

Flexural Behavior of GFRP-Reinforced Concrete Masonry Beams

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Abstract: An experimental and analytical study is conducted in order to investigate the flexural behavior of masonry beams that are internally reinforced using glass fiber-reinforced polymers (GFRP) rebars. Seven reinforced masonry beams with 4.0- and 2.4-m spans were tested under four-point bending setup. The beams were loaded monotonically up to failure. One had two courses of hollow concrete masonry units and the remaining six beams had three courses. Two masonry beams were reinforced using conventional steel rebars and were considered as the control specimens. The remaining five beams were internally reinforced using GFRP rods with different reinforcement ratios. Beams were detailed to have sufficient shear reinforcement such that they do not fail in shear. Flexural capacity, deformation, curvature, and strains of the tested GFRP-reinforced and steel-reinforced masonry beams were compared and discussed. Using the acquired data from the experimental and analytical studies, effectiveness of GFRP rods as internal reinforcement for concrete masonry beams is demonstrated.

DOI: 10.1061/(ASCE)CC.1943-5614.0000148

CE Database subject headings: Fiber reinforced polymer; Reinforcement; Masonry; Concrete beams; Flexural strength.

Author keywords: FRP reinforcement; Reinforced masonry; Beams; Flexure.

Introduction

Masonry is one of the ancient construction methods and has been used by number of civilizations for centuries. Beams in masonry construction are mostly used as bond beams or lintel beams. They are located at the roof level or at floor levels and may have multiple functions such as tying the structure around its perimeter, transferring the diaphragm action of the roof to the shear walls, and spanning over the openings in the walls supporting the gravity loads coming from above.

Current limit states design (LSD) codes for reinforced masonry structures, such as the Canadian Standards Association CSA S304.1-04 (2004a), enforce the use of reinforced masonry beams even for short spans. Although reinforced masonry flexural members exhibit similar behavior compared to reinforced concrete members, design difficulties may be encountered with conventional steel bars due to the limited width of the masonry units. Considering the underreinforced design principle of the steel-reinforced masonry, design of beams with deep cross sections is inevitable for most of the cases, giving rise to higher material and labor costs, as well as an increase of the floor height. In addition to that, conventional steel reinforcement is prone to degradation due to corrosion which is a well-known phenomenon.

Concrete elements reinforced with fiber-reinforced polymer

(FRP) rods have been extensively examined by researchers in the past decade and has been identified as an alternate construction material for reinforcing and strengthening concrete primarily due to its promising strength and durability characteristics. FRP comes with several significant advantages over steel, such as high durability, noncorrosiveness, high strength-to-weight ratio, and resistance against fatigue. On the other hand, unlike conventional steel reinforcement, FRP uses overreinforced design approach in that there will not be any upper limit for reinforcement ratio as long as the deflections are within the allowable limits set by, for example, the design guidelines of ISIS (2001) and ACI 440.1R-06 (2006). In this sense, it is suitable to say that one could explore the benefits of the overreinforced design concept of FRP-reinforced concrete-based elements, beside other advantageous characteristics of FRP rebars compared to steel ones, toward an efficient and durable structural element.

To the writers' knowledge, applications of FRP as an internal reinforcement for reinforced masonry beams have not yet been investigated. Although design guidelines such as ISIS (2001) and ACI 440.1R-06 (2006) for the applications of FRP in reinforced concrete are well established, a design guideline addressing the applications of FRP in the design of reinforced masonry elements is not available in the literature.

Previous Studies

Tests on conventional steel-reinforced masonry assemblages have been conducted for decades and consequently masonry design guidelines, such as the CSA S304.1-04 (2004a), were developed. Khalaf (1981), as well as Suter and Fenton (1986), conducted experimental investigations on many steel-reinforced concrete and brick masonry beams with different percentages of reinforcement and different configurations of grouting. These researchers reported that the LSD approach of reinforced concrete can be safely used for reinforced masonry beams by assuming the maxi-

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Note. This manuscript was submitted on November 8, 2009; approved on June 7, 2010; published online on June 10, 2010. Discussion period open until July 1, 2011; separate discussions must be submitted for individual papers. This paper is part of the *Journal of Composites for Construction*, Vol. 15, No. 1, February 1, 2011. ©ASCE, ISSN 1090-0268/2011/1-21-31/\$25.00.

Table 1. Properties of the GFRP Rods Used in the Experimental Program (Adapted from Technical Datasheet of V-ROD, Produced by Pultrall Inc. 2007)

Metric size	Nominal diameter (mm)	Area (mm ²)	E_{frp} (GPa)	f_{frpu} (MPa)	ε_{frpu} ($\times 10^{-6}$)
13	12.70	126.7	46.3	786	17,000
16	15.88	197.9	48.2	751	15,600
19	19.05	285.0	47.6	728	15,300

mum usable compressive strain as 0.003 and by designating masonry compressive strength (f'_m) instead of f'_c . They proposed a representative value of f'_m and modulus of elasticity (E_m) for concrete masonry units with a correct method of prisms testing and concluded by saying that the flexural behavior of the beams was very similar to those of reinforced concrete.

There is plenty of literature available related to the use of FRP as an internal reinforcement method and external strengthening method for reinforced concrete structural members and both methods were found to be effective and practical. Theriault and Benmokrane (1998), Abdalla (2002), Ashour (2006), and Saikia et al. (2007) carried out monotonic and cyclic tests on FRP-reinforced concrete beams and investigated the effect of FRP reinforcement ratio and strength of concrete on the flexural resistance of the specimens as well as the crack patterns, deflection, and deformability characteristic of the GFRP-reinforced concrete beams. The researchers concluded that the ultimate moment capacity of the beams increase as the reinforcement ratio increases and tested beams were considered to be safe in terms of deformability. On the other hand, deflections predicted by ISIS (2001) were in good agreement with the experimental results.

Previous studies conducted on applications of FRP in reinforced concrete masonry by Kiss et al. (2002), Ghobarah and Galal (2004), Hao (2007), and Moon et al. (2007) focused on developing FRP-strengthening techniques for conventional steel-reinforced structural masonry members. Researchers showed that the flexural capacities and the ductility of specimens increased noticeably after strengthening. Even though researchers have started conducting tests on FRP-reinforced concrete beams during the past decade, literature regarding internally FRP-reinforced masonry assemblages is very limited. Recently, Galal and Sasanian (2009) investigated the out-of-plane flexural performance of glass fiber-reinforced polymer (GFRP) reinforced concrete masonry walls. Eight full-scale walls were constructed using internally GFRP reinforced, either partially or fully grouted concrete masonry blocks. It was concluded that higher flexural capacities, compared to steel-reinforced masonry walls, can be achieved with acceptable deformability. The GFRP-reinforced masonry walls showed sufficient flexural forewarning deformability prior failure.

