ABSTRACT

A significant amount of research has been conducted on individual fibre reinforced polymer reinforced concrete (FRP-RC) structural elements such as beams, slabs, and recently columns, which have shown considerable inelasticity or deformability under monotonic and fatigue loading. However, the behaviour of FRP bars in tension-compression reversals in FRP-RC columns and frame structures has not been investigated yet. Furthermore, the elastic-linear behaviour of the FRP material up to failure is raising concerns on the ability to dissipate energy of frame structures in seismic loading events. Therefore, this research project aims to investigate the feasibility of using FRP reinforcement in such structural elements. A total of four full-scale exterior T-shaped beam-column joint prototypes are constructed and tested under simulated seismic load conditions. The beam measures 2100 mm long, 350 mm wide and 450 mm deep, while the column measures 3650 mm long with a 350 mm square section. Reversal lateral quasi-static cyclic loads are applied directly at the beam tip simulating seismic loading. This paper is focusing on the test results and analysis of two test prototypes. One prototype is totally reinforced with glass FRP bars and stirrups, while the other one is reinforced with steel. The experimental results showed that the joint drift capacity can reach more than 3.0% safely without any considerable damage, which indicates the validity of using glass FRP bars and stirrups as reinforcement in the beam-column joints subjected to seismic-type loading.

KEYWORDS

Glass FRP, Stirrups, RC frames, Seismic design, Beam-column joints.

INTRODUCTION

During a strong earthquake, beam-column joints are subjected to severe reversed cyclic loading. If not designed and detailed properly, the performance of these joints can significantly affect the overall response of a ductile moment-resisting frame building. The design philosophy of such frames is based on providing sufficient ductility to the structure to dissipate the seismic energy by means of inelastic rotations. Extensive research in steel-reinforced concrete frames showed that the inelastic rotations spread over specific regions defined as plastic hinges. During these inelastic deformations, concrete and steel properties are beyond the elastic range. Ductile RC frames are designed based on the concept of strong column-weak beam, where plastic hinges are allowed to form within beams, not the columns (Hanson and Connor 1967). The main function of the joint is to enable the adjoining members to develop and sustain their ultimate capacity. Such design requires that the beam-column joints should be capable of withstanding several inelastic load reversals at the beam plastic hinge without significant loss of strength and energy dissipating ability. Beam-column joints also should have limited deformations in order not to affect the column designed capacity or increase the storey horizontal drift (ACI-ASCE Committee 352 2002).

On the other hand, the corrosion problem of steel reinforcing bars is a major factor in limiting the life expectancy of reinforced concrete structures. In some cases, the repair cost can be twice as high as the original one (Yunovich and Thompson 2003). The fibre reinforced polymer (FRP) reinforcement is currently being used as a construction material in new concrete structures especially those in harsh environments such as parking garages and bridges. The main driving force behind this effort is the superior performance of FRP in corrosive environments due to its non-corrodible nature. However, the FRP material exhibits linear-elastic stress-strain characteristics up to failure with relatively low modulus of elasticity (40-50 GPa for glass FRP and 110-140 MPa for carbon FRP compared to 200 GPa for steel). Moreover, they have different bond characteristics and low strength under compression. These mechanical and physical properties of FRP materials make the behaviour and
performance of concrete structures reinforced with FRP bars different from those reinforced with steel (ACI 2006). Since plastic behaviour is required at framing elements (beam-column joints), the use of the linear-elastic FRP in such elements needs to be investigated. Very little research has been conducted to study the behaviour of concrete columns and frame structures reinforced with FRP reinforcement particularly subjected to seismic loading. Test results have shown some inelasticity and deformability (ductility) under reversed cyclic loading of FRP reinforced concrete columns and frame elements (Fukuyama et al. 1995; Sharbatdar et al. 2007).

