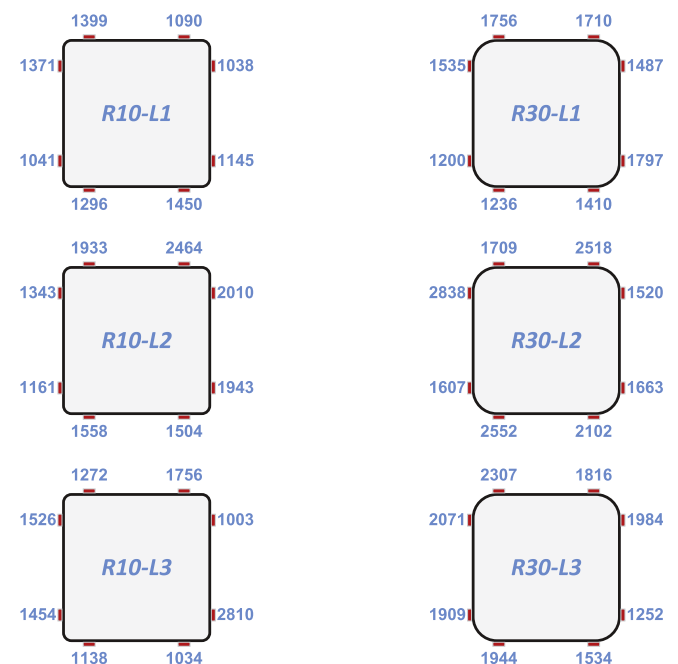


Fig. 14. Average readings of the effective tensile strain of CFRP at the peak strength along the faces of the prism (all readings in μ strain).

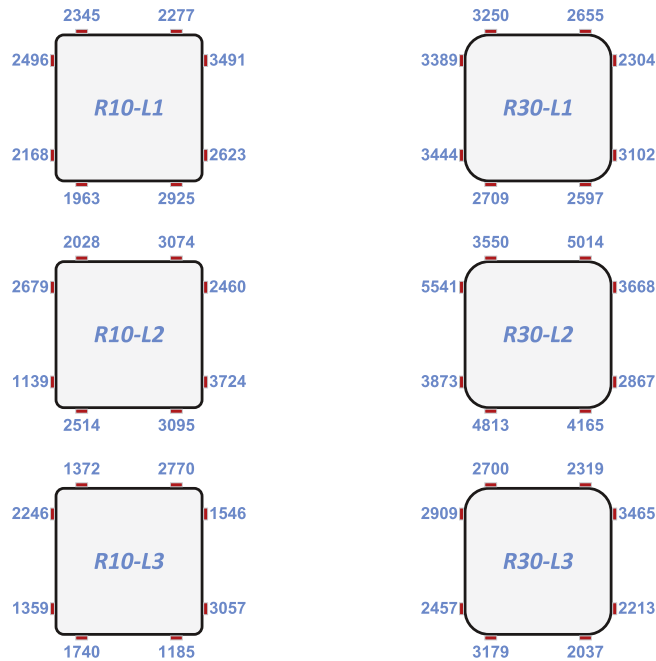


Fig. 15. Average readings of the effective tensile strain of CFRP at 15% strength degradation along the faces of the prism (all readings in μstrain).

jacket and concrete masonry prism is assumed. Furthermore, hand lay-up method creates fiber misalignment and produces variable thickness in the epoxy layer which influences the strain in the CFRP jackets.

Table 6 illustrates that the CFRP strains of 30 mm rounded prisms are greater than those of 10 mm corner radius with the same confinement level. Generally, the strain in CFRP jacket increases when the corner radius increases because the sharp edge effect has been reduced. One important observation is that the effective tensile strains in CFRP jacket at 15% strength degradation is much higher than CFRP strain at maximum load carrying capacity. The increase ranges between 27 and 103%. This indicates jacketing does not provide enough confinement pressure at maximum strength of prisms compared to that provided at the ultimate failure. The increase of confinement during the descending branch of the stress-strain curve mainly affects the axial strain of the prisms, which explains the higher gain in the strain comparing to the gain in strength.