Significance and Parameters of Study

This paper is primarily targeting to examine the flexural response of fully grouted and internally GFRP-reinforced full-scale concrete masonry beams. For that purpose, five GFRP-reinforced concrete masonry beams were prepared having different reinforcement ratios. In addition to that, two other fully grouted steel-reinforced full-scale beams were built and tested as control specimens. The scope of this paper is to investigate the flexural performance of internally GFRP-reinforced fully grouted concrete masonry beams toward developing capacity design diagram and

to investigate the curvature and deformability characteristics of the GFRP-reinforced masonry beams. The main parameter of this research is the GFRP reinforcement ratio that differs among the GFRP-reinforced specimens.

This research work investigates the flexural performances of the GFRP-reinforced masonry beams, meaning that the beams were detailed to have adequate shear resistance such that they were forced to fail in flexure. In addition to that, it is assumed that GFRP rods maintained perfect bonding with the surrounding grout filling during the tests.

Experimental Program

Material Properties

Hollow rectangular masonry blocks with dimensions $190 \times 190 \times 390$ mm, square blocks with dimensions $190 \times 190 \times 190$ mm and lintel blocks with dimensions $190 \times 190 \times 390$ mm were used for the construction of the specimens. Compressive resistance of concrete masonry blocks were reported as 15 MPa by the manufacturer's technical datasheet (Simard-Beaudry 2001).

During construction, general purpose mortar mix that is classified as "Type S" mortar and "coarse" grout was used in accordance with the masonry standards ASTM C476 (ASTM 2002a), ASTM C270 (ASTM 2002b), and CSA A179-04 (2004b). Ten mortar cubes with dimensions 50×50 mm were prepared in order to be tested 7 and 28 days after construction. Following average compressive resistances of the mortar cubes were found to be 7.9 and 14.5 MPa, respectively. In addition to that, five cylinders filled with the grout mix were prepared in order to be tested 28 days after construction. Average compressive resistances of the grout cylinders were found to be 14.2 MPa. Average compressive strength of the tested mortar cubes and grout cylinders were within the allowable limits set by the above mentioned masonry standards.

Steel reinforcement used for the tested control beams was conventional 15M rebar with 492 MPa yield strength and 698 MPa ultimate strength. Both ends of the steel rebars had 90° hooks.

GFRP rods, also known as "V-ROD," that were used in this project were provided by Pultrall Inc. of Québec (Pultrall Inc. 2007). They are manufactured by pultrusion procedure of continuous glass fibers and thermosetting polyester resin. The surface of the rod is wrapped by glass fiber strings and sand coated in order to improve its bond characteristics. In order to have a variety of GFRP reinforcement percentages, different diameters and numbers of rods were used as an internal reinforcement for the specimens. Table 1 provides detailed information on the strength characteristics of the rods.

Full-scale masonry beams were constructed on previously prepared wooden platforms to avoid mobility problems while transferring the reinforced masonry beams to the test setup. On the other hand, beams had the risk of achieving their cracking moments (M_{cr}) due to their own weight as they were being transferred to the test setup. For that reason, they had to be carried from underneath to the test area with the help of a crane.

While one of the conventional steel-reinforced beams had internal steel stirrups, for the rest of the beams, external shear reinforcement was used instead of internal stirrups to ensure sufficient shear capacity in shear zone in order to obtain flexural failure. Application of exterior shear clamps is a common technique that is being used to strengthen beams and previous researches conducted on shear clamps by Kim and White (1999),

Table 2. Tested Full-Scale Reinforced Concrete Masonry Beams

Beam	Reinforcing material	Courses	Exterior shear reinforcement	Span (mm)	$A_{\text{reinf.}}/A_g$ (%)
S-3-1-15M-I	Steel	3	No	4,000	0.20
S-3-1-15M	Steel	3	Yes	4,000	0.20
F-3-1#13	GFRP	3	Yes	4,000	0.13
F-3-1#19	GFRP	3	Yes	4,000	0.29
F-3-2#16	GFRP	3	Yes	4,000	0.40
F-2-2#16	GFRP	2	Yes	2,400	0.65
F-3-2#19 and 1#16	GFRP	3	Yes	4,000	0.78

Altin et al. (2003), and Anil (2008) showed similarity between the strengthened beams and internally shear-reinforced control specimens in terms of rigidity and crack patterns. The steel clamps used for this experiment were prepared under laboratory conditions and were made of two square hollow steel sections, two 10-mm diameter threaded bars, eight high-strength nuts, and four washers.

Test Specimens

All the beams and auxiliary specimens were constructed by experienced masons certified in province of Québec, under the surveillance of the writers and their quality of workmanship was judged to be very good. In order to determine the compressive strength of grouted masonry assemblages normal to the bed joint (f'_m), five one-block wide and five-block high unreinforced grouted masonry prisms were constructed following the guidelines of ASTM C1314 (ASTM 2002c).

While reinforced masonry columns rely on the block's compressive strength normal to the bed joint, beams are influenced by the block's compressive strength parallel to the bed joint due to the compression zone located at the top course of the beam. For that purpose, CSA S304.1-04 (2004a) provided correction factor χ for f'_m to be used for reinforced masonry beam design. The correction factor χ can range from 0.5 to 0.7 depending on the conditions as stated by CSA S304.1-04 (2004a). In order to determine the actual compressive strength parallel to the bed joint and compare it with the approximate value suggested by the design guideline, four other auxiliary specimens were constructed having identical dimensions with the previous specimens.

In order to determine the cracking moment (M_{cr}) of the reinforced masonry assemblages, flexural tensile strength normal to the bed joints of the grouted hollow concrete masonry blocks has to be determined. For this purpose, following the guidelines of ASTM E518 (ASTM 2002d), five seven-blocks high and one-block wide unreinforced grouted masonry prisms were constructed and tested under four point bending test setup.

In total, seven reinforced fully grouted concrete masonry beams with 4.0 and 2.4 m spans (center to center of supports) were constructed. One beam had two courses of hollow concrete masonry units and the remaining six beams had three courses. Two masonry beams were reinforced using conventional steel rebars to be used as control specimens. The other five beams were internally reinforced using GFRP rods with different reinforcement ratios. All the beams were designed to have 400-mm additional length beyond the support points at both ends in order to provide sufficient development length and anchorage for the internal reinforcement. The internal conventional steel reinforcement had 90° hooks at their ends to avoid slippage. Although GFRP rods of the three-course beams had sufficient develop-

ment length as required by ISIS (2001), an increase in anchorage was required for the short-spanned two-course GFRP-reinforced masonry beam. As such, two L-shaped GFRP rods with the same diameters were spliced with the main internal GFRP reinforcement at both ends. Beams were detailed to have sufficient shear reinforcement such that they do not fail in shear. One of the conventional steel-reinforced beams had internal steel stirrups while for the rest of the beams, external shear clamps were used on their shear spans. These clamps had 400-mm spacing. Performance of the external shear clamps and their effects on sectional moment resistance, shear resistance, overall deflection, and stiffness characteristics of the full-scale specimens were monitored by comparing two conventional steel-reinforced concrete masonry beams. Thickness of the mortar joints were on average 10 mm.