To date, structural behaviour and performance of beam-column joints totally reinforced with FRP bars and stirrups have not been investigated. Several codes and guidelines for design and constructions of concrete structures reinforced with FRP bars have been recently published (CSA 2002 & 2006; ACI 2006). However, due to lack of data and test results, none of these codes and guidelines provides any recommendations on the design of beam-column joints when FRP bars are used as primary reinforcement in both longitudinal and transverse directions. In addition, glass FRP (GFRP) bars are cheaper compared to the other available types, carbon and aramid, which make them attractive to the construction industry. Therefore, this research is designed to investigate the feasibility of using GFRP bars and stirrups as reinforcement in the beam-column joints, subjected to seismic loading.

BACKGROUND

Investigating the behaviour of RC beam-column joints has attracted many researchers in the past 45 years due to its critical influence on the overall behaviour of RC ductile moment-resisting frames subjected to large seismic lateral forces. Early studies by Hanson and Connor (1967) indicated that large inelastic deformation limits of individual members allow entire structures to endure severe ground motion while dissipating significant levels of seismic energy. Paulay et al. (1978) adopted an acceptable design approach for beam-column joints as follows:

- The joint strength should be more than the weakest member strength to avoid the need of repair and also prevent the need of mechanisms for energy dissipation,
- The column capacity should be kept safe against any possible degradation in joint strength,
- The elastic performance of the joint during moderate seismic loads is desirable, and
- Ease of construction should be targeted regardless of the joint necessary reinforcement for its performance.

Meanwhile, as a result of the extensive use of FRP in construction, research studies conducting FRP reinforced beam-column joints were required. Fukuyama et al. (1995) studied the feasibility of reinforcing a half-scale, two-bay, two-storey, intermediate concrete frame with aramid FRP bars as a longitudinal and transverse reinforcement under the effect of seismic loading. The design of the FRP-reinforced concrete sections was governed by limiting the frame deformation under ultimate earthquake loading, and the crack width under service loading. Results showed an elastic behaviour of the frame till crushing of concrete. Therefore it was concluded that the rehabilitation processes of FRP reinforced concrete members are easier than those with steel.

Sharbatdar et al. (2007) studied the behaviour of three full-scale exterior beam-column joints reinforced with carbon FRP longitudinal bars and carbon FRP grids (as stirrups). The test parameters were the stirrups spacing within the joint and the arrangement of longitudinal reinforcement in beams and columns. The test specimens achieved a lateral drift ratio exceeds 3.0% while stable elastic behaviour was observed, which means fulfilling the strength and ductility (deformability) requirements of earthquake resistant structures. Also, results showed a good agreement with the confinement provisions required by the CSA-S806-02 (CSA 2002).

DETAILS OF THE EXPERIMENTAL PROGRAM

Test Prototypes

An experimental program is currently in progress in the McQuade Heavy Structural Laboratory at the University of Manitoba to investigate the feasibility of using GFRP reinforcement in RC frames subjected to seismic loading. The full experimental program includes construction and testing of a number of full-size exterior (T-shape) beam-column joint prototypes. Figure 1 shows the overall dimensions of a typical test prototype where the column is 3650 mm high with a 350-mm square cross section and the beam is 2100 mm long with 350×450 mm in cross section. To date, only two test prototypes have been constructed and tested. The first test prototype (SC) is reinforced with steel bars and stirrups as a control specimen. The second one, GC, is reinforced with GFRP bars and stirrup. The CSA-A23.3-04 (CSA 2004) and CSA-S806-02 (CSA 2002) codes were used to design the two test specimens, as appropriate. The resulting reinforcement is shown in Fig. (1).
It is worth to mention that the two specimens were designed to have the same flexural strength ratio, \( M_R = 1.15 \) (where \( M_R \) is the sum of columns’ flexural strength to the sum of flexural strength of the beam attached to the joint) to compare the behaviour of the two test prototypes.