The ratios of the measured tensile strain in CFRP jacket to the ultimate tensile strain of the flat coupon test for peak loading and 15% strength degradation are reported in Table 6. At peak strength, the ratio fell between 10% and 16%. At 15% strength degradation, this ratio increases to range between 14% and 32%. Only 18 out of 256 strain gages instrumented on the CFRP jackets captured strain values higher than 10000 μstrain at around 50% strength

degradation which is lower than the ultimate strain of CFRP material 13400 μstrain . The difference between the measured strain in the CFRP jacket and the ultimate strain of coupon test has been well researched in different works related to concrete confined by FRP (e.g. Refs. [35–44]). Various reasons have been proposed for this phenomena, including: stress concentration in FRP jacket produced by cracked concrete, which is expected to be more in masonry considering the heterogeneity of masonry; the curvature of the FRP jacket, especially in non-rounded corner columns; overlap regions in the FRP jacket; multiaxial stress condition generated on the FRP jacket; accidental load eccentricity caused by imperfections in the specimen or in the test setup; imperfections in manufacturing process or the laying-up process of fiber sheets; and uniformity of the resin in FRP laminate.

4. Comparison of analytical predictions with experimental results

The confinement model recommended by the Italian design guideline CNR-DT 200 R1 [26] will be assessed in this section. The experimental results are compared with the last updated version of the technical document for the use of FRP systems for strengthening existing structures published by the Italian National Research Council (CNR) in May 2014. The technical document provides an analytical confinement model to predict strength gains of masonry columns confined with FRP composites. The nonlinear confinement model suggests expressing the compressive strength of masonry columns confined with FRP as follows:

$$f_{mcd} = f_{md} \cdot \left(1 + k' \cdot \left(\frac{f_{l,eff}}{f_{md}} \right)^{\alpha_1} \right) \quad (1)$$

where f_{mcd} is characteristic compressive strength of FRP-confined masonry, f_{md} is compressive strength of masonry, $f_{l,eff}$ represents the effective confining pressure, and α_1 is a coefficient equal to 0.5 if further experimental data is not available. Unless a more detailed analysis is performed, the non-dimensional coefficient k' is indicated as the masonry mass density in kg/m^3 divided by 1000. The effective confining pressure, $f_{l,eff}$ is a function of the cross section shape and the FRP system as follows:

$$f_{l,eff} = k_{eff} \cdot f_l = k_H \cdot k_V \cdot f_l \quad (2)$$

where k_{eff} is the coefficient of efficiency expressed as the product of a horizontal and vertical coefficient of efficiency, k_H and k_V , respectively, and f_l is the lateral confining pressure. For square or rectangular cross sections with continuous FRP wrapping, the confinement lateral pressure can be calculated as follows:

$$f_l = 2 \cdot \frac{t_f \cdot E_f}{\max\{b, h\}} \cdot \epsilon_{fd,rid} \quad (3)$$

where E_f and t_f , are the tensile modulus of elasticity of FRP jacket

Table 6

The comparison of the CFRP effective tensile strains at peak strength, at 15% strength degradation and the ultimate strain of CFRP flat coupon tests.

Set	At maximum strength			At 15% strength degradation			Gain ratio
	ϵ_p (μstrain)	COV (%)	ϵ_p/ϵ_{fu} (%)	ϵ_u (μstrain)	COV (%)	ϵ_u/ϵ_{fu} (%)	
R10-L1	1330	76	10	2559	71	19	1.92
R10-L2	1767	75	13	2550	70	19	1.44
R10-L3	1499	56	11	1909	66	14	1.27
R30-L1	1526	58	11	2963	57	22	1.94
R30-L2	2077	58	16	4210	41	32	2.03
R30-L3	1852	52	14	2714	58	20	1.47