Beams were designated with letters and numbers to indicate its internal reinforcement type (FRP or steel), height in terms of courses of units (2 or 3), type and amount of internal reinforcement (1#13, 1-15M). Thus, F-3-1#13 indicates internally single #13 GFRP rod reinforced three-course clamped beam. Table 2 lists all the tested full-scale beams in detail with corresponding designations. Fig. 1 demonstrates the schematics of cross sections.

Instrumentation and Test Setup

Prior to loading, beams were supported by two permanent supports (hinged and roller) located at the last vertical joint at both ends and one temporary support located at the midpoint. Temporary support was removed before each test as soon as the data acquisition system was turned on to monitor and record the initial strains and deflections due to self-weight before applying any load. All the beams were tested under four-point bending setup. The beams were subjected to an increasing monotonic load up to failure and then unloaded when possible to determine the permanent deflection and energy dissipation characteristics of the beams. All the beams had a pure flexure zone that was designed to be 800 mm in length, which was located between the supports of the loading apparatus.

Each interior reinforcing bar had five strain gauges installed prior to construction in order to record the strain response of the reinforcement at different load levels. In addition to that, five cable-operated potentiometers were used during the tests in order to be able to monitor the load-deflection relationship of each specimen. Fig. 2 is a schematic showing the locations of the strain gauges and potentiometers on the three-course concrete masonry beam.

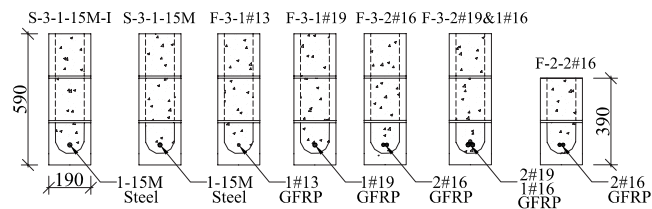


Fig. 1. Cross sections of the full-scale reinforced concrete masonry beams

Results and Discussion

Auxiliary Specimens

The mean compressive strength normal to the bed joint of the masonry prisms was found to be 13.66 MPa and the Young's modulus (E_m) was obtained from the slope of the stress-strain curves, using the section between 5 and 33% of the ultimate loads. The average E_m of the prisms was determined to be 7.5 GPa. Conical breaks and face shell splitting were among the fracture types experienced before failure. The mean value of the compressive strength of the masonry prisms loaded parallel to their bed joints was found to be 6.0 MPa which was found to be 15% less than the estimation of CSA S304.1-04 (2004a). The average flexural bond strength, also known as modulus of rupture (R) of the remaining prisms was 0.72 MPa. One of the specimens damaged during its transportation to the test setup therefore could not be tested.

Full-Scale Reinforced Concrete Masonry Beams

Self-weight of the fully grouted reinforced masonry beam was calculated and was corresponding to a distributed load of 2.6 kN/m in case of a three-course beam and 1.6 kN/m for a two-course beam. For a same moment at midspan section, these distributed loads are equivalent to 6.5- and 3-kN point loads, respectively, hence these values were added to the loading data of the corresponding beams.

Table 3 shows a summary of nominal flexural capacity and reinforcement strains of the seven tested reinforced masonry beams. The moment, strain, and deflection values mentioned in the following detailed test results of the beams are referring to the values recorded at the midspan, which is the primary point of interest of this study.

Table 3. Test Results of Full-Scale Reinforced Concrete Masonry Beams

Beam	Failure load (kN)	Nominal flexural capacity (kN·m)	Max. deflection (mm)	Max. tensile strain ($\times 10^{-6}$)	Type of failure
S-3-1-15M-I	67	54	46.7	10,100	Mode 1
S-3-1-15M	70	56	52.4	13,600	Mode 1
F-3-1#13	58	46	50.5	13,300	Mode 2
F-3-1#19	86	69	68.2	13,400	Mode 3
F-3-2#16	80	64	28.6	6,500	Mode 3
F-2-2#16	70	28	35.9	12,000	Mode 3
F-3-2#19 and 1#16	88	69	16.3	5,600	Mode 3

Note: Mode 1=yielding of longitudinal rebars followed by crushing of concrete masonry; Mode 2=rupture of GFRP rod; and Mode 3=crushing of concrete masonry.

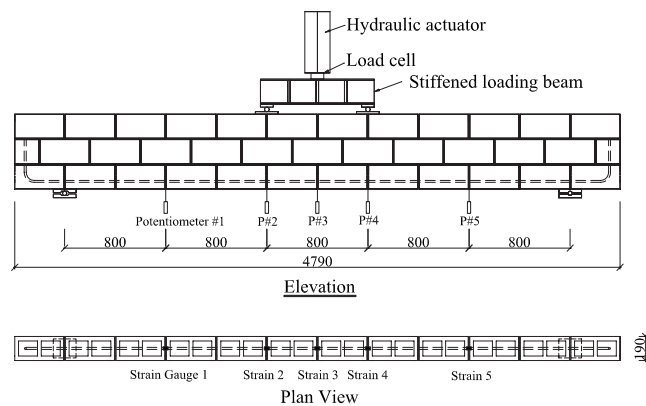


Fig. 2. Elevation of reinforced masonry specimen showing locations of strain gauges and potentiometers

Modes of Failure and Postfailure Behavior

Compression failure in the constant moment zone was observed in all the GFRP overreinforced specimens and their postfailure behaviors were similar. Compression failure started with crushing of the vertical mortar joint located between the masonry units on the top row at the midspan. As the load increased, horizontal cracks started to form on the face of the concrete masonry unit located at the top row of the midspan, right beneath the loading apparatus. As the loading continued and horizontal cracks widened, splitting of the block shells in the compression zone occurred and the flexural resistance of the specimen decreased significantly. GFRP underreinforced specimen F-3-1#13 failed due to the rupture in the GFRP rod as expected without experiencing compression failure in grout-filled masonry units. Conventional steel-reinforced, underreinforced concrete masonry beams S-3-1-15M-I and S-3-1-15M failed in compression after the yielding of steel reinforcement. All the failures were ductile since the internal steel reinforcement underwent gradual plastic deformation after yielding. Fig. 3 shows the tested reinforced masonry beams at failure.

