

Out-of-Plane Flexural Performance of GFRP-Reinforced Masonry Walls

Khaled Galal¹ and Navid Sasanian²

Abstract: The objective of this paper is to assess the out-of-plane flexural performance of masonry walls that are reinforced with glass fiber-reinforced polymers (GFRPs) rods, as an alternative for steel rebars. Eight 1 m×3 m full-scale walls were constructed using hollow concrete masonry units and tested in four-point bending with an effective span of 2.4 m between the supports. The walls were tested when subjected to increasing monotonic loads up to failure. The applied loads would represent out-of-plane loads arising from wind, soil pressure, or inertia force during earthquakes. One wall is unreinforced; another wall is reinforced with customary steel rebars; and the other six walls are reinforced with different amounts of GFRP reinforcement. Two of the GFRP-reinforced walls were grouted only in the cells where the rods were placed to investigate the effect of grouting the empty cells. The force-deformation relationship of the walls and the associated strains in the reinforcement were monitored throughout the tests. The relative performance of different walls is assessed to quantify the effect of different design variables. The range of GFRP reinforcement ratios covered in the experiments was used to propose a capacity diagram for the design of FRP-reinforced masonry walls similar to that of reinforced concrete elements.

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Introduction

Masonry walls are considered as one of the most common structural masonry elements that are broadly employed to undertake axial and lateral loads. Depending on the application and also the orientation of masonry walls, these elements can be submitted to out-of-plane bending actions arising from wind, soil pressure, or seismic excitations, in which situations, the role of flexural reinforcement is critically influential in flexural strength, behavior, and serviceability of the walls.

Fiber-reinforced polymers (FRPs) have been studied and used extensively to reinforce concrete structures as a new substitute for steel reinforcement for more than a decade. FRP bars have been proven to be an effective means to replace steel reinforcement in various concrete structures such as bridge deckings and parking garages. In addition to their superior durability, mainly due to outstanding noncorrosive characteristics, these composite materials have the benefits of high strength-to-weight ratio, considerable fatigue properties, and electromagnetic transparency. Moreover, their usage in concrete structures has been codified in Canadian Highway Bridge Design Code (CHBDC) [Canadian Standards Association (CSA) 2006]. Lower fire resistance and higher costs are considered as the disadvantages therein. However, the former

is not an issue in the case of concrete masonry in that the minimum required cover, which is found to be 64 mm or less (Saafi 2002; CSA 2006), is fulfilled by the dimensions of masonry units. Moreover, the decreasing cost of FRP as well as lower transportation and handling costs of lighter materials are making the use of FRP in construction more competitive (Grace et al. 1998). Significant amount of work has been directed to using different types of FRP that are externally bonded or mounted on the surface of masonry walls for strengthening and rehabilitating reinforced and unreinforced masonry (URM) walls. It is reported by several researchers that notably higher shear and flexural strength as well as sufficient ductility can be attained by adding these external FRP systems to deficient masonry panels. On the contrary, to the authors' knowledge, there is not much work corresponding to the use of FRP for reinforcing masonry walls and, in particular, there has been no effort to exploit FRP as an interior reinforcement of masonry structural elements.

Previous Studies

Similar to concrete, the major key to the failure of masonry elements in flexure is the lack of sufficient tensile strength. Over the past four decades, considerable studies have been developed to assess and increase the resistance of masonry walls subject to lateral loads, which mainly started by introducing steel rebars as the main reinforcing element to compensate for poor tensile strength in masonry assemblage. Abboud et al. (1996) carried out full-scale out-of-plane bending tests thoroughly on steel-reinforced masonry walls as a part of the U.S.-Japan Coordinated Program on Masonry Building Research (TCCMAR), in which the postpeak behavior and displacement ductility of the reinforced masonry walls were inspected for the first time. The reported postyield load-deformation curves proved the significant effect of

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longitudinal reinforcement on improving the deficient flexural resistance of URM walls and justified its necessity.

Literature review indicates remarkable research with the purpose of improving the flexural capacity of URM and steel-reinforced masonry walls in terms of retrofitting and rehabilitation using new techniques and materials specifically FRP, while there is not much data available with regard to reinforcing new masonry walls with FRP to achieve higher level of lateral load-bearing capacity. Mierzejewski et al. (2008) investigated the behavior of strengthened URM walls made of hollow concrete blocks under out-of-plane bending. Different types of materials, including glass FRP (GFRP) bars mounted in the precast grooves and carbon-fiber-reinforced polymer (CFRP) strips epoxy glued on the surface of the specimens, were examined and compared with each other. Besides the significant increase in capacity and ductility, increasing the effective depth of the reinforcement by spreading them away from neutral axis and also avoiding grouting the cells were the other substantial advantages of the suggested methods. Near surface mounted (NSM) reinforcement instead of internal reinforcement was also proposed as a new approach in constructing reinforced masonry, providing that blocks with molded grooves could be fabricated. Comparable experiments were reported by Galati et al. (2006), in which they investigated the improved performance and modes of failure of URM walls strengthened with NSM FRP bars, considering different composite materials (GFRP and CFRP), ratios, and shapes (circular and rectangular) as well as embedding details as the main parameters of the study. It was proven that "flexural strengthening with FRP systems" enhances the flexural strength and pseudoductility of masonry walls to a great extent. Turco et al. (2006) conducted a similar study that concerns the issue of retrofitting URM for shear and flexure walls with NSM FRP bars, asserting the conclusions drawn by the aforementioned researchers. However, debonding of the GFRP rods, which was observed as the mode of failure of some of the walls tested in these research works, underscores the effectiveness of internal reinforcement.

Several researchers have carried out experimental studies that assess the use of externally bonded FRP whether it be sheets or fabrics of GFRP or CFRP (Gilstrap and Dolan 1998; Hamoush et al. 2001; Silva et al. 2001; Kiss et al. 2002; Ghobarah and Galal 2004; Tan and Patoary 2004). Remarkable boost to the out-of-plane load-carrying capacity and ductility of unreinforced and reinforced masonry walls, despite the brittle behavior of both FRP and masonry assemblage, is the common major outcome that can be found in all of them while parameters, such as thickness of the layers, adhesive material, surface preparation method, and surface mounting approach, were the decisive variables in the experiments.

Significance and Parameters of the Research

This paper examines the use of GFRP rods as a new reinforcing technique in concrete masonry walls and its effect on their out-of-plane flexural behavior through an experimental study on eight full-scale masonry walls, six of which were reinforced with GFRP rebars. The GFRPs used in this study are sand-coated rebars known as V.ROD and manufactured by Pultrall Inc. (located in Qué., Canada) (Pultrall 2007). The major objective of the research is to reach higher flexural capacity for the same cross-sectional dimensions while the deformability is maintained satisfactory. It is needless to say that durability problems, such as

corrosion, would be also eliminated by the nature of GFRP materials. Thus, it is intended to comprehend whether the possibility of overreinforcing the section with GFRP, as opposed to steel, resolves the upper limit of flexural strength for a certain cross-sectional dimension that confronts the designer when choosing steel as the reinforcing material for masonry. Furthermore, it is necessary to investigate the behavior of the GFRP-reinforced walls prior to failure, with regard to their deformability, for the compressive failure of masonry is imposed to be the mode of failure.

The main parameter of the research is the ratio of GFRP reinforcement that varies for GFRP-reinforced specimens included in the experimental program. The effect of (not) filling the cells, into which the longitudinal reinforcement is not positioned, has been considered as another aspect of this study that leads to a more efficient design. The relative performance of the walls with different reinforcement ratios demonstrates the variation of their capacity with respect to their reinforcement ratio. The foregoing, eventually, is used to verify the proposed design diagram which can be used to obtain the proper amount of GFRP reinforcement for a given flexural demand similar to that of FRP-reinforced concrete (FRP-RC). In addition, the behavior of the GFRP-reinforced specimens compared with the control specimens, unreinforced, and steel reinforced exhibits the effectiveness of the suggested reinforcement method.

This research work is restricted to the behavior of GFRP-reinforced masonry walls subjected to pure out-of-plane bending and the authors acknowledge that axial and beam-column behavior of the FRP-reinforced masonry walls should be investigated through further studies and experiments. Nevertheless, the experiments conducted in this research yield to a better perceptive of masonry walls' inelastic behavior in out-of-plane bending when reinforced with GFRP rods that can assist in theorizing a limit-states design methodology in future codes for masonry structures reinforced with FRP.