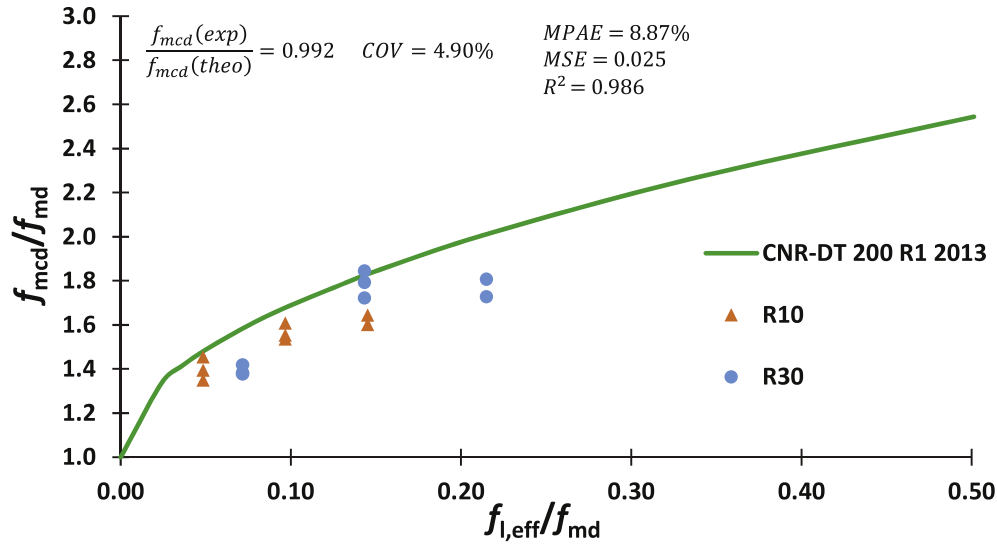


Fig. 16. Comparison between the experimental results and the theoretical predictions by CNR-DT 200 R1.

and the FRP thickness. The calculation of the theoretical prediction in this section is based on nominal thickness of cured laminate at 0.381 mm for one layer and 69.36 GPa tensile modulus of elasticity of CFRP jacket obtained by flat coupon test. The width b and the thickness h are the cross section dimensions. The reduced design value of the FRP strain, $\varepsilon_{fd,rid}$, can be computed as follows:

$$\varepsilon_{fd,rid} = \min \left\{ n_a \cdot \frac{\varepsilon_{fk}}{\gamma_f}; 0.004 \right\} \quad (4)$$

where n_a is the environmental conversion factor, ε_{fk} and γ_f represent characteristic rupture strain and the partial factors of FRP sheets, respectively, and 0.004 is the maximum allowed strain. The environmental and safety coefficients n_a and γ_f were taken equal to one. Value of $\varepsilon_{fd,rid}$ equals to 0.004 was adopted while calculating the effective confining pressure for theoretical predictions. The coefficient of efficiency of square or rectangular columns confined by external FRP wrapping is:

$$k_{eff} = k_H \cdot k_V \quad (5)$$

$$k_H = \left(1 - \frac{b'^2 + d'^2}{3A_m} \right) \quad (6)$$

with $b' = b - 2r_c$ and $h' = h - 2r_c$, r_c is cross section corner radius, and A_m masonry column area, as shown in Fig. 1. For a continuous FRP wrapping, the vertical coefficient of efficiency k_V is equal to one. The confinement model suggests different formulas to calculate the lateral confining pressure for circular and rectangular masonry columns in case of non-continuous FRP wrapping and in case of installing FRP sheets with fiber bars running in the orthogonal direction of the strengthened column. However, these formulas are out of scope of this work. The comparison between the experimental results and CNR-DT 200 R1 theoretical predictions can be seen in Fig. 16. Strength gains f_{mcd}/f_{md} are plotted against $f_{l,eff}/f_{md}$. Differences between experimental results and theoretical predictions are calculated in Table 7. The absolute error is equal to the absolute value of experimental value minus theoretical prediction divided by the experimental value. The performance of confinement model was evaluated by using three statistical metrics: The mean absolute percentage error (MAPE), the mean square

error (MSE) and the absolute fraction of variance (R^2).