Crack Patterns and Deformation

As expected, majority of the beams reached their cracking moments (M_{cr}) as soon as the loading process started. Those first cracks were not visible, however, they resulted in a significant drop in the stiffness. After the first crack, beams continued carry-

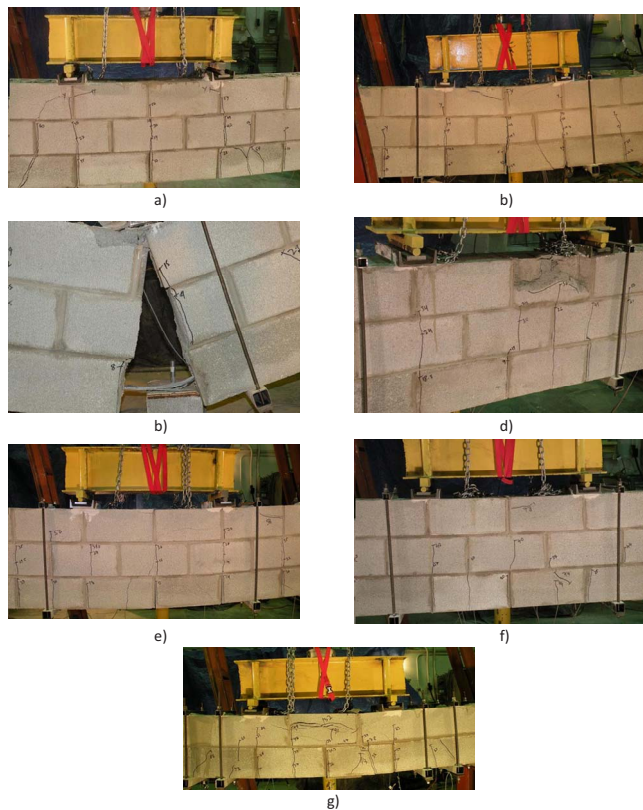


Fig. 3. Full-scale reinforced concrete masonry beams at failure: (a) S-3-1-15M-I; (b) S-3-1-15M; (c) F-3-1#13; (d) F-3-1#19; (e) F-3-2#16; (f) F-3-2#19 and 1#16; and (g) F-2-2#16

ing more load as the visible flexural cracks started to appear and mortar joints close to the soffit of the beam in the constant moment zone started splitting. As the load increased, cracks at the mortar joints close to the midspan began widening and propagated vertically toward the compression zone. Cracks did not necessarily follow the mortar joints and propagated through the masonry blocks as well. Inclined cracks in the shear span started to appear as well with the continuous increase in the applied load.

On the other hand, visual comparison of the tested externally shear reinforced and internally shear reinforced steel-reinforced masonry beams, S-3-1-15M and S-3-1-15M-I, respectively, showed that they had similar flexural and shear crack patterns, meaning that the clamps in the shear zone did not limit the formation of shear cracks but prevented them from widening. Clamps were deliberately placed before or after mortar joint in order to allow the joints separate freely during loading. In the case of steel-reinforced beams, flexural cracks widened significantly as soon as the steel reinforcement reached yielding load.

During the tests, separation of the joints in the constant moment zone was monitored and recorded at intervals of 10 kN. As soon as joints started separating, using a scaled transparency, images of the joints were taken. As much as 10 mm of separation at the mortar joints were observed.

Crack patterns of the tested internally GFRP-reinforced concrete masonry beams showed similarities with those of GFRP-reinforced concrete beams reported in the literature. Crack formation was initiated when cracking moment M_{cr} point was reached in the constant moment zone, after which, propagated

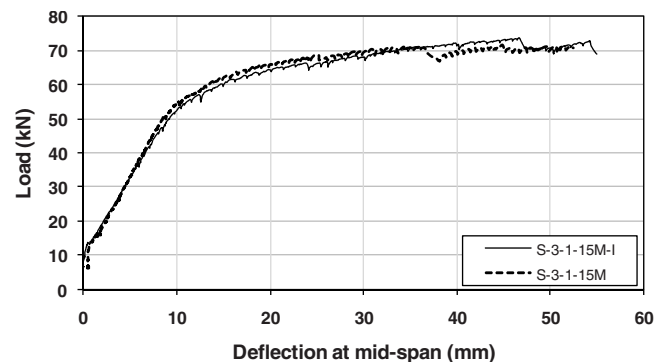


Fig. 4. Experimental load-deflection performance of the steel-reinforced concrete masonry control beams

vertically and widened as the load increased. Cracking in the shear zone started with vertical cracks whereas they started to incline at the midheight of the beam.

Load versus Deflection Relationship

In general, none of the beams experienced significant amount of deflection prior to reaching their cracking moments. Load-deflection curves for all the specimens were initially linear. In case of steel-reinforced beams S-3-1-15M and S-3-1-15M-I, load-deflection curves' slope started to decrease as the internal steel reinforcement yielded and had similar load-deflection relationship as can be observed at Fig. 4. All the other GFRP-reinforced masonry beams had close to linear load-deflection relationship up to their ultimate loads.

Compared to the other tested beams, GFRP underreinforced masonry beam F-3-1#13 experienced highest midspan deflection as expected at failure load of 58 kN. Other GFRP-reinforced masonry beams were all stiffer as the reinforcement ratios were increased. They were all overreinforced as required by ISIS (2001) in case of FRP-reinforced concrete beams. Beam F-3-2#19 and 1#16 had the highest GFRP reinforcement ratio among the other GFRP-reinforced three-course masonry beams and was significantly stiffer than the steel-reinforced shear clamped control beam S-3-1-15M. This shows that GFRP overreinforced masonry beams can provide high flexural resistances with reasonable deformability that would provide sufficient forewarning before failure.

As another comparison, the relative load-deflection performance of conventional underreinforced steel-reinforced beam S-3-1-15M and GFRP underreinforced beam F-3-1#13 were analyzed as both of the beams were designed to have comparable amount of internal reinforcement. It was concluded that although the reinforcement ratio of the GFRP-reinforced beam was 7% less than the compared steel-reinforced beam, it showed significant sectional moment resistance at 50 mm of ultimate midspan deflection. Beam F-3-1#13 experienced similar ultimate midspan deflection values compared to the steel-reinforced concrete masonry beam, forewarning failure prior to the rupture of the GFRP rod. Fig. 5 shows a comparison of experimental load-deflection curves of the GFRP-reinforced concrete masonry beams and conventional steel-reinforced concrete masonry beam.

Load versus Strain Relationship

As expected, conventional 492 MPa steel reinforcement of the steel-reinforced masonry beams S-3-1-15M and S-3-1-15M-I

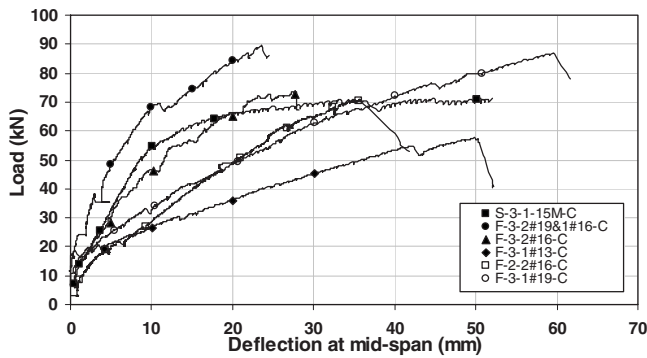


Fig. 5. Experimental load-deflection curves of the GFRP-reinforced concrete masonry beams and conventional steel-reinforced masonry beam

yielded at strains around 2,400 microstrains. Their load-strain relationships were satisfactory as well since the plotted curves were linear up to yield and exhibited strain hardening plateau as the beams were reaching their ultimate loads.