Experimental Program

The experiments that are carried out as a part of this study mainly consist of two parts: (a) the auxiliary tests that are meant to provide the structural characteristics of the construction materials and also the whole masonry compound and (b) main tests that include testing eight full-scale masonry walls that are GFRP-reinforced, steel-reinforced, and unreinforced. All the auxiliary specimens (masonry prisms) and full-scale walls were constructed by domestic professional masons, representing the current method of practice in Québec during four consecutive days.

Concrete Masonry Units

The masonry unit that is used in this study is hollow concrete block available by domestic supplier with nominal dimensions of $390 \times 190 \times 190$ mm. The minimum nominal compressive strength of the unit is 15 MPa and the average net-to-gross area ratio is 0.54 as per the supplier's provided specifications.

Mortar

Type S mortar, that is, a mixture of 0.5-volumetric units Portland cement, one-unit masonry cement, 2.9-units sand, and 0.7-units water, was chosen after several trial mixtures to be conforming to the requirements brought in ASTM (2002b) and CSA (2004b).

Table 1. Summary of the Test Results for the Compressive Auxiliary Prisms

Prisms	Failure load (kN)	Compressive strength (MPa)	Axial strain at peak stress
1	707	11.6	0.0017
2	538	8.9	0.0019
3	596	9.8	0.0020
4	687	11.3	0.0024
5	652	10.7	0.0020

Since the compressive strength of the mortar was highly sensitive for each day of construction, a series of six 50-mm-mortar cubes were sampled and tested according to ASTM (2002e) at the age of seven and 28 days. The average seven-day and 28-day compressive strengths were reported 8.6 MPa and 20.7 MPa, 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 (2004b) and ASTM (2002b) for laboratory made mortar cubes.

Grout

The grout used in the program, categorized as “coarse grout” in accordance with CSA (2004a) and ASTM (2002a), was a mixture of one volumetric unit Portland cement, 2.8-units fine aggregate (sand), two units coarse aggregates with the maximum size of 7 mm (1/4”), and 0.9-units water. The average compressive strength at the age of 28 days was 21.6 MPa. Slump test of fluid grout was done every day of the construction period reporting 260–280 mm slump that assures the lower limit in the Canadian code (CSA 2004a).

Auxiliary Tests for the Masonry Assemblage

To determine the compressive strength of the masonry assemblage, a series of five unreinforced grouted prisms were tested as stated in ASTM (2002c). It was decided to build five-block high and one-block wide prisms for this purpose to minimize the influence of slenderness by providing a central uniform stress zone so that the prism is a better representative of the real circumstances in the walls (Drysdale and Hamid 2005). The axial deformation over a gauge length of 600 mm on both sides of each prism was acquired by displacement transducers (potentiometers) and recorded as well as the applied load continuously up to the failure point. The summary of the related results is tabulated in Table 1. The average compressive strength (f'_m) was determined to be 10.9 MPa with a coefficient of variation of 6.4%, neglecting the result of the second prism. The second prism failed prematurely due to the unevenness of the upper loading surface, where a crack initiated at early stage of loading and caused the failure. The Young's modulus (E) of the prisms is also determined to be 6.0 GPa based on a secant line between 5 and 33% of average ultimate load. CSA (2004a) specifies E , using $850 \times f'_m$, equal to 9.2 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 has attested that the properties of the grout are of the major factors, affecting the failure of grouted concrete masonry, which may occur even at lower stress levels compared to ungrouted concrete masonry (Dhanasekar and Shrive 2002). The relatively lower strength of the prisms of this program, compared to their constituents, can be attributed to the excess of grout expanding inside the

Table 2. Summary of the Test Results for Tensile Auxiliary Prisms

Prisms	Failure load (kN)	Modulus of rupture (MPa)
1	11.9	1.06
2	12.2	1.09
3	14.5	1.28
4	11.1	0.99
5	8.1	0.74

cells that resulted in splitting the face shells. 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 cracking moment of the walls, another series of five prisms were tested by third-point loading method described in ASTM (2002d). Prisms with the height of seven blocks and width of one block were tested such as to locate the two point loads and supports in the middle of the block and also to provide for sufficient span-to-depth ratio (ASTM 2002d). The midspan deflection of the prisms was measured during each test using a potentiometer to establish the load-deflection curves. The average flexural bond strength of the prisms, also referred to as modulus of rupture (R), was found to be 1.11 MPa with a coefficient of variation of 9.5%, disregarding the result of the last prism (see Table 2). The last prism failed at a considerably weak bed joint outside of the constant moment zone. Table 3 summarizes the properties of the masonry assemblage and its constituents.

Tests on Full-Scale Reinforced Masonry Walls

The main part of the experimental program in this study consists of testing eight full-scale walls under out-of-plane bending. Each specimen is a single-wythe masonry wall with nominal dimensions of 1 m $\times$ 3 m that is made of 15 courses of two and a half concrete blocks with half running bond. As shown in Table 4 and Fig. 1, the first wall is unreinforced, the second wall is reinforced with customary steel rebars, and the other six walls are reinforced with different GFRP reinforcement ratios. Wall G-3#13-P and G-4#13-P are identical, with regard to the reinforcement area, to wall G-3#13-F and G-4#13-F, respectively; yet they are only grouted at the locations of the longitudinal reinforcements to study the effect of extent of grouting. The reinforcing rods were intended to be placed in the middle of the cells; however, positioning the rebars exactly in the middle of the blocks was not feasible during the construction. This could be attributed to the relative flexibility of GFRP rods compared to steel rebars. As such, the actual effective depth was measured after each test, at the failed section, to be used in the analyses. The nominal and actual arrangements of the reinforcing rebars are tabulated in Table 4. Although the tested masonry walls were not subjected to

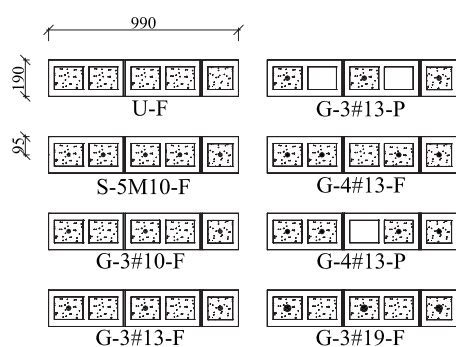
Table 3. Properties of the Masonry Assemblage and Its Constituents

Characteristic	Average (MPa)	C_v (%)
Compressive strength of masonry unit	15.0	—
Compressive strength of mortar	20.7	26.2
Compressive strength of grout	21.6	3.9
Compressive strength of masonry assemblage (f'_m)	10.9	6.4
Modulus of rupture of masonry assemblage (R)	1.1	9.5

Table 4. Matrix of the Full-Scale Masonry Walls

Wall	Reinforcing material	Average effective depth (mm)		Reinforcement ratio (%)		Extent of grouting
		Nominal	Measured	Nominal	Measured	
U-F	—	—	—	—	—	Fully
S-5M10-F	Steel	95	90	0.53	0.56	Fully
G-3#10-F	GFRP	95	95	0.25	0.25	Fully
G-3#13-F	GFRP	95	100	0.42	0.40	Fully
G-3#13-P	GFRP	95	105	0.50	0.45	Partially
G-4#13-F	GFRP	95	105	0.56	0.51	Fully
G-4#13-P	GFRP	95	125	0.61	0.46	Partially
G-3#19-F	GFRP	95	125	0.89	0.69	Fully

any horizontal load, bed joint reinforcement that consisted of truss type 3.66-mm gauge wire was placed for all the main specimens with the spacing of 400 mm (i.e., every other joint) since it is the common practice in contemporary construction of concrete masonry walls. The wall designation refers to (1) type of reinforcement (Steel, GFRP, or Unreinforced); (2) number and size of the rebars; and (3) the extent of grouting (Full or Partial). The walls were designed to have adequate shear capacity so that a compression failure in concrete block due to flexural action would be the dominant mode of failure. The longitudinal reinforcing bars were pushed into the grouted cells immediately after grouting the cells. Table 5 shows the properties of the GFRP rods used in the experimental program. 400-MPa-ordinary steel rebars with ribbed surface were used in reinforcing wall S-5M10-F. Walls were constructed, cured, and hardened vertically. Afterward, they were transferred and laid horizontally on the setup using a steel braced frame designed for this purpose. After the failure of four walls, the load was removed to determine the permanent (inelastic) displacement and applied again gradually to examine the postfailure strength.