$$MAPE = \frac{\sum_{i=1}^n \left| \frac{exp_i - theo_i}{exp_i} \right|}{n} \quad (7)$$

$$MSE = \frac{\sum_{i=1}^n (exp_i - theo_i)^2}{n} \quad (8)$$

$$R^2 = 1 - \left(\frac{\sum_{i=1}^n (exp_i - theo_i)^2}{\sum_{i=1}^n theo_i^2} \right) \quad (9)$$

where $theo$ is the theoretical prediction, exp is the experimental data, and n is the total number of data.

From both Fig. 16 and Table 7, It can clearly concluded that CNR-DT 200 R1 [26] theoretical model can provide a good correlation between experimental data and theoretical prediction. Fig. 16 points out that the mean absolute percentage error is 8.87%. However, the model generally gives non-conservative predictions for concrete masonry columns confined with CFRP composite. The accuracy of the model could be improve by refining the theoretical expression against the experimental data.

The main difference between the experimental data and theoretical prediction comes from that the CNR-DT 200 R1 model overestimates the effective strain in CFRP jacket which lead to overestimate the effective confining pressure. According to the experimental test data, the maximum average strain in the CFRP jackets at peak strength was 2077 μ strain which is 48% lower than the strain limit 4000 μ strain set by the theoretical model. Reducing the maximum allowed strain to 2000 μ strain would improve the accuracy of the model and reduce the mean absolute percentage error (MAPE) to 4.92% with absolute fraction of variance (R^2) equal to 0.992. The refined model gives conservative predictions with average ratio of experimental data to theoretical predictions around 1.05. The comparison between the experimental results and refined CNR-DT 200 R1 theoretical predictions are shown in Fig. 17.

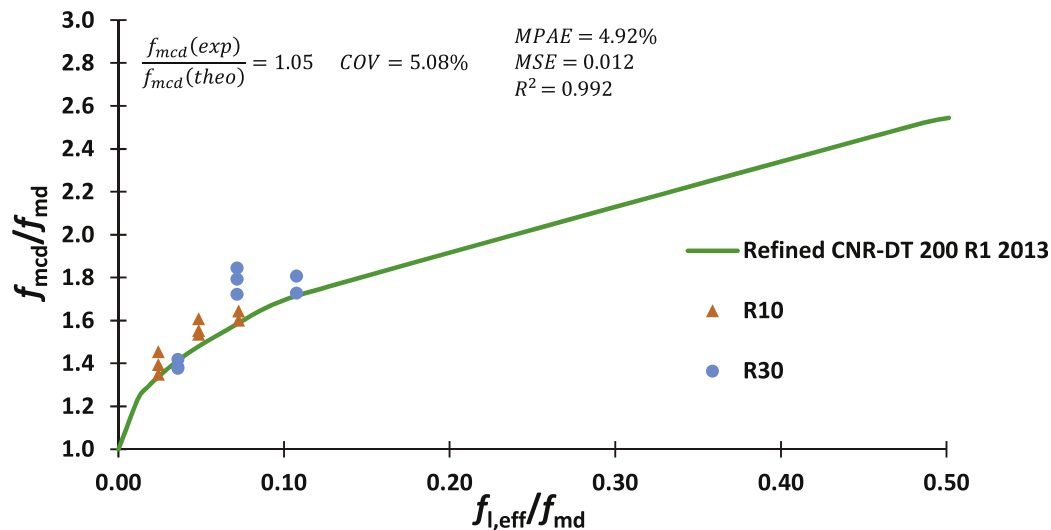
5. Conclusions

This paper presents an experimental investigation on the influence of CFRP confinement on the axial stress-strain behavior of

Table 7

Comparison between theoretical predictions and experimental results.