GFRP rods exhibited linear stress-strain relationship until flexural failure of the beams. Only the underreinforced GFRP beam F-3-1#13 reached its ultimate strain in tension and ruptured. The strains of the #13 GFRP rod was monitored up to 13,290 microstrains before the rupture occurred. Manufacturer reported the ultimate strain of that rod as 17,000 microstrains (Pultrall Inc. 2007). Since the rest of the specimens were all overreinforced, failures of the specimens were compression controlled and the rods stayed intact. Examinations performed on GFRP rods of the failed specimens showed that the sand coating on the surface of the bars were fully existent, showing no signs of slippage and proving that the provided anchorage was sufficient.

Analytical Predictions and Comparisons

Section Analysis

In order to determine the nominal moment resistances of the beams, the following assumptions and constant coefficients were used along with the available design guidelines, CSA S304.1-04 (2004a), ISIS (2001), and ACI 440.1R-06 (2006): (1) plane sections remain plane after bending; (2) GFRP rods exhibit linear elastic behavior up to the rupture stresses; (3) longitudinal reinforcement is fully bonded to the surrounding grout infill and slippage does not occur; (4) ultimate compressive strain of masonry is taken as $\epsilon_{mu}=0.003$; (5) modulus of elasticity of masonry is used as $E_m=850f'_m$; (6) χ factor is taken as 1.0 and sectional moment resistance predictions were carried out using different compressive strength (f'_m) values which are the corrected value suggested by CSA S304.1-04 (2004a) and the result of the auxiliary prism tests loaded parallel to their bed joints; and (7) depth of the reinforcement (d) varied with the type of longitudinal reinforcement used. For all cases, 50-mm cover was assumed. Computer software Response 2000, developed by Bentz (2000), was used to further support the applicability of analytical predictions that are stated in this paper. Response 2000 is a sectional analysis software that calculates strength, ductility, and stiffness of a given cross section under flexural, shear, and axial load. The software utilizes a modified compression field theory along with nonlinear analysis. The aforementioned assumptions were used as an input to the software to model the full-scale reinforced concrete ma-

sonry beams along with the material properties of grout-filled masonry, GFRP rods, and conventional steel rebars. These properties were either obtained from auxiliary test specimens or from the manufacturer's technical data sheets.

Flexural capacity of the steel-reinforced concrete masonry beam was predicted using equivalent rectangular stress block approach suggested by the CSA S304.1-04 (2004a) along with the α_1 and β_1 values suggested by CSA S304.1-04 (2004a) as 0.85 and 0.8, respectively. Primarily the area of steel of balanced condition was checked using

$$A_{sb} = \frac{\chi(\alpha_1 f'_m) b \beta_1 \left[\frac{600}{600 + f_y} \right] d}{f_y} \quad (1)$$

where b =width of the cross section and f_y =yield stress of the steel reinforcement. Consequently, once the depth of the stress block is determined using force equilibrium of the cross section, sectional nominal flexural capacity of the beam was determined using

$$M_r = \chi(0.85 f'_m) \beta_1 c b \left(d - \frac{\beta_1 c}{2} \right) \quad (2)$$

Design of GFRP-reinforced concrete masonry beams was carried out using equivalent rectangular stress block approach suggested by the ISIS (2001) for the design of FRP reinforced concrete beams. Compressive stress of concrete (f'_c) was substituted by compressive strength of masonry (f'_m). Nominal flexural capacity of the sections was calculated by using coefficients $\beta_1=0.95$ and $\alpha_1=0.84$ which are recommended by Dhanasekar and Shrive (2002) for masonry assemblages. Although underreinforced section is not recommended by ISIS (2001), design of an underreinforced beam was carried out primarily by determination of the balanced condition using

$$\rho_{frpb} = \frac{\alpha_1 \beta_1 f'_m}{f_{frpu}} \left[\frac{\epsilon_{mu}}{\epsilon_{mu} + \epsilon_{frpu}} \right] \quad (3)$$

where f_{frpu} =ultimate stress of the GFRP rods; ϵ_m =compressive strain of masonry; ϵ_{mu} =ultimate compressive strain of masonry; and ϵ_{frpu} =ultimate tensile strain in the GFRP rods. Next, depth of the compressive block, factors α_1 and β_1 were determined iteratively using charts provided in the design guideline ISIS (2001). Consequently the expected flexural capacity of the section is obtained using

$$M_r = A_{frpb} f_{frpu} \left(d - \frac{\beta_1 c}{2} \right) \quad (4)$$

where A_{frpb} =area of internal GFRP reinforcement. In case of overreinforced cross section, the following equation was used to determine the stress in the GFRP rod once the quantity of the rods were selected using the balanced section condition:

$$f_{frp} = 0.5 E_{frp} \epsilon_{mu} \left[\left(1 + \frac{4 \alpha_1 \beta_1 f'_m}{\rho_{frpb} E_{frp} \epsilon_{mu}} \right)^{1/2} - 1 \right] \quad (5)$$

where E_{frp} =modulus of GFRP rods. Consequently, once the depth of the stress block is determined using force equilibrium of the cross section, sectional flexural resistance of the beam was determined substituting f_{frp} to f_{frpu} in Eq. (4). Findings were later compared with the test results. Flexural resistance of the section was calculated using two different f'_m ; the corrected value suggested by CSA S304.1-04 (2004a) and the average compressive resis-

Table 4. Predicted Nominal Flexural Capacity of the GFRP-Reinforced and Conventional Steel-Reinforced Concrete Masonry Beam Sections

Beam	M_n experimental (kN·m)	CSA ^a (kN·m)	ISIS ^a (kN·m)	ISIS ^b (kN·m)	Response 2000 ^a (kN·m)
S-3-1-15M	53.6	49.2	—	—	52.2
F-3-1#13	46.4	47.3	46.8	46.4	30.4
F-3-1#19	69.3	55.3	59.2	49.1	54.8
F-3-2#16	63.6	62.7	67.1	55.4	68.2
F-2-2#16	28.3	28.1	30.4	24.6	31.7
F-3-2#19 and 1#16	69.4	78.1	83.5	68.4	79.4

^a $f'_m = 6.9$ MPa [CSA S304.1-04 (2004a) corrected compressive strength of masonry assemblage loaded parallel to its bed joint].

^b $f'_m = 5.9$ MPa (actual compressive strength of masonry assemblage loaded parallel to its bed joint, obtained from prism tests).

tance obtained by testing prisms loaded parallel to their bed joints.

CSA S304.1-04 (2004a) was able to predict the nominal flexural resistance of the steel-reinforced control beam S-3-1-15M with 8% error which is seen to be reasonable. Using different f'_m values did not result with a noticeable change in the sectional resistance. Computer software Response 2000 estimated the resistance of the section correctly as well, having an error of 3%. For steel-reinforced beam, f'_m , taken as 6.9 MPa, showed better accordance with the test results.