**Fig. 1.** Cross section of the eight tested full-scale masonry walls (dimensions in mm)**Table 5.** Properties of the GFRP Rods Used in this Research

Diameter (mm)	Tensile modulus of elasticity (GPa)	Ultimate tensile strength (MPa)	Ultimate tensile strain
10	45.4	856	0.0189
13	46.3	786	0.0170
19	47.6	728	0.0153

Test Setup and Instrumentation

The full-scale masonry walls were tested in a third-point loading setup, shown schematically in Fig. 2, with an effective span of 2.4 m. Two point loads, at third spans, were applied and monotonically increased by a 15-ton hydraulic actuator, reacting against a rigid steel loading frame, up to the ultimate failure. Load was transferred through the loading apparatus and applied over two 152.4-mm (6")-wide channels to avoid crushing the blocks due to stress concentration. The walls were tested while positioned horizontally on a hinge at one end and a roller at the other end, similar to previous tests on masonry walls performed by several researchers (Galati et al. 2006; Tan and Patoary 2004; Kiss et al. 2002; Gilstrap and Dolan 1998; Hamoush et al. 2001). The reason was that it was not as convenient and secure to test them vertically. Potentiometers were located to record the displacement at 10 different points of each tested wall (i.e., three at midspan, six at third spans, and one at quarter span) to have the longitudinal profile of the wall at each stage and also to make sure that the wall is not tilted or twisted in width due to probable test setup imperfections. The maximum axial strain, assumingly in the midspan, for every reinforcing rod (GFRP and steel) was recorded using strain gauges that had been installed on the rods before construction. Furthermore, the strain in two of the reinforcing rods for each reinforced wall at quarter span was recorded.

Results of the Wall Tests

The following sections present the results and describe the observations associated to the wall tests. To begin with, crack patterns and deformed shape of the tested walls will be discussed. The second part expresses the relative performance of the walls in which both lateral deflection and tensile fiber strain versus the applied load are demonstrated in different series of comparison. Lastly, walls' behaviors at ultimate and also after failure will be discussed in the third part. Table 6 includes a summary of the test results of the eight full-scale masonry walls.

Cracks and Deformation

Based on flexural bond strength of the tested auxiliary prisms, no cracking was expected to occur due to the self-weight of the walls. The first crack was generated in the constant moment zone as the applied moment reached the vicinity of the expected cracking moment. More flexural cracks as well as shear cracks outside the constant moment zone started to appear by increasing the

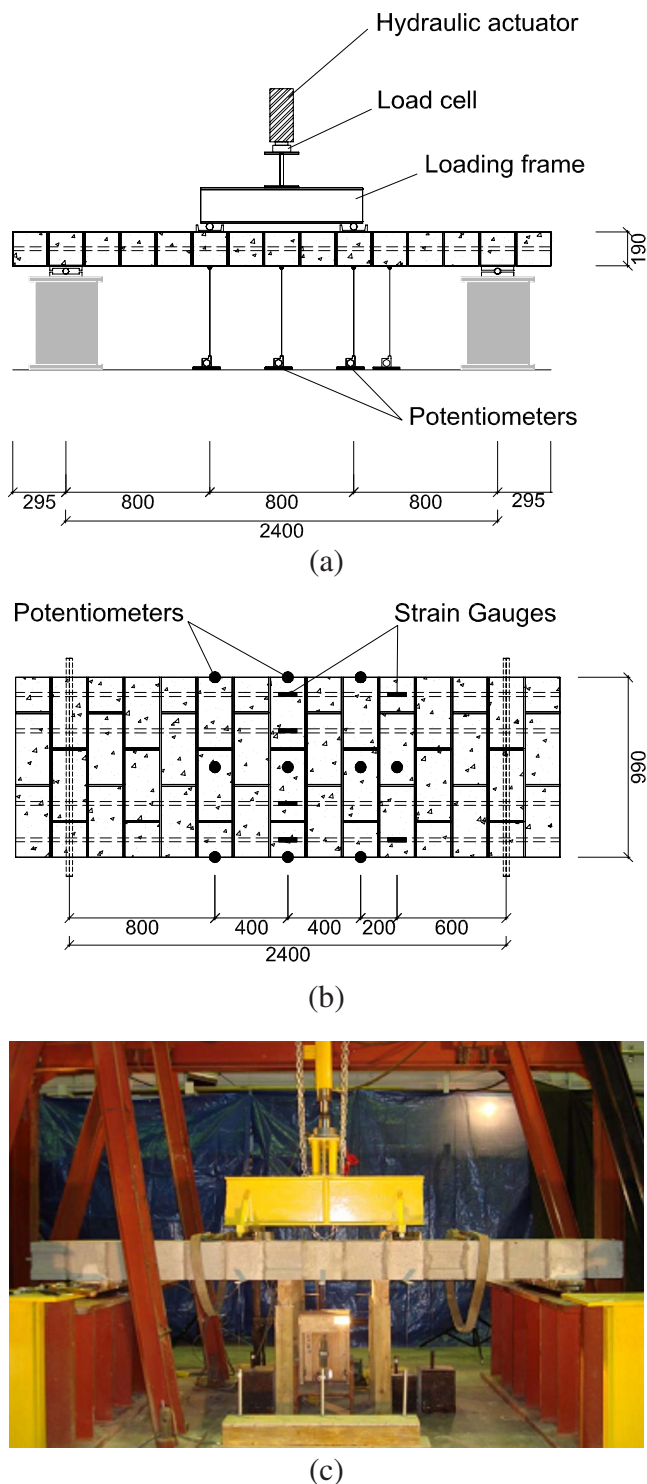


Fig. 2. Test setup and instrumentations (dimensions in mm): (a) elevation of the test setup for the full-scale walls; (b) plan and instrumentation for a typical wall reinforced with four rebars; and (c) test setup before the test

applied moment beyond the cracking moment, which also deepened and widened the existing cracks gradually. All the cracks were initiated primarily from the bottom of the section at the interface of masonry unit and mortar and developed upward in the grout. Each crack resulted in separating the block from mortar along the lower face shell at one instant and, as a result, the deflections increased dramatically.

The threshold of flexural strength was preceded by a big opening in one of the block-mortar joints between the two point loads and cracks in the concrete blocks that culminated in crushing of the masonry units in compression. In all the tested walls, that block-mortar (bed) joint was noticed to have shear reinforcement. In some of the tests, the mortar in the section of the cracked bed joint spalled off the compression side.

For the steel-reinforced wall, the shear cracks appeared, after the rebars had yielded, only in two bed joints, which underwent the constant shear force and bending moment close to the maximum value. The relatively higher contribution of steel reinforcement in shear strength compared to GFRP can explain less shear cracks compared to GFRP-reinforced walls. On the other hand, the less wide cracks in the GFRP-reinforced walls underline the superior bond characteristics of this type of reinforcement compared to steel.

Another finding was the effect that the extent of grouting had on the cracking load. The partially grouted walls cracked at fairly lower levels of applied load when compared to their analogous companions. However, it can be stated that the type of the reinforcement (steel versus GFRP) had not a significant effect on either the onset of cracking or the immediate width of the cracks.

Load-Deflection and Load-Strain Curves

It should be mentioned that to take into consideration the self-weight of the walls (between 6.9 and 12.3 kN), the equivalent load (dead load) that produces the same moment, as the distributed self-weight over the wall does, was added to the recorded live load, which was applied by the hydraulic actuator. This was done without missing any useful information; in that, as mentioned before, the expected cracking moment (6.6 kN m) was larger than the moment produced by the own weight of the walls (between 2.5 and 4.1 kN m). Furthermore, a pair of temporary supports were fabricated to carry the weight of the walls and removed after having launched the data acquisition system, so that the strains and deflections due to the self-weight of the walls would not be missed either. Having the equivalent dead load and its corresponding deflection and strain, the initial portion of the force-deformation and force-strain curves was superimposed to that of live load.

Fig. 3 shows the out-of-plane load versus the deflection at midspan of the GFRP-reinforced walls along with the steel-reinforced and unreinforced ones. The drastic drops in the recorded loads correspond to the cracks' occurrence, while the tiny drops can be associated with the existing cracks widening or extending in depth, which also happened intermittently. It is observed that after each drop, the load has caught up to a higher level but with a diminished stiffness. It illustrates how the section approaches to the cracked-section properties progressively.

The remarkably poor strength and deformation of the wall U-F simply depicts the drastic effect that the use of even smallest amount of GFRP reinforcement can have on the flexural performance of the masonry walls. More importantly, the sudden failure is replaced with ample deformation of the walls after the first crack.