Prism	Experimental results	Theoretical predictions CNR-DT 200 R1/2013		Difference between experimental result and theoretical predictions		
	f_{mcd}/f_{md}	$f_{l,eff}/f_{md}$	f_{mcd}/f_{md}	$f_{mcd}(exp)/f_{mcd}(theo)$	Absolute error (%)	Average absolute error (%)
R10-L1-1	1.46	0.05	1.48	0.98	1.67	5.79
R10-L1-2	1.35			0.91	9.61	
R10-L1-3	1.39			0.94	6.10	
R10-L2-1	1.61	0.10	1.68	0.96	4.33	7.19
R10-L2-2	1.54			0.92	9.25	
R10-L2-3	1.55			0.93	7.98	
R10-L3-1	1.64	0.15	1.83	0.90	11.31	12.75
R10-L3-2	1.60			0.88	14.19	
R30-L1-1	1.38	0.07	1.58	0.87	14.51	13.70
R30-L1-2	1.42			0.90	11.62	
R30-L1-3	1.38			0.87	14.96	
R30-L2-1	1.72	0.14	1.83	0.94	5.94	2.96
R30-L2-2	1.79			0.98	1.83	
R30-L2-3	1.85			1.01	1.11	
R30-L3-1	1.81	0.21	2.01	0.90	11.24	13.79
R30-L3-2	1.73			0.86	16.34	

**Fig. 17.** Comparison between the experimental results and refined CNR-DT 200 R1 theoretical predictions.

concrete block masonry columns. Scaled fully grouted concrete masonry prisms with a square cross section were confined by continuous CFRP jackets to upgrade their performance level by increasing the ductility and the axial load capacity. The experimental investigation studied the role of corner radius and the CFRP jackets thickness in increasing the axial load and the ultimate strain. The tensile strain in CFRP jacket was measured in order to identify the CFRP lateral confining pressure. The experimental results of testing CFRP concrete masonry prisms has been used to assess and calibrate the analytical model suggested by CNRDT 200 R1 to predict strength gain of masonry column confined by FRP composites.

The following conclusions are drawn from the test results of 19 half-scale concrete block masonry prisms confined with CFRP:

- 1) CFRP jacketing proved to be an effective method for confining concrete block masonry columns. CFRP jacket significantly increased the peak strength and ultimate axial strain of the confined prisms compared to unreinforced prisms.
- 2) Concrete masonry columns confined with CFRP sheets exhibits enhancement in the post peak behavior by softening the

descending branches of the stress-strain relationships compared to unreinforced prisms.

- 3) A higher corner radius provides a higher ultimate axial strain for prisms with the same level of confinement.
- 4) The CFRP strain profile at peak load and 15% strength degradation along the perimeter of the prisms are highly non-uniform.
- 5) The tensile strains measured on the CFRP Jackets are lower than the average ultimate tensile strain obtained by flat coupons test.
- 6) The tensile strain in CFRP jacket increases when the corner radius increases.
- 7) CFRP jacketing provides higher confinement pressure at 15% strength degradation compared to that provided at the maximum strength of prisms.
- 8) The refined CNR-DT 200 R1 theoretical model can provide a good correlation with the experimental data with mean absolute percentage error around 4.92%.

Experimental tests were conducted on CFRP confined concrete masonry columns to collect necessary data to assess the CNR-DT 200 R1 theoretical model. The refined theoretical expression shows low mean absolute percentage error. However, further

experimental works investigating different FRP materials and considering various geometries of concrete masonry units are required to generalize the findings.

Acknowledgements

The first author gratefully acknowledges the financial support of Saudi Arabian Cultural Bureau in Canada and the scholarship sponsored by Imam Abdulrahman bin Faisal University. Sika Canada is acknowledged for donating CFRP materials. The financial support from the Natural Sciences and Engineering Research Council of Canada (NSERC) and Concordia University are acknowledged. The authors appreciate the in-kind support of the Canadian Concrete Masonry Producers Association (CCMPA) by providing half-scale blocks and masons.