ISIS (2001) was able to predict the flexural resistances of the majority of GFRP-reinforced specimens with good accuracy where corrected f'_m value of CSA S304.1-04 (2004a), 6.9 MPa, showed better accordance with the test results. While beam F-3-1#19 exceeded its expected flexural capacity by 15%, beam F-3-2#19 and 1#16, having the highest reinforcement ratio among the other GFRP-reinforced beams, was able to reach 83% of its expected flexural capacity. These differences can be justified by considering the fact that the compressive strength of the masonry assemblage can easily be influenced by the quality of the labor and materials.

On the other hand, Response 2000 estimated the resistances of beams F-3-1#19 and F-3-2#19 and 1#16 close to the predictions of ISIS method. Predicted nominal flexural capacities of the sections using available methods are tabulated on Table 4.

Curvature Prediction

Curvature of the tested specimens at different bending moment levels were compared in order to better assess and compare the deformability characteristics of GFRP-reinforced concrete masonry beams.

Using different methods of I_{eff} predictions provided by ISIS (2001) and CSA S304.1-04 (2004a), curvature characteristics of the beams were evaluated

$$\psi = \frac{M}{E_m I_{eff}} \quad (6)$$

Moreover, the strains of the GFRP rods obtained from the recorded test data were used in a numerical evaluation to mathematically derive the curvature of the cross section. Curvature was determined through numerical integration based on the actual stress-strain relationship of masonry assemblage proposed by Dhanasekar and Shrive (2002). During the process, some iterative solutions were needed. Experimental curvature was obtained using the following relationship:

$$\varphi = \frac{1}{\text{radius}} = \frac{\varepsilon_{frp}}{d-c} \quad (7)$$

All of the outcomes obtained from those methods were later compared with the moment-curvature relationships of the computer software Response 2000. Corresponding comparison of moment-curvature characteristics of the specimens taking into account all the aforementioned methods as well as the outputs obtained from the software is shown in Table 5 and Fig. 6. Experimental moment-curvature relationships of all the GFRP-reinforced specimens are shown in Fig. 7. The comparison of curvatures of the GFRP-reinforced concrete masonry beams shows that as the reinforcement ratio increased and consequently the stiffness characteristics of the beam improved, the curvature of the beam decreased. Beam F-3-1#19 was expected to have curvature values higher than beam F-3-2#16 since the latter beam has higher reinforcement ratio. However, as the beam F-3-1#19 exceeded its expected capacity by 15%, the slope of the curve showing its moment-curvature relationship increased. On the other hand, although the two-course beam F-2-2#16 has the second highest reinforcement ratio, due to its cross-sectional dimensions, its curvature was determined to be higher than the rest of the three-course concrete masonry beams.

Load versus Deflection Prediction

In order to determine the load-deformation relationship of the specimens, effective moment of inertia (I_{eff}) of the cross section had to be determined. Upon selecting the most suitable approach for estimating the effective moment of inertia for GFRP-reinforced concrete masonry beams, various methods were used to estimate the deflections and results were compared. CSA S304.1-04 (2004a) defines the following equation for estimation:

$$I_{eff} = \left(\frac{M_{cr}}{M_a} \right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a} \right)^3 \right] I_{cr} \leq I_g \quad (8)$$

where M_a =applied moment on the reinforced concrete masonry beams; M_{cr} =cracking moment of the reinforced concrete masonry

Table 5. Predicted Maximum Curvatures of the GFRP-Reinforced Concrete Masonry Beams

Beam	Experimental result (mm)	ISIS (mm)	Response 2000 (mm)
S-3-1-15M	26.2	—	26.4
F-3-1#13	25.6	27.4	20.4
F-3-1#19	24.7	19.1	18.7
F-3-2#16	12.4	13.7	20.7
F-2-2#16	37.4	19.2	27.3
F-3-2#19 and 1#16	10.8	10.3	16.9

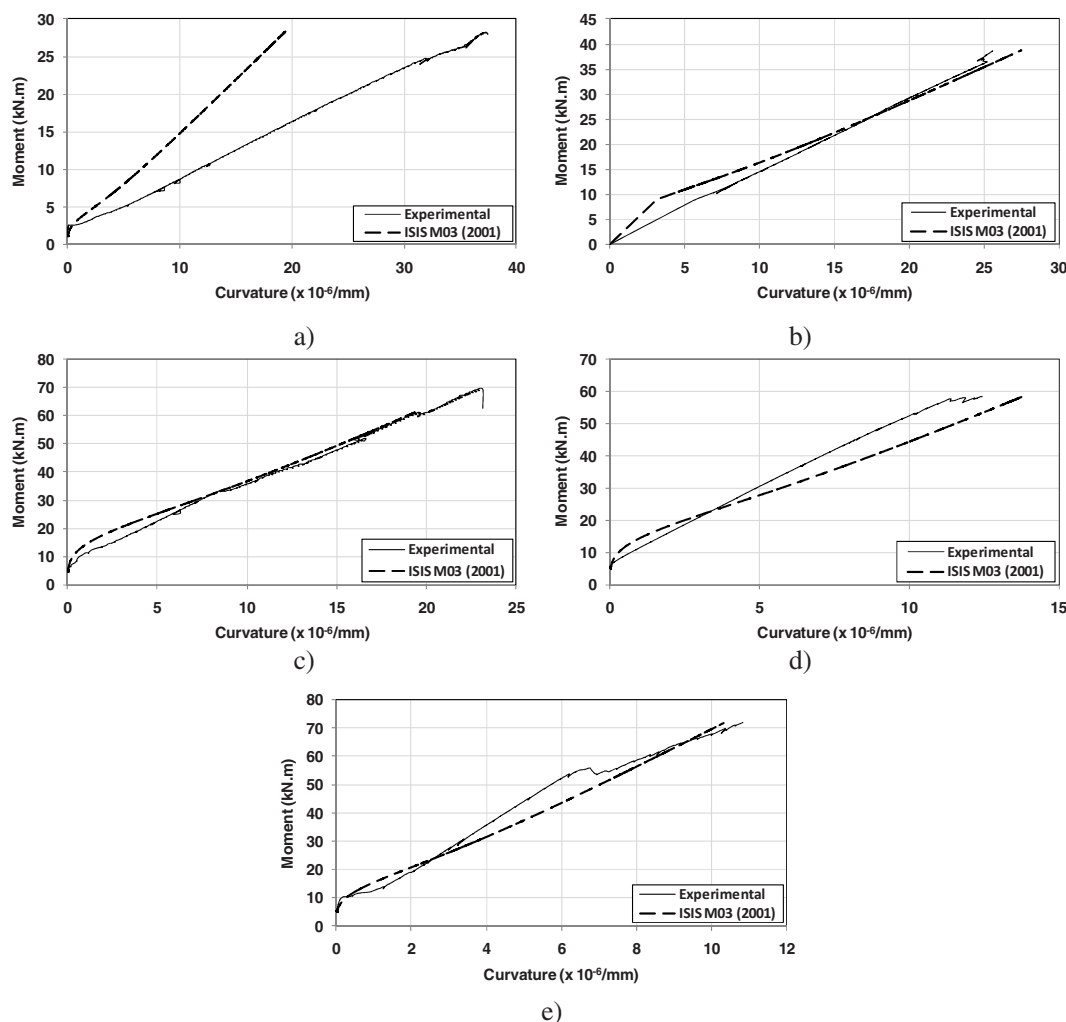


Fig. 6. Experimental and analytical moment-curvature relationships of GFRP-reinforced concrete masonry beams: (a) F-2-2#16; (b) F-3-1#13; (c) F-3-1#19; (d) F-3-2#16; and (e) F-3-2#19 and 1#16

beams; I_{cr} =moment of inertia of fully cracked section; and I_g =moment of inertia of the gross section. ISIS (2001) suggested a similar expression introduced by Benmokrane et al. (1996)