As one of the authors' intentions, the relative performance of the steel-reinforced wall is also demonstrated to be compared with that of the wall, which was designed to have comparable amounts of GFRP reinforcement. In other words, the product of reinforcement area (A_{FRP}) and ultimate tensile strength ($f_{u,FRP}$) for

Table 6. Summary of the Test Results for the Full-Scale Walls

Wall	Analytical		Experimental		$M_{n,exp.}/M_{n,analyt.}$	Max defl. (mm)	Max tensile strain	Mode of failure
	M_{cr} (kN m)	M_n (kN m)	M_{cr} (kN m)	M_n (kN m)				
U-F	6.6	6.6	5.6	5.6	0.85	1.4	—	Tensile in bed joint
S-5M10-F	6.6	16.2	6.9	16.9	1.05	143.2	—	Compressive in masonry
G-3#10-F	6.6	13.1	6.0	14.2	1.09	111.4	0.0140	
G-3#13-F	6.6	17.7	7.6	18.3	1.03	107.4	0.0112	
G-3#13-P	6.2	17.6	4.3	17.9	1.02	108.5	0.0100	
G-4#13-F	6.6	21.4	5.8	21.7	1.01	108.9	0.0096	
G-4#13-P	6.4	27.3	5.9	30.3	1.11	94.3	0.0102	Compressive in masonry
G-3#19-F	6.6	34.2	6.2	39.3	1.15	93.6	0.0084	

wall G-3#10-F was intended to be close to that of the steel-reinforced wall. This product is calculated for wall G-3#10-F and S-5M10-F as shown below

$$A_{FRP}f_{u,FRP} = (3 \times 71.3 \text{ mm}^2)(856 \text{ MPa}) = 183 \text{ kN}$$

$$A_s f_y = (5 \times 100 \text{ mm}^2)(400 \text{ MPa}) = 200 \text{ kN} \tag{1}$$

where A_s and f_y =area and yield stress of the steel reinforcement. It can be seen that wall G-3#10-F exhibited slightly lower strength, as expected; even though the S-5M10-F experienced more deformation, the GFRP-reinforced wall could undergo adequate deflections to forewarn the failure.

Another notable observation is that the performance of wall G-3#19-F with the largest reinforcement ratio, compared to S-5M10-F, shows that we can reach higher capacities with acceptable deformability, which could not be achieved by steel-reinforced masonry walls due to the constraint of not exceeding the balanced reinforcement ratio.

Wall G-4#13-P exhibited significantly higher strength compared to wall G-4#13-F; this could be explained by the actual location of the GFRP rods inside the cells, which was determined for the walls after the tests. The average effective depth of the bars for wall G-4#13-P was found to be 125 mm, whereas that of wall G-4#13-F was found 105 mm (see Table 4). Nevertheless, looking at the relative performance of walls G-3#13-F and G-3#13-P (with comparable arrangement of GFRP reinforce-

ment), it can be concluded that grout filling the empty cells does not have a significant effect on the general performance of the walls.

Fig. 4 illustrates the changes in tensile strains of the longitudinal reinforcements with out-of-plane load. Since the reinforcing rods are in the vicinity of the neutral axis of the gross section, there is not much stress and, consequently, strain recorded before the occurrence of the first crack, at which stage the neutral axis is raised up notably. The effect of the cracks can similarly be detected herein. In general, the curves show that the tensile strain and, accordingly, the tensile stress in GFRP rebars increases linearly with the applied load up to the failure of the walls. The maximum recorded strains highlight the fact that the GFRP rods did not rupture and the compressive crushing in masonry caused the failure of the walls.

Modes of Failure and Postfailure Behavior

The unreinforced wall encountered a sudden failure immediately after one of the bed joints in the constant moment zone cracked and failed in tension; whereas for the reinforced walls, the failure was always in compression zone of the masonry compound even though it was preceded by different phenomena (see Fig. 5).

The performance of wall G-3#10-F was pleasing with regard to deformability as it undertook considerable amount of deformation prior to failure [Fig. 5(a)]. This specimen failed after the bed joint next to the point load inside the moment zone had opened up

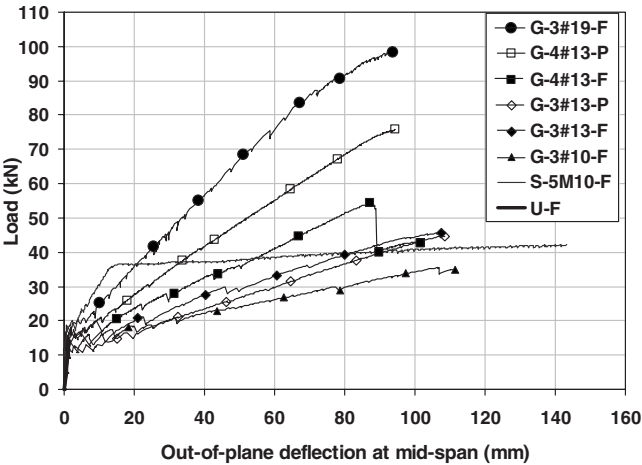


Fig. 3. Load-deflection performance of the tested walls

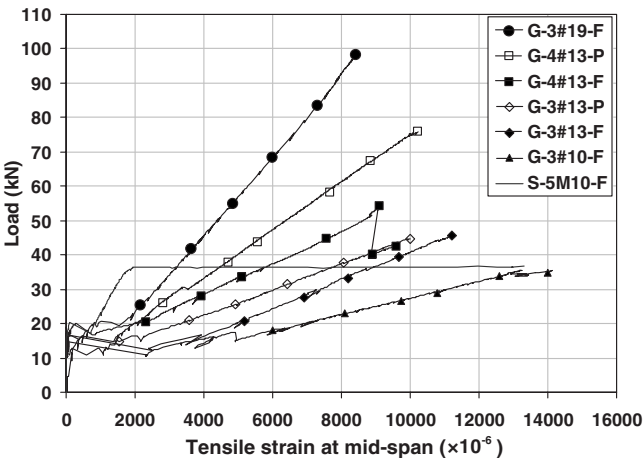


Fig. 4. Load-strain performance of the tested walls

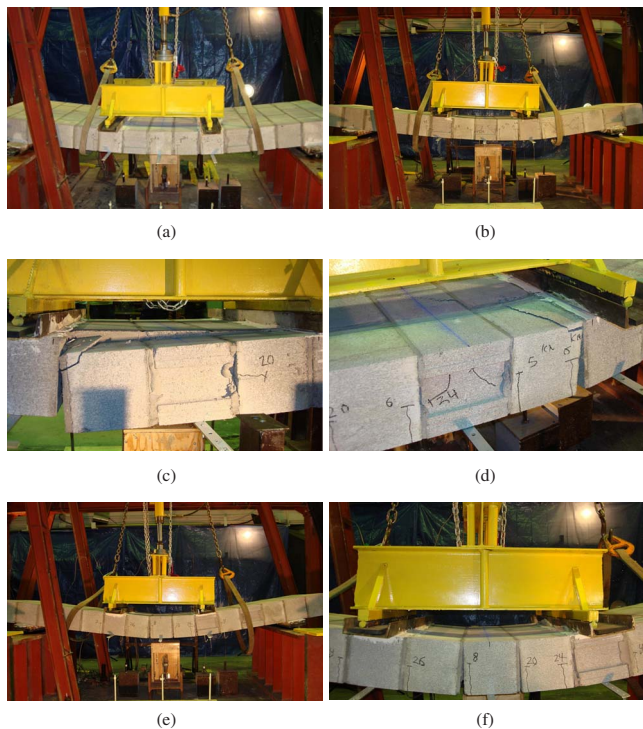


Fig. 5. Failure of the walls: (a) flexural cracks that occur at the joints in constant moment zone; (b) deformed shape of a GFRP-reinforced wall close to failure; (c) a strip of grouted blocks separated from wall G-3#13-P; (d) a wide diagonal crack developed on the compression side of wall G-3#13-F; (e) deformed shape of the steel-reinforced wall close to failure; and (f) no sign of diagonal cracks for the steel-reinforced wall

significantly and the top face shell of one cell next to the same bed joint was crushed. Same response occurred in G-3#13-F, G-4#13-F, G-4#13-P, and G-3#19-F; however, there were some diagonal cracks detected on the side of the blocks next to the failed joints [Fig. 5(b)]. G-3#13-P failed after a strip of three consecutive grouted cells separated on one side of the wall inside the constant moment zone [Fig. 5(c)]. A half block on one side was separated and misaligned before the failure of G-4#13-F. For the case of G-3#19-F, the cracks in the masonry units were relatively wide. G-3#13-F, G-4#13-P, and G-3#19-F experienced shear cracks on the side of masonry units outside the constant moment region. For G-3#13-F, these cracks joined a mortar-block interface next to the point load, which widened considerably; nevertheless, the failure was induced by a long and wide crack on the top surface of the wall that was diagonally developed over the constant moment zone [Fig. 5(d)]. Overall, it was observed in the course of GFRP-reinforced wall tests that the amount of GFRP reinforcement can significantly influence the pattern of shear cracks, in that walls with higher amounts of reinforcement experienced more severe cracking in the shear zone especially on the side of masonry units. While the walls with higher GFRP reinforcement ratios (i.e., G-4#13-P and G-3#19-F) reached higher levels of applied load, their deformations were comparable with the other specimens and satisfactory; thus, it can be said that by increasing the amount of GFRP reinforcement the desirable behavior of the walls would not be affected.