References

- [1] Seible F, Priestley MN, Hegemier GA, Innamorato D. Seismic retrofit of RC columns with continuous carbon fiber jackets. *J Compos Constr* 1997;1(2): 52–62.
- [2] Ozbakkaloglu T, Lim JC. Axial compressive behavior of FRP-confined concrete: experimental test database and a new design-oriented model. *Compos Part B Eng* 2013;55:607–34.
- [3] Ozbakkaloglu T, Lim JC, Vincent T. FRP-confined concrete in circular sections: review and assessment of stress-strain models. *Eng Struct* 2013;49:1068–88.
- [4] ACI Committee 440. Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures ACI 440.2R-08. Farmington Hills, MI. 2008.
- [5] Concrete Society. Design guidance on strengthening concrete structures using fibre composite materials. Technical Rep. No. 55, London. 2004.
- [6] FIB. FRP reinforcement in RC structures. Fib task group 9.3, Lausanne. *fib Bull* 2007;40:1–147.
- [7] ISIS Canada. Strengthening reinforced concrete structures with externally-bonded fibre reinforced polymers. ISIS Canada design manual no. 4, Canada. 2001.
- [8] Masia MJ, Shrive NG. Carbon fibre reinforced polymer wrapping for the rehabilitation of masonry columns. *Can J Civ Eng* 2003;30(4):734–44.
- [9] Di Ludovico M, D'Ambra C, Prota A, Manfredi G. FRP confinement of tuff and clay brick columns: experimental study and assessment of analytical models. *J Compos Constr* 2010;14(5):583–96.
- [10] Faella C, Martinelli E, Paciello S, Camorani G, Aiello MA, Micelli F, et al. Masonry columns confined by composite materials: experimental investigation. *Compos Part B Eng* 2011;42(4):692–704.
- [11] Alecci V, Bati SB, Ranocchiai G. Study of brick masonry columns confined with CFRP composite. *J Compos Constr* 2009;13(3):179–87.
- [12] Krevaiakas TD, Triantafillou TC. Masonry confinement with fiber-reinforced polymers. *J Compos Constr* 2005;9(2):128–35.
- [13] Corradi M, Grazini A, Borri A. Confinement of brick masonry columns with CFRP materials. *Compos Sci Technol* 2007;67(9):1772–83.
- [14] Faella C, Martinelli E, Camorani G, Aiello MA, Micelli F, Nigro E. Masonry columns confined by composite materials: design formulae. *Compos Part B Eng* 2011;42(4):705–16.
- [15] Micelli F, Di Ludovico M, Balsamo A, Manfredi G. Mechanical behaviour of FRP-confined masonry by testing of full-scale columns. *Mater Struct* 2014;47(12):2081–100.
- [16] Aiello MA, Micelli F, Valente L. Structural upgrading of masonry columns by using composite reinforcements. *J Compos Constr* 2007;11(6):650–8.
- [17] Aiello MA, Micelli F, Valente L. FRP confinement of square masonry columns. *J Compos Constr* 2009;13(2):148–58.
- [18] Shaheen E, Shrive N. Sprayed glass fibre reinforced polymer masonry columns under concentric and eccentric loading. *Can J Civ Eng* 2007;34(11):1495–505.
- [19] Witzany J, Čejka T, Zigler R. Failure mechanism of compressed short brick masonry columns confined with FRP strips. *Constr Build Mater* 2014;63: 180–8.
- [20] Witzany J, Zigler R. Failure mechanism of compressed reinforced and non-reinforced stone columns. *Mater Struct* 2015;48(5):1603–13.
- [21] Micelli F, Angiuli R, Corvaglia P, Aiello MA. Passive and SMA-activated confinement of circular masonry columns with basalt and glass fibers composites. *Compos Part B Eng* 2014;67:348–62.
- [22] Galal K, Farnia N, Pekau OA. Upgrading the seismic performance of reinforced masonry columns using CFRP wraps. *J Compos Constr* 2011;16(2):196–206.