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

$$\text{where } \beta_b = 0.5 \left[1 + \frac{E_{frp}}{E_s} \right] \quad (9)$$

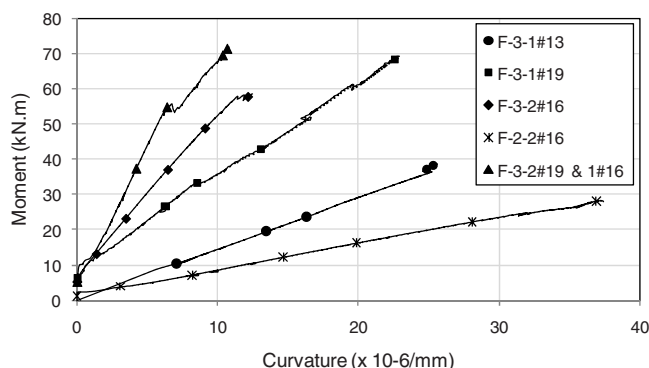


Fig. 7. Experimental moment-curvature curves of the GFRP-reinforced concrete masonry beams

For both CSA S304.1-04 (2004a) and ISIS (2001) methods, deflections at any given load are calculated using

$$\Delta = \frac{Pa}{24EI_{eff}} (3l^2 - 4a^2) \quad (10)$$

where l =reinforced concrete masonry beam span; P =load applied; and a =shear span length. Horton and Tadros (1990) recommended the following method for deflection calculation for steel-reinforced concrete masonry beams:

$$\Delta = \Delta_{cr} \left[1 - \left(\frac{M_{cr}}{M_a} \right)^2 \left(2 - \frac{M_{cr}}{M_a} \right) \left(1 - \frac{I_{cr}}{I_g} \right) \right] \quad (11)$$

In case of GFRP-reinforced beams, it was observed that the numerical methods used to calculate the deflection patterns over-estimated the deflections of the individual beams. Overall results of the calculations showed that CSA S304.1-04 (2004a) and ISIS (2001) methods yielded similar predictions since they are different by a factor β_b and for GFRP-reinforced concrete masonry beams this factor was in the vicinity of unity. As expected, the stiffness of the GFRP-reinforced beams increased as their reinforcement ratio increased meaning that they experienced lower deformations even at higher loads. It was also observed that Response 2000 tends to underestimate the maximum deflections of the GFRP-reinforced masonry beams, while estimating the pat-

Table 6. Predicted Maximum Deflections of the GFRP-Reinforced Concrete Masonry Beams

Beam	Exp. result (mm)	CSA (mm)	ISIS (mm)	Horton and Tadros (mm)	Response 2000 (mm)
F-3-1#13	50.5	73.8	74.2	95.4	28.3
F-3-1#19	68.2	67.2	67.2	68.3	30.6
F-3-2#16	28.6	45.9	45.9	45.8	30.7
F-2-2#16	35.9	40.3	40.4	16.1	16.3
F-3-2#19 and 1#16	16.3	35.2	35.2	34.5	25.2

tern of the load-deflection curve correctly. Corresponding comparison of load-deflection characteristics of the specimens taking into account all the above mentioned methods as well as the outputs obtained from the software is shown in Table 6.

Deformability

Prediction of the deformability of GFRP-reinforced concrete masonry beams using the conventional assumptions adopted in steel-reinforced elements can be misleading due to the linear stress-strain relationship of the FRP reinforcement until rupture. Although GFRP rods have higher ultimate capacities compared to conventional steel reinforcement, their low elastic modulus characteristics leads to large deformations which must be taken into account for the design of FRP-reinforced structural members.

In order to estimate the deformability of GFRP-reinforced concrete beams, specimens were evaluated using several different approaches that exist for deformability and ductility estimation of FRP-reinforced concrete members. ISIS (2001) adopted the calculation of a deformability factor (DF) which is a ratio of ultimate curvature over curvature at serviceability stage

$$DF = \left| \frac{\Psi_u M_u}{\Psi_s M_s} \right| \quad (12)$$

ISIS (2001) assumed that an FRP rod reaches its serviceability stage at 2,000 microstrains. According to the guideline, the allowable DF must be over four for rectangular cross sections. Curvatures of the tested beams were calculated using the experimental strains of the GFRP rods.

On the other hand, energy related deformability was calculated using the area under the load-deflection curves that were plotted using the experimental data. All the specimens excluding the GFRP-reinforced concrete masonry beam F-3-1#13 were unloaded as soon as the compressive failure occurred at the midspan as the load and deflection readings were still being monitored.

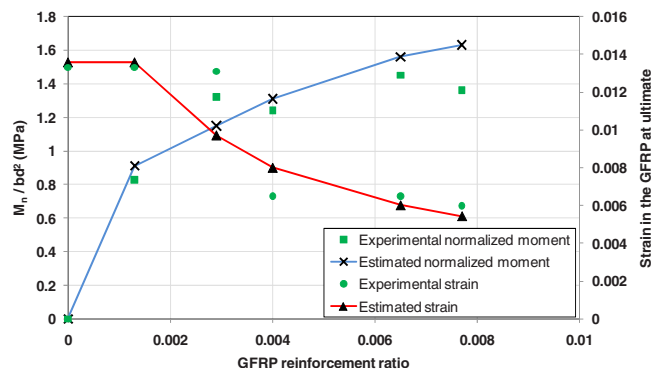


Fig. 8. Proposed design chart for GFRP-reinforced concrete masonry beams ($f'_m = 6.9$ MPa, $f_{frp(ave)} = 755$ MPa, $E_{frp(ave)} = 47.4$ GPa)

Unloading section of the load-deflection curve represents the border line of the areas namely inelastic energy consumed prior to failure and elastic energy released after failure. Grace et al. (1998) reported that the inelastic to total absorbed energy ratio of the specimens should be higher than 75% in order to have ductile behavior and should be 70% for a semiductile behavior. As expected, deformability of the beams decreases as the reinforcement ratios of the cross section increases. However, the beam with the highest reinforcement ratio, beam F-3-2#19 and 1#16 went through large deformations before failure in the compression zone. This tested specimen had a ductility index of 5.9 which was still within the limits recommended by ISIS (2001). On the other hand, dissipated inelastic and elastic energies of the tested GFRP-reinforced concrete masonry beams were calculated and the inelastic to total absorbed energy ratio of the specimens was observed to be in the vicinity of 70%. Specimen F-3-1#13 could not be unloaded since the GFRP bar ruptured as expected, and it was therefore excluded from the calculations. Table 7 shows the energy-based DF of GFRP-reinforced concrete masonry beams and their ductility indexes.