The steel-reinforced wall exhibited a ductile behavior, as expected, and failed after the joint cracks were widened and compressive cracks had appeared on the compression side of the

blocks, although no shear crack was detected on the masonry units [Figs. 5(e and f)]. This could be attributed to the dowel action of the steel rebars.

G-3#13-F, G-3#13-P, G-3#19-F, and S-5M10-F were unloaded and reloaded gradually after the failure to investigate their post-failure strength and inelastic deformation. Their inelastic deformations, as percentages of their total deformations, were found to be 7.8%, 39%, 36.5%, and 7.3%, respectively. The postfailure resistances corresponding to the first three were found to be 34.4%, 46.6%, and 47.3% of the ultimate flexural capacity, respectively. It can be concluded that the larger amounts of GFRP reinforcement increase both parameters significantly. The notable observation therein is that not only did the postfailure strength of the FRP-reinforced walls exceed their dead loads but it also helped to encounter considerable amount of applied live load.

Section Analysis in Flexure

The tested walls' section analysis is conducted using the common method of ultimate strength conditions that is adopted in CSA (2004a). For the case of GFRP-reinforced sections, necessary modifications are adopted to the procedures in accordance with The Canadian Network of Centres of Excellence on Intelligent Sensing for Innovative Structures (ISIS Canada) (2001). The approach employed in this part is assumed to be appropriate, in that all the tested walls failed due to the compressive failure of the masonry. The analysis is carried out assuming that (1) the strain varies linearly in the depth of the section; (2) deformations are small; (3) tensile strength in the masonry and compressive strength of composites are negligible; (4) the GFRP rods bond with concrete with no interfacial slippage; (5) the ultimate compressive strain of the masonry compound is 0.003, as suggested in CSA (2004a); and (6) all the GFRP-reinforced sections fail due to masonry crushing as they were designed to be overreinforced. The ultimate fiber tensile strains were to be checked against the fiber rupture strains ultimately to verify the last assumption. The choice of 0.003 as the ultimate masonry strain (with 0.002 as the strain corresponding to the peak stress) agrees with the average strain, corresponding to the peak stress for the compressive masonry prisms, which is 0.002 with the coefficient of variation of 0.12 (see Table 1). Moreover, the recorded strains in the GFRP rods of the tested walls were well below their ultimate tensile fiber strain, which shows that the sections were overreinforced. As for the GFRP rods, the stress-strain relationship is presumed to be linear elastic up to the point where the rupture happens. Their tensile properties, as can be seen in Table 5, vary slightly for different diameters. Although the rods were initially intended to be placed in the middle of the unit cells with an effective depth (d) of 95 mm, after the failure of each wall the actual d of the rods were measured at the failed section to have a more precise analysis. The average measured d ranged for the walls from 100 to 125 mm.

For the overreinforced walls, the equilibrium equations of internal forces were solved to find the tensile stress in the GFRP at ultimate state directly using Eq. (2)

$$f_{FRP} = 0.5 E_{FRP} \epsilon_{m,u} \left[\left(1 + \frac{4\alpha_1 \beta_1 f'_m}{\rho_{FRP} E_{FRP} \epsilon_{m,u}} \right)^{1/2} - 1 \right] \quad (2)$$

where ρ_{FRP} = actual GFRP reinforcement ratio of the wall (after the measurement of the depth of the rods); E_{FRP} = modulus of elasticity of the GFRP rod that varies for the three different di-

ameters slightly (Pultrall 2007); and $\varepsilon_{m,u}$ =ultimate compressive strain of the masonry compound. α_1 and β_1 =Whitney uniform stress block factors. The outcomes of the numerical evaluations state that the theoretical prediction is fairly conservative since the ratio of experimental to theoretical ultimate strength varies from 1.13 to 1.28. To have a more accurate prediction, in lieu of equivalent Whitney stress block, three different stress-strain models that account for more realistic stress-strain behavior of concrete masonry were adopted in the aforesaid procedure. Dhanasekar and Shrive (2002) proposed two stress-strain curves, defined by a simple and a refined equation, and found their proposed model reliable for predicting flexural behavior of reinforced masonry. Furthermore, Priestley and Elder (1983) suggested the use of another model for concrete masonry. All the three equations were integrated into the walls' section analysis leading to closer estimate of strengths, even though still slightly conservative. The one most conforming to the results of the tests was found to be the refined equation proposed by Dhanasekar and Shrive (2002) as shown below

$$\sigma_m = f'_m \left(\frac{[1 + u_0(1 + u_1)]x}{u_0(1 + u_1x) + x^{1+u_0}} \right) \quad (3)$$

where u_0 and u_1 =constants taken as 1.5 and 1.0, respectively, for grouted concrete masonry and 2.1 and 0.1 for ungrouted concrete masonry, x =ratio of strain to the strain at maximum stress and σ_m =compressive stress of masonry. Stress-strain relationship defined by Eq. (3) estimates the modulus of elasticity of masonry to be 10.7 GPa based on a secant line between 5 and 33% of f'_m . Theoretical flexural strength of the sections based on the stress-strain models were evaluated in the same way but using the two stress block factors α_1 and β_1 calculated for the real stress-strain curves. The ratio of experimental to theoretical ultimate strength using the chosen stress-strain curve varies from 1.01 to 1.15. The results of section analysis and wall tests are compared and summarized along with maximum lateral deflections and tensile strains, cracking moments, and modes of failure of the walls in Table 6.

The relatively lower theoretical values can be attributed to a number of causes. Concrete masonry as a nonhomogeneous complex could not be accounted for perfectly by the same approach that is used for concrete, unless its stress-strain relationship is established properly. In addition, the structural properties of the masonry assemblage are influenced by labor and materials; hence, they acquire notable discrepancy from day to day of the construction period. Another parameter conducive to the results mismatch can be the presence of grout cores inside the hollow concrete blocks. The theoretical analysis is carried out assuming that, according to the results of auxiliary tests, f'_m is 10.9 MPa. For the prisms tested under compression, the grout expansion was of major reasons initiating the failure, whereas for the walls tested under out-of-plane bending, the grout is not subjected to compression, thus, not expanding, since the depth of the compression zone (c), except for G-3#19-F, was always less than the face shell thickness. Therefore, the masonry compound can undertake larger amounts of stress in the compression zone leading to higher flexural resistance for the walls. Furthermore, it can be concluded that grouting the cells with no reinforcement is not necessarily effective, as seen before in the results of the wall tests, since the grout is not contributing to the flexural strength of the section.

Load-Deflection Prediction

As another facet of response of the walls to out-of-plane bending, it is invaluable to have the ability of foreseeing their force-deformation relationship up to the ultimate failure. To begin with, various models that have been proposed for predicting deflection in FRP-RC and masonry members are inspected and put on display in comparison to the results of the wall tests. Later on, a computer program is used for the same purpose; accordingly, efforts have been made to propose a finer numerical method of evaluating this characteristic of GFRP-reinforced walls based on the results and observations of the wall tests.

Compressive strength and modulus of rupture used in calculating the deflections are the ones obtained in the course of the auxiliary tests, while for the modulus of elasticity it was decided to use the one corresponding to the stress-strain model defined by Dhanasekar and Shrive (2002). The midspan deflections of tested walls before the cracking moment are calculated using the gross-section properties, whereas the postcrack deflections are calculated using the effective moment of inertia (I_{eff}) method, such that the tension stiffening below the neutral axis between the uncracked and cracked grout is taken into account.