- [23] Farnia N. Seismic rehabilitation of reinforced masonry columns using CFRP and GFRP wraps. Ph.D. Dissertation. Concordia University; 2011.
- [24] Lam L, Teng J. Design-oriented stress-strain model for FRP-confined concrete in rectangular columns. *J Reinf Plastics Compos* 2003;22(13):1149–86.
- [25] Priestley M, Elder D. Stress-strain curves for unconfined and confined concrete masonry. *J Proc* 1983:192–201.
- [26] CNR-DT 200 R1. Guide for the design and construction of externally bonded FRP systems for strengthening existing structures. CNR-DT 200 R1, Rome. 2013. p. 1–167.
- [27] Long LM. Behaviour of half-scale reinforced concrete masonry shear walls. MSc Dissertation. McMaster University; 2006.
- [28] ASTM C140/C140M-15ae1. Standard test methods for sampling and testing concrete masonry units and related units. West Conshohocken, PA: ASTM International; 2015.
- [29] ASTM C476-10. Standard specification for grout for masonry. West Conshohocken, PA: ASTM International; 2010.
- [30] ASTM C270-14a. Standard specification for mortar for unit masonry. West Conshohocken, PA: ASTM International; 2014.
- [31] ASTM C780-15a. Standard test method for preconstruction and construction evaluation of mortars for plain and reinforced unit masonry. West Conshohocken, PA: ASTM International; 2015.
- [32] Sikawrap Hex 230C. Product data sheet: carbon fibre fabric for structural strengthening system (Edition 05.2013/v1). 2013. https://can.sika.com/dms/getdocument.get/3e96623c-3e8e-3d4d-b95e-813eb7f9f87d/SikaWrapHex230C_pds.pdf. Retrieved 28.12.2016.
- [33] ASTM D3039/D3039M-14. Standard test method for tensile properties of polymer matrix composite materials. West Conshohocken, PA: ASTM International; 2014.
- [34] ASTM D7565/D7565M-10. Standard test method for tensile properties of polymer matrix composite materials. West Conshohocken, PA: ASTM International; 2010.
- [35] Mirmiran A, Shahawy M, Samaan M, Echary HE, Mastrapa JC, Pico O. Effect of column parameters on FRP-confined concrete. *J Compos Constr* 1998;2(4): 175–85.
- [36] Harries KA, Carey SA. Shape and “gap” effects on the behavior of variably confined concrete. *Cem Concr Res* 2003;33(6):881–90.
- [37] Lam L, Teng J. Design-oriented stress-strain model for FRP-confined concrete. *Constr Build Mater* 2003;17(6):471–89.
- [38] Lam L, Teng J. Ultimate condition of fiber reinforced polymer-confined concrete. *J Compos Constr* 2004;8(6):539–48.
- [39] Cui C. Behaviour of normal and high strength concrete confined with fibre reinforced polymers (FRP). University of Toronto; 2009.
- [40] Wu Y-F, Jiang J-F. Effective strain of FRP for confined circular concrete columns. *Compos Struct* 2013;95:479–91.
- [41] Alecci V, Bati SB, Ranocchiai G. Concrete columns confined with CFRP wraps. *Mater Struct* 2014;47(3):397–410.
- [42] Shahawy M, Mirmiran A, Beitelman T. Tests and modeling of carbon-wrapped concrete columns. *Compos Part B Eng* 2000;31(6–7):471–80.
- [43] Zinno A, Lignola G, Prota A, Manfredi G, Cosenza E. Influence of free edge stress concentration on effectiveness of FRP confinement. *Compos Part B Eng* 2010;41(7):523–32.
- [44] Lignola GP, Nardone F, Prota A, Manfredi G. Analytical model for the effective strain in FRP-wrapped circular RC columns. *Compos Part B Eng* 2012;43(8): 3208–18.