Capacity Diagram

Using the studied flexural performance of internally GFRP-reinforced fully grouted concrete masonry beams, a capacity diagram for GFRP-reinforced concrete masonry beams was proposed which is showed in Fig. 8. This diagram helps the designer determine the necessary reinforcement ratio and corresponding strains in the internal GFRP rods of a reinforced masonry beam by simply assuming cross-sectional dimensions and considering the factored loads acting on the beam. The writers would like to point out the necessity of further studies on internally FRP-reinforced concrete masonry that will not only generalize the findings of this study but also can be used to publish a design guideline for FRP-reinforced concrete masonry.

Conclusions

This paper investigated the flexural response of fully grouted and internally GFRP-reinforced full-scale concrete masonry beams. For that purpose, five GFRP-reinforced concrete masonry beams were tested having different reinforcement ratios. In addition to that, two other fully grouted steel-reinforced full-scale beams were tested as control specimens. The main parameter of this research was the GFRP reinforcement ratio that differed among the GFRP-reinforced specimens. Conclusions are not applicable unless sufficient shear reinforcement is provided following the design guidelines available for reinforced concrete masonry. Also, it has to be mentioned that if the loading scheme and boundary conditions are modified, overall behavior of the tested beams is expected to change accordingly, for which further studies have to

Table 7. Energy-Based DF of GFRP-Reinforced Concrete Masonry Beams

Beam	Inelastic energy	Elastic energy	Total energy	Inelastic elastic (%)	Ductility
F-2-2#16	3.9	1.5	5.4	72	Semiductile
F-3-1#19	3.2	1.4	4.6	70	Semiductile
F-3-2#16	3.1	1.3	4.4	70	Semiductile
F-3-2#19 and 1#16	3.3	1.2	4.5	73	Semiductile

Note: Table values are representations of relative areas under the load-deflection curves of tested specimens therefore do not have units.

be conducted. Further study has to be conducted in order to fine tune the discussed sectional analysis approaches to FRP-reinforced masonry beams in general. The following conclusions were drawn from the experimental and analytical study conducted on the tested full-scale GFRP-reinforced concrete masonry beams and auxiliary prisms:

1. Compression tests conducted on masonry prisms constructed using conventional concrete cinder blocks demonstrated that the CSA S304.1-04 (2004a) slightly overestimates the actual compressive resistance f'_m of the masonry assemblage parallel to its bed joints.
2. Cross-sectional nominal flexural capacity and stiffness of the reinforced concrete masonry beams significantly improved as the internal GFRP reinforcement ratio increased.
3. During each test, compression zone failures of GFRP over-reinforced specimens started with crushing of the mortar joints (head joints) at midspan. As the load increased, cracks on both faces of the blocks widened and the block shells eventually separated resulting with a significant drop in the load. Cracks did not necessarily follow the mortar joints and propagated through the masonry blocks as well. In this sense, GFRP-reinforced concrete masonry beams exhibited similar crack patterns with the GFRP-reinforced concrete beams.
4. For the majority of the GFRP-reinforced full-scale masonry beams, experimental results had good agreement with analytical method described in ISIS (2001) while having minor percentages of errors using value of 6.9 MPa which is the corrected compressive resistance of masonry parallel to its bed joint (f'_m) recommended by the CSA S304.1-04 (2004a). Differences in the capacities could be due to the varying compressive strength of grouted masonry units loaded in the direction parallel to the bed joints.
5. It was observed that the numerical methods used to calculate the deflection patterns overestimated the deflections of the GFRP-reinforced concrete masonry beams. Overall results showed that the methods suggested by the CSA S304.1-04 (2004a) and ISIS (2001) yielded similar predictions.
6. Methods used to calculate curvatures of the GFRP-reinforced masonry beams at different bending moment levels showed good agreement with each other.
7. Curvature-based DF of the GFRP-reinforced beams were within the recommended limits set by ISIS (2001) for FRP-reinforced concrete beams. On the other hand, energy based deformability estimations demonstrated that the GFRP-reinforced concrete masonry beams exhibited semiductile behavior.

Acknowledgments

The writers would like to acknowledge the financial support of le Fonds Québécois de la Recherche sur la Nature et les Tech-

nologies (FQRNT) and Natural Sciences and Engineering Research Council of Canada (NSERC). The experimental phase of this project became possible with support and collaboration of l'Association des Entrepreneurs en Maçonnerie du Québec and Canada Masonry Design Centre (CMDCC).

Notation

The following symbols are used in this paper:

- A_{frp} = total area of the GFRP reinforcement in beam cross section, mm²;
- A_s = total area of steel reinforcement in beam cross section, mm²;
- a = shear span of the reinforced masonry beam, mm;
- b = width of the cross section, mm;
- c = depth of the compression zone in the cross section of the beams, mm;
- d = effective depth of internal longitudinal reinforcement, mm;
- d_b = diameter of the reinforcing bars, mm;
- E_{frp} = Young's modulus of GFRP bars, GPa;
- E_m = Young's modulus of masonry assemblage, GPa;
- E_{steel} = Young's modulus of steel reinforcement, GPa;
- f'_c = compressive strength of concrete, MPa;
- f_{frp} = stress in the GFRP rods, MPa;
- f_{frpu} = ultimate tensile strength of the GFRP rods, MPa;
- f'_m = compressive strength of the masonry assemblage, MPa;
- f_y = field stress of the steel reinforcement, MPa;
- 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;
- l = reinforced masonry beam span, mm;
- M_a = applied moment on the reinforced concrete masonry beams, kN·m;
- M_{cr} = cracking moment of the reinforced concrete masonry beams, kN·m;
- M_n = nominal flexural resistance of the masonry beams, kN·m;
- M_r = moment resistance of the reinforced masonry beam cross section, kN·m;
- M_s = serviceability moment of the reinforced concrete masonry beams, kN·m;
- P = load applied by the hydraulic actuator, N;
- α_1 = stress block factor;
- β = constant dependent on the mode of failure;
- β_1 = stress block factor;
- ϵ_c = compressive strain of concrete;
- ϵ_{frpu} = ultimate tensile strain of the GFRP;

ε_m = compressive strain of masonry;
 ε_{mu} = ultimate compressive strain of masonry;
 ρ_{frp} = GFRP reinforcement ratio;
 ρ_{frpb} = balanced GFRP reinforcement ratio of the masonry beam cross section;
 χ = modification factor to account for the direction of the compressive stress;
 ψ_s = curvature of the reinforced concrete masonry beams at service stage; and
 ψ_u = curvature of the reinforced concrete masonry beams at ultimate stage.

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