Available Methods of Cracked-Section Analysis

CSA (2004a) defines an equation to calculate I_{eff} for steel-reinforced masonry similar to steel-reinforced concrete. Thus, it could be said that it is an appropriate approach to use the proposed I_{eff} for FRP-RC members in calculating the deflections of the GFRP-reinforced walls. There are various ways of determining I_{eff} , which are proposed and verified by several researchers (e.g., Horton and Tadros 1990; Brown and Bartholomew 1996; Thériault and Benmokrane 1998; Gao et al. 1998). However, all the different available numerical methods of deflection evaluation underestimate the out-of-plane deformation of the walls in view of the fact that they are either not taking the special characteristics and differences of concrete masonry into account or they have been developed based on a limited knowledge of FRP's bond characteristics. The methods proposed for FRP-RC would not completely be capable of explaining the significant openings that are imposed to take place in the bed joints of the maximum moment zone. The ones that are specialized for reinforced masonry, on the other hand, could not be representative of the special characteristics of FRP. The load-deflection relationships of the wall predicted, using I_{eff} defined by Gao et al. (1998), has been chosen herein, as the commonly acknowledged method, to be illustrated and compared with the test results; since, it has been recognized by The Canadian Network of Centres of Excellence on Intelligent Sensing for Innovative Structures (ISIS Canada) (2001) to be used for predicting the deflection of FRP-RC members in flexure. It is calculated using the following equation:

$$I_{eff} = I_{cr} + (\beta_b I_g - I_{cr}) \left[1 - \left(\frac{M_{cr}}{M_a} \right)^3 \right] \quad (4)$$

In which, I_{cr} and I_g =moment of inertia of the cracked and gross section; M_{cr} and M_a are the cracking moment and applied moment of the section at each stage; and last, β_b depending on the properties of the composites is calculated as follows:

$$\beta_b = 0.5 \left[1 + \frac{E_{FRP}}{E_{steel}} \right] \quad (5)$$

where E_{steel} =modulus of elasticity of the steel rebars.

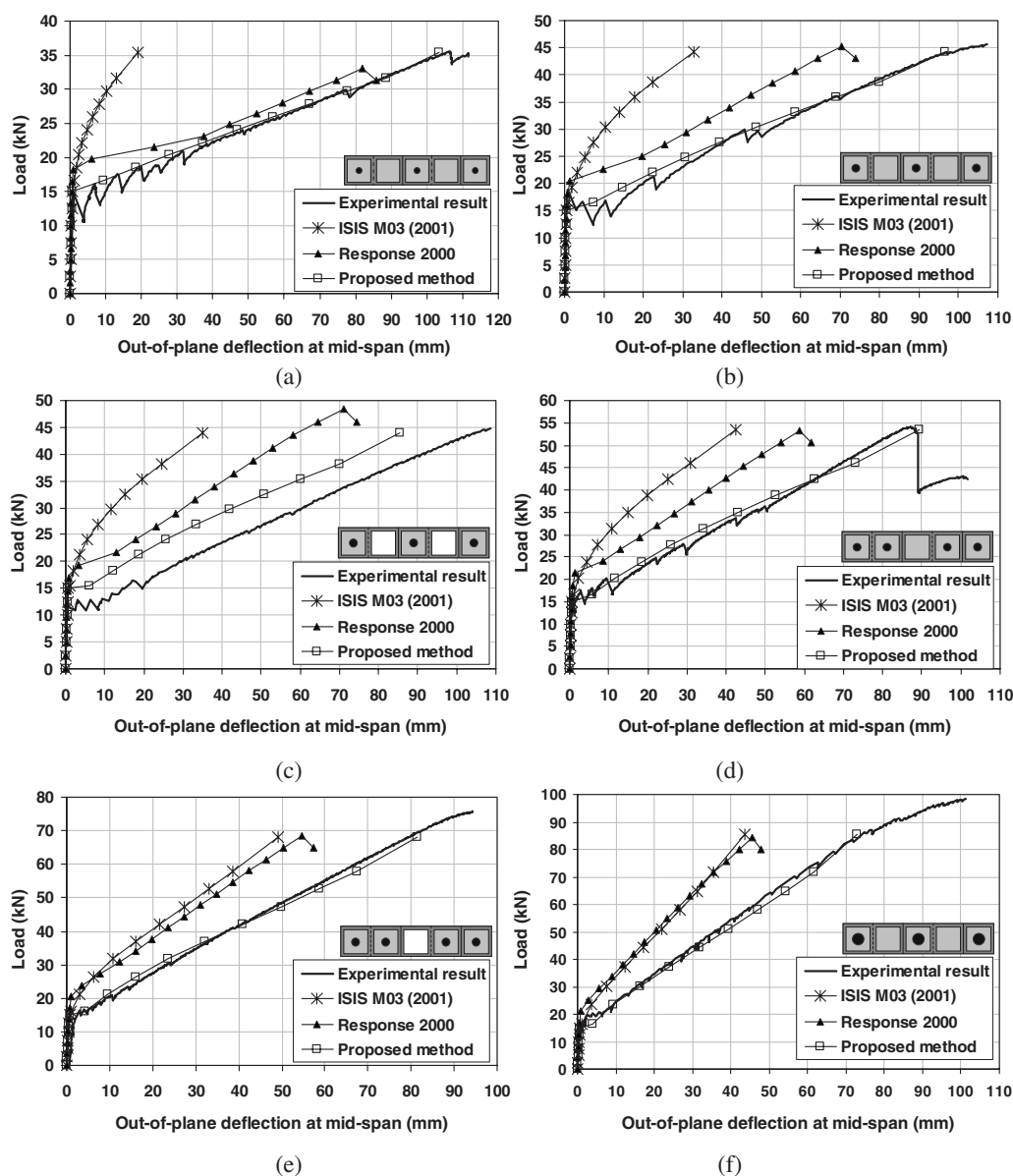


Fig. 6. Experimental and analytical load-deflection performances of the GFRP-reinforced masonry walls: (a) G-3#10-F; (b) G-3#13-F; (c) G-3#13-P; (d) G-4#13-F; (e) G-4#13-P; and (f) G-3#19-F

In addition to the numerical evaluation, the tested walls were modeled in Response-2000 Version 1.0.5 (Bentz 2000), which is a freely available specialty program for RC section analysis developed at the University of Toronto, based on the modified compression field theory. It has the ability to input the full axial stress-strain curve of the masonry (i.e., the refined model proposed by Dhanasekar and Shrive 2002) as well as the tensile properties of steel and GFRP reinforcements for modeling the wall sections. The load-deflection performances of the GFRP-reinforced walls predicted based on The Canadian Network of Centres of Excellence on Intelligent Sensing for Innovative Structures (ISIS Canada) (2001) as well as Response-2000 (Bentz 2000) are illustrated and compared with test results in Fig. 6. It can be observed that the load-deformation curves predicted by the program are a finer evaluation of the experimental results for the walls with lower reinforcement ratios, although neither the method recommended in The Canadian Network of Centres of Excellence on

Intelligent Sensing for Innovative Structures (ISIS Canada) (2001) nor the computer program is presenting an acceptable prediction of postcrack deflections.

Proposed Method

One of the potential reasons why the available methods underestimate the experimental deflection that was also observed in the course of the tests of the masonry walls can be the considerable width of the flexural cracks occurring at the mortar-to-block interface, which magnifies the deformed shape of the wall. In other words, the total displacement of the wall is also comprised by the displacement caused by the cracking pattern, which is not accounted for by the current methods of predicting the deflection. A simple approach is proposed based on the observations of the conducted experiments to improve the predictions of load-deformation behavior of masonry walls reinforced with GFRP by

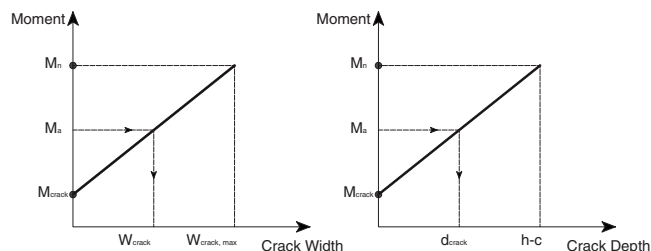


Fig. 7. Assumed relationships between the width and depth of the cracks at each postcrack stage of loading

adding the effect of flexural cracks to the deflections predicted by cracked-section analysis [introduced in The Canadian Network of Centres of Excellence on Intelligent Sensing for Innovative Structures (ISIS Canada) (2001)]. The width of the cracks can be calculated similar to FRP-RC as stated in The Canadian Network of Centres of Excellence on Intelligent Sensing for Innovative Structures (ISIS Canada) (2001) (units are N and mm)

$$w_{\text{crack}} = 11 \times 10^{-6} \frac{E_{\text{steel}}}{E_{\text{FRP}}} k_b f_{\text{FRP}} \frac{h-c}{d-c} (d_c A)^{1/3} \quad (6)$$

In this equation, h =thickness of the cross section of the wall (i.e., 190 mm); d_c =cover for the reinforcing rods; A =effective tension area surrounding the reinforcing rods divided by the number of the rods; and k_b =bond dependent coefficient that is found by Gao et al. (1998) to be 0.71, 1.00, and 1.83 for GFRP bars with superior, similar, and inferior bond properties when compared to steel [The Canadian Network of Centres of Excellence on Intelligent Sensing for Innovative Structures (ISIS Canada) 2001]. k_b equal to 0.71 is employed in the calculation of crack width for the GFRP-reinforced walls in this research for two reasons; first, it was observed in the course of the experiments that the GFRP-reinforced walls experienced less wide cracks than the steel-reinforced wall at the same level of loading, and last, Ahmed et al. (2006), who conducted an experimental investigation on a total 90 sand-coated GFRP V-ROD specimens (i.e., the type that is used in the experimental program of this research) with respect to their bond behavior, concluded the superior bond properties of the sand-coated V-ROD when compared to steel rebars.