![Figure (1): Dimensions and reinforcement of the beam-column joint specimens](image)

**Materials**

All test specimens were constructed using normal weight, ready-mixed concrete with a targeted 28-day concrete compressive strength of 30 MPa and maximum aggregate size of 20 mm. All test prototypes were cast in the horizontal position and wet-cured in the laboratory for 7 days. The actual concrete compressive and tensile strengths were determined from standard cylinder tests (100×200 and 150×300 mm). On the day of testing the specimens, the obtained average concrete compressive strength was about 32.4 MPa and 32.9 MPa for the two test prototypes SC and GC, respectively.

Two types of reinforcing bars were used in this study; CSA grade 400 deformed steel bars for specimen SC, and sand-coated GFRP V-ROD™ (Pultrall Inc. 2007) for specimen GC. The nominal strength of GFRP No.16 bar is 751 MPa with a tensile modulus of 48.2 GPa. Also, for No.13 GFRP stirrups, the strength of the straight portion is 590 MPa with a modulus of elasticity of 46.3 GPa.

**Test Set-up and Loading Scheme**

The test specimen represents an exterior beam-column connection cut from the frame structure at the points of contra flexure (assumed at mid-span and mid-height of the beam and the column, respectively). The isolation of test prototypes from the frame structure requires that proper boundary conditions are simulated in the test setup. The column two ends were restrained against both vertical and horizontal displacements meanwhile their rotations were allowed (hinged boundary conditions). During testing, the test specimens were rotated 90 degrees such that the column member was in the horizontal position and the beam member in the vertical position as shown in Figure (2). A heavy strong reaction steel frame was pre-stressed to the strong floor at the other end of the column to sustain the column reaction against the applied loads.

![Figure (2): Test Set-up](image)  ![Figure (3): Loading History](image)
A 1000-kN fully dynamic actuator of ± 250 mm stroke, positioned horizontally against a rigid RC reaction wall, was used to apply displacement cycles at the beam tip. The column was concentrically subjected to an axial load of 670 kN (15% of the column ultimate capacity) and kept constant during testing. As reported by many researchers (Hakuto et al. 2000; Ghobarah and El-Amoury 2005), the loading process was carried out based on a load-control mode followed by a displacement-control mode. At the load-control phase, specimens were subjected to a cycle of 35.0 kN to detect the cracking load and test the instrumentations. Then, another cycle of 75.0 kN representing service loading conditions (corresponding to 60% of the yielding strain of specimen SC) prior to applying the seismic loading. As per the acceptance criteria for moment frames based on structural testing (ACI 2005), the seismic loading then was applied under a displacement-controlled mode at a rate of 0.01 Hz, quasi-static type-loading as shown in Figure (3). The first three cycles represent the displacement corresponding to the yielding of the longitudinal reinforcement of the beam, \( \Delta_y \), then followed by steps of three cycles each of amplitudes equal to multipliers of the yield displacement; 1.27 \( \Delta_y \), 1.92 \( \Delta_y \), 2.53 \( \Delta_y \), 3.80 \( \Delta_y \), 5.07 \( \Delta_y \), and 6.30 \( \Delta_y \). The same seismic loading scheme and test procedure were applied to the GFRP-reinforced specimen.

Both internal and external instrumentation were used to monitor the behaviour during the test. A large number of strain gauges were mounted on longitudinal and transverse reinforcement at critical locations to detect strains during testing. While a group of twelve linear variable displacement transducers (LVDTs) were used to monitor the column and beam rotations, joint distortion, and crack width.

**TEST RESULTS AND ANALYSIS**

**Hysteresis Behaviour**

Plots of applied lateral load versus the drift ratio at the beam tip for both specimens are presented in Figure (4). The drift ratio was calculated as the horizontal displacement of the beam end divided by the distance between the point of load application and the column centreline. The hysteresis diagrams indicate that the steel-reinforced specimen (SC) sustained the probable design capacity in a stable behaviour up to a drift ratio of 4.0% then load degradation was observed at a drift ratio of 5.0% as shown in Figure (4.a). Specimen SC had an elastic performance with no stiffness degradation up to a drift ratio level of 1.0%. Test results showed that specimen SC had a gradual reduction in stiffness up to 80% of its initial stiffness, this is primarily due to degradation of concrete in and near the joint, and localized slippage of beam longitudinal reinforcement. Within each cycle of loading, it can be noted that a loss of stiffness, pinching of the hysteresis loop, near the zero displacement point was recorded. This may be attributed to the wide flexural cracks formed near the face of the column as a result of steel yielding (permanent damage).

Regarding the GFRP-reinforced specimen (GC), Figure (4.b) shows the hysteresis relationship of the applied load at the third cycle versus the lateral drift ratio. It is clear that the specimen had elastic-stable behaviour up to failure at 3.0% drift ratio. No stiffness degradation was recorded during the testing up to failure as shown in Figure (4.b).

![Figure (4): Hysteretic load-drift ratio relationship](image)

**Strain Measurements**

Longitudinal strains in flexural reinforcing bars of the beam at the column face versus the corresponding lateral load at the third cycle of each seismic loading step are shown in Figure (5). For the steel-reinforced specimen
(SC), it was observed that up to a drift ratio level of 1.0% strains developed in an elastic mode. As shown in Figure (5.a), the maximum developed strain was about 10,000 micro-strain; measured at a drift ratio of 2.0%. Afterwards, a reduction in the developed strains was observed at subsequent load cycles up to failure (6200, 2700, and 2100 micro-strain at drift ratios of 3.0%, 4.0%, and 5.0%, respectively). This may be due to the slippage of beam’s flexural reinforcing bars near the beam plastic hinge and joint zones.

For the GFRP-reinforced specimen (GC), contrary to specimen SC, the strains remain linear-elastic up to failure at a drift ratio level of 3.0%. Moreover, the maximum measured strains in the beam bars were close enough to those of steel (9500 micro-strain), as shown in Figure (5.b).

\[ \text{Crack Pattern and Mode of Failure} \]

Figure (6) shows the crack patterns for both specimens, SC and GC. Specimen SC exhibited the basic joint-shear mode of failure, which resulted in degradation of the concrete strength inside the joint. Flexural cracks started and concentrated at the column face, however, due to the relatively small flexural strength ratio of the specimen \( (MR = 1.15) \), cracks penetration into the joint was observed leading to the reported joint shear failure (Fig. 6.a).

On the other hand, specimen GC exhibited a flexural slippage failure due to the insufficient development length for the GFRP longitudinal reinforcement of the beam. It was observed that flexural cracks were distributed over the beam length due to the elastic behaviour of the GFRP bars. It is worth to mention that the joint area of the GC specimen exhibited elastic-stable behaviour with insignificant deformation up to failure (3.0% drift ratio) compared to the same area of specimen SC as shown in Fig. (6.b).
CONCLUSIONS

The purpose of this investigation was to study the feasibility of using the GFRP bars as a longitudinal and transverse reinforcement for reinforced concrete frames subjected to high seismic loads. Based on the results of the tested specimens, the following conclusions are drawn.

1- GFRP bars and stirrups can be used as reinforcement in the beam-column joints subjected to seismic loading conditions.
2- GFRP bars are capable of resisting tension-compression cycles with no strength degradation.
3- GFRP-reinforced joints can be designed to satisfy both strength and ductility (deformability) requirements of earthquake-resistant structures.
4- The GFRP-reinforced joint reached a drift capacity of 3.0% without any significant permanent damage. In other words, high ductility of steel-reinforced joints can be adequately replaced with the large-elastic strains developed by GFRP bars.

If bond/slip failure can be avoided, it is expected that the GFRP-reinforced joint can safely reach a higher drift capacity under reversed cyclic loading. This is the objective of the remaining two specimens in the current study.

ACKNOWLEDGEMENTS

The authors wish to express their gratitude and sincere appreciation for the financial support received from the Natural Science and Engineering Research Council of Canada (NSERC), through Discovery and Canada Research Chair programs. The help received from the technical staff of the McQuade Heavy Structural Laboratory in the department of civil engineering at the University of Manitoba is also acknowledged.

REFERENCES