The additional deformation due to the flexural crack width is then calculated based on the rotation of the wall at the cracked bed joints assuming that (1) the cracks are imposed to occur at the interface of blocks and mortar; (2) the width and depth of the cracks in all the joints inside the constant moment zone are the same; (3) the effect of cracks in the shear zones are neglected; (4) the crack width and depth increase linearly from zero, at the stage of cracking moment, to the maximum value at failure (see Fig. 7); and (5) the maximum depth of the crack is the depth of the tension zone (i.e., $h-c$). Knowing the width and depth of the crack, the rotation due to each crack is calculated at each loading stage as such

$$\theta = k_{\text{crack}} \frac{w_{\text{crack}}}{d_{\text{crack}}} \quad (7)$$

w_{crack} and d_{crack} =width and depth of the crack and k_{crack} =ratio of the depth of the crack to the depth of the cross section, which represents the resistance of uncracked zone. The deflection due to θ is the moment at midspan of the conjugate structure of the wall, when a point load equal to θ is applied at the location of the crack. Therefore, as illustrated in Fig. 8, the total deflection of the

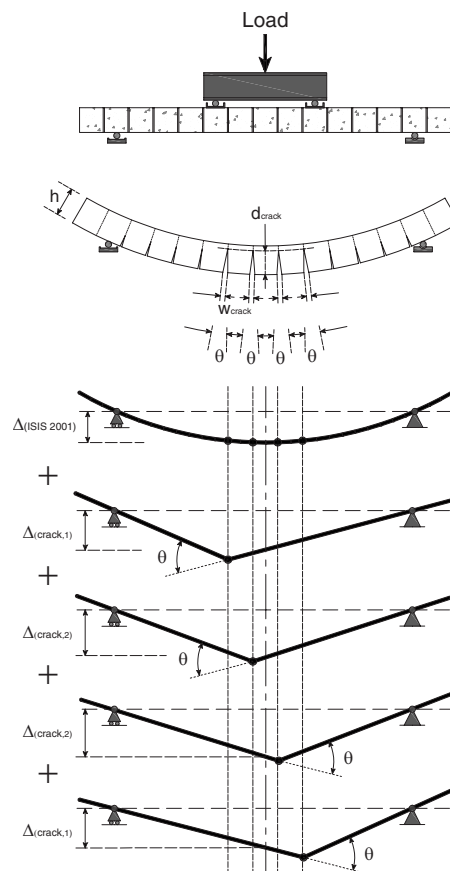


Fig. 8. Proposed method for predicting deflection considering the deflection due to excessive flexural cracking

wall at each postcrack stage of the loading is the superposition of the deflection found by the cracked-section analysis and the extra deflection due to the excessive width of the cracks in the maximum moment zone. Fig. 6 shows that the load-deformation performance of the GFRP-reinforced walls predicted by the method introduced herein shows agreeable consistency with the experimental result.

Ductility

When steel is substituted with FRP whose behavior is linearly elastic up to the sudden rupture, the ductility of the walls arises as a concern affecting their performance before failure. For steel-reinforced elements, conventional definition of deflection ductility as the ultimate to yield deformation ratio ($\mu_{\Delta} = \Delta_u / \Delta_y$) determines whether or not the members have sufficient ductility; however, for FRP-reinforced elements ductility should be defined apparently in a fashion that is independent of reinforcement yield point. To evaluate the ductility of GFRP-reinforced walls, two different approaches have been used based on different aspects of the load-deformation performance of the walls.

One method is to calculate the deformability factor or J -factor proposed by Jaeger et al. (1995), which is modified to be introduced as a criterion for FRP-RC members in The Canadian Network of Centres of Excellence on Intelligent Sensing for Innovative Structures (ISIS Canada) (2001), and calculated based on the curvature of the walls at ultimate and service load as such

Table 7. Classification of the Tested Walls according to Their Deformability

Wall	Curvature factor	Moment factor	Deformability factor	$E_{inelastic}/E_{total}$	Type of behavior
G-3#10-F	13.7	3.3	45.5	—	Ductile
G-3#13-F	6.2	3.7	22.9	0.91	Ductile
G-3#13-P	5.8	2.7	15.8	0.81	Ductile
G-4#13-F	14.8	2.0	29.8	—	Ductile
G-4#13-P	8.9	3.8	33.5	—	Ductile
G-3#19-F	7.4	4.1	30.6	0.76	Ductile

$$\text{deformability factor} = \left(\frac{\psi_u M_u}{\psi_s M_s} \right) \tag{8}$$

where the indices s and u refer to the service and ultimate stages and ψ and M are, respectively, the curvature and moment at each stage of loading. The service stage was primarily designated by Jaeger et al. (1995) to be the load at which point the compressive strain in concrete is 0.001. However, The Canadian Network of Centres of Excellence on Intelligent Sensing for Innovative Structures (ISIS Canada) (2001) associates it with a load due to which the tensile strain in FRP reaches 0.002. The curvature therein is calculated based on the experimental deformed shapes of the walls. In other words, the maximum absolute value of the second derivative of the deformed shape with respect to the length of the wall at each stage was considered as the curvature. This factor, which is required by The Canadian Network of Centres of Excellence on Intelligent Sensing for Innovative Structures (ISIS Canada) (2001) to be more than 4 for all sections in flexure, is calculated for the GFRP-reinforced walls to investigate their deformability (see Table 7).

The second method that was used for evaluating the ductility of the GFRP-reinforced walls is based on the absorbed energy. Having the load-displacement diagram, the total absorbed energy by the wall can be calculated as the area under the curve. The permanent (i.e., inelastic) deformation and the unloading curve differentiate the elastic and inelastic energies. That is to say, the dissipated energy by permanent deformations is represented by the inelastic portion of the area (i.e., the area surrounded between the loading and unloading curves). Given the fact that the more inelastic energy, the more ductile behavior, the ductility of the walls can be evaluated based on the ratio of the inelastic energy to the total energy (energy ratio). Jeong (1994) expressed the ductility of FRP-RC members, disregarding the existence of yielding phenomenon, with a new ductility index (the Naaman index) using the energy ratio

$$\mu = \frac{1}{2} \left(\frac{E_{total}}{E_{elastic}} + 1 \right) \tag{9}$$

where μ =Naaman index and E_{total} and $E_{elastic}$ are, respectively, the total and elastic absorbed energies by the wall. A ductility index of 2.5, which has been adopted in structural codes, as the accepted level of ductility for steel-reinforced members, was considered to be adequate by Jeong (1994), since this new index was derived from the conventional definition of deflection ductility. Grace et al. (1998) also categorized the flexural members with energy ratio of 75% and higher to have ductile behaviors. This energy ratio results in a ductility index of 2.5 and higher which was set to be the limit by Jeong (1994). Three of the GFRP-reinforced walls were unloaded after the ultimate failure, for which the energy ratio can be found based on the experimental results (see Fig. 9). Despite the fact that the use of GFRP instead of steel diminishes the ductile behavior of the masonry walls, it

can be observed that all the GFRP-reinforced walls have exhibited a ductile failure (see Table 7).

Capacity Diagram

One of the main objectives of this program was to achieve a facilitated approach to design the masonry walls for out-of-plane bending when reinforced with GFRP rebars. For designing FRP-RC members in flexure, design charts are provided by The Canadian Network of Centres of Excellence on Intelligent Sensing for Innovative Structures (ISIS Canada) (2001) for given cross-sectional dimensions. The reinforcement ratio required for a certain amount of resistant moment can be chosen simply by following the proper curves of the charts. Having the results of the tested walls (i.e., the nominal flexural capacity and ultimate FRP tensile strain at failure) for different reinforcement ratios, a similar attempt has been made to propose a design diagram illustrated in Fig. 10. The theoretical predictions are based on f'_m equal to 10.9 MPa and corresponding properties for different diameters of GFRP rods. However, to simplify the use of this type of diagrams, they can be achieved eventually based on the average properties of the composites regardless of their slight variations due to the chosen diameters in design. The results of the tested walls seem to have consistency with the predicted values, that is to say, the methods of analysis presented herein can be relied on to design this type of structural members. However, the authors believe that further studies and experiments on FRP-reinforced masonry are required to validate and generalize the outcomes of this study.

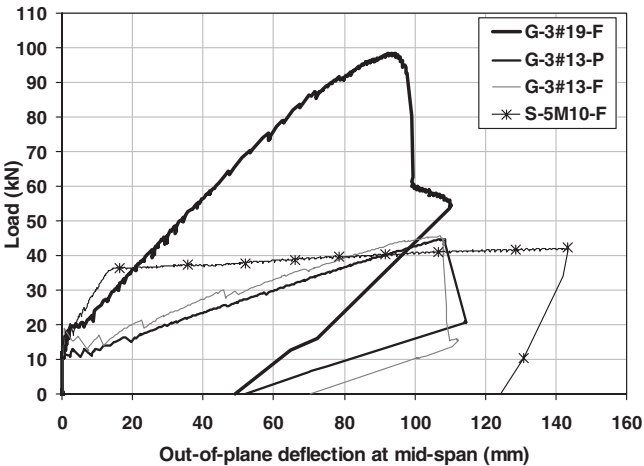


Fig. 9. Categorizing the behavior of the walls in terms of deformability based on the energy ratio using the unloading part of the experimental load-deflection curves

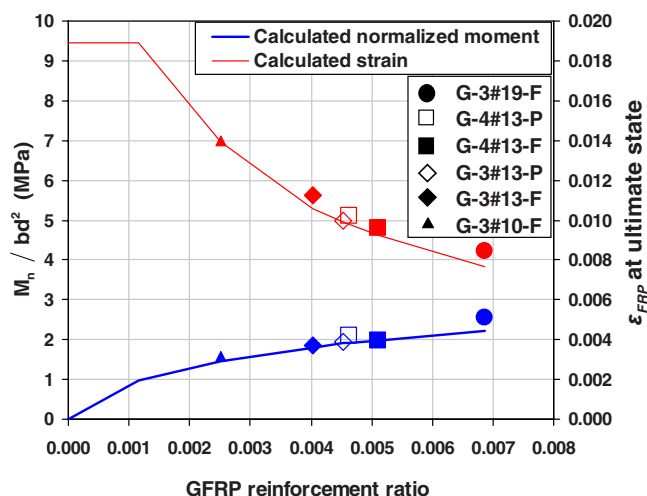


Fig. 10. Proposed capacity chart for designing masonry walls reinforced with GFRP rods ($f'_m = 10.9$ MPa; $f_{u,FRP(ave.)} = 790$ MPa; $E_{FRP(ave.)} = 46.4$ GPa)

Conclusions

Eight full-scale masonry walls (unreinforced and reinforced with steel and GFRP rebars) were built and tested under out-of-plane bending condition. The behavior of the panels was surveyed by monitoring the crack patterns and measuring the midspan deflections and tensile strains of the reinforcing rebars up to the ultimate failure. Based on the results of the main and auxiliary tests and analysis performed on different aspects of behaviors of the walls, the following remarks can be concluded:

1. Although all practical attempts were made during the construction of the tested masonry walls to position the GFRP rebars exactly in the midcells as per the design, it was found that this was not always achievable. This could be attributed mainly to the relative flexibility of GFRP rods compared to steel rebars. Further studies could investigate practical measures, procedures, devices, etc., to overcome such limitation;
2. The partially grouted walls cracked at fairly lower levels of applied load when compared to their analogous companions (i.e., fully grouted). However, the type of the reinforcement (i.e., steel or GFRP) was noted to have no effect on either the onset of cracking or the immediate width of the cracks;
3. It was observed that for all the tested walls, the compression failure at the constant moment region occurred in the section of the bed joint that has shear reinforcement. This would imply that shear reinforcement could result in weakening of the bond in the block-mortar interface;
4. It was observed that after the first crack, the tensile strain and, accordingly, the tensile stress of GFRP rods increases linearly with the applied load up to the failure of the walls;
5. The GFRP-reinforced masonry walls exhibited a linear behavior up to and after cracking moment, but the stiffness was decreased significantly after the first crack. It was also observed that the stiffness of the GFRP-reinforced walls increased with higher reinforcement ratios;
6. The wall G-3#10-F that was designed to achieve ultimate limit flexural capacity equivalent to the steel-reinforced wall showed sufficient flexural deformability to forewarn the failure;
7. The performance of G-3#19-F compared to that of S-5M10-F shows that we can reach higher capacities with acceptable

deformability; this could not be achieved by steel-reinforced masonry walls due to the constraint of not exceeding the balanced reinforcement ratio;

8. Despite the fact that the use of GFRP instead of steel diminishes the ductile behavior of the masonry walls, it was observed that all the GFRP-reinforced walls have exhibited sufficient deformability;
9. The method of calculating the deflection that is proposed in this research provides a considerably better interpretation and more precise prediction of the force-deformation behavior of the tested GFRP-reinforced masonry walls; and
10. The stress-strain model proposed by Dhanasekar and Shrive (2002) showed good agreement with the experimental results, which suggests that it could be used in developing capacity charts for designing masonry walls reinforced with GFRP.

It should be noted that the aforementioned conclusions are based on the limited experimental work described herein, which is restricted to the behavior of GFRP-reinforced masonry walls subjected to pure out-of-plane bending. The authors believe that further studies and experiments on axial and beam-column behavior of the FRP-reinforced masonry walls and, in general, FRP-reinforced masonry should be conducted to validate and generalize the findings of this experimental program.

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Notation

The following symbols are used in this paper:

- A = effective tension area surrounding the reinforcing rods divided by the number of the bars (mm^2);
- A_{FRP} = total area of the GFRP reinforcements (mm^2);
- A_s = total area of steel reinforcements (mm^2);
- c = depth of the compression zone in the cross section of the walls (mm);
- d = effective depth of reinforcing rebars (mm);
- d_c = concrete cover for the reinforcing bars (mm);
- d_{crack} = depth of the crack (mm);
- E = Young's modulus of masonry assemblage (GPa);
- $E_{elastic}$ = elastic energy absorbed by the masonry wall;
- E_{FRP} = Young's modulus of GFRP rods (GPa);
- $E_{inelastic}$ = inelastic energy absorbed by the masonry wall;
- E_{steel} = Young's modulus of steel rebars (GPa);
- E_{total} = total energy absorbed by the masonry wall;
- f_{FRP} = stress in the GFRP rods (MPa);
- f'_m = compressive strength of the masonry assemblage (MPa);

$f_{u,FRP}$ = ultimate tensile strength of the GFRP rods (MPa);
 f_y = yield stress of the steel rebars (MPa);
 h = thickness of the wall (i.e., depth of the cross section) (mm);
 I_{cr} = moment of inertia of fully cracked section (mm⁴);
 I_{eff} = effective moment of inertia of the cracked section (mm⁴);
 I_g = moment of inertia of the gross section (mm⁴);
 K_b = coefficient dependent on the bond properties of FRP;
 k_{crack} = ratio of the depth of the crack to the maximum depth of the crack;
 M_a = applied moment of the masonry walls (kN m);
 M_{cr} = cracking moment of the masonry walls (kN m);
 M_n = Nominal flexural resistance of the masonry walls (kN m);
 M_s = applied moment at the service stage (kN m);
 R = modulus of rupture of the masonry assemblage (MPa);
 u_0, u_1 = constants used in the stress-strain equation of the masonry;
 w_{crack} = width of the crack (mm);
 x = ratio of strain to strain corresponding to the peak compressive stress in masonry;
 α_1, β_1 = stress block factors;
 β_b = coefficient dependent on the properties of FRP;
 $\epsilon_{m,u}$ = ultimate compressive strain of masonry;
 θ = rotation of the wall at the cracked bed joint;
 μ = deformability index;
 ρ_{FRP} = GFRP reinforcement ratio;
 σ_m = compressive stress in masonry;
 ψ_s = curvature of the masonry walls at service stage; and
 ψ_u = curvature of the masonry walls at ultimate stage.

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