SIMULATED SEISMIC BEHAVIOR OF GFRP-REINFORCED CONCRETE COLUMNS

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ABSTRACT: This paper presents experimental results of four full-scale square concrete columns reinforced with longitudinal and transverse glass-fiber-reinforced-polymer (GFRP) bars. Each specimen consisted of a column with a 400×400 mm cross section and 1850 mm in length, cast not monolithically with a 1200×1200×600 mm stub. The column part of the specimen represents the column between the sections of maximum moment and zero moment in a structure, while the stub represents a footing or a joint. These specimens were tested under lateral cyclic quasi-static loading while simultaneously subjected to constant axial load. Based on the measured hysteretic loops of lateral load vs. tip deflection relationships, a series of parameters related to deformability and flexural strength are used to evaluate the seismic behavior of each column and provide valuable parameters in understanding the seismic behavior of these columns. The results showed that concrete columns reinforced with GFRP longitudinal bars and transverse ties can behave in a manner with a stable post-peak response and achieve high levels of deformability. The results indicate that GFRP bars can be used as internal reinforcement in ductile concrete columns.

1. Introduction

Fiber-reinforced-polymer (FRP) reinforcement is being used as an effective replacement for steel in new reinforced-concrete structures, especially those that are exposed to corrosive environments. The reason is the corrosion resistance of FRP materials. Given the linear-elastic stress–strain characteristics up to failure without a yielding plateau with low modulus of elasticity of FRPs compared to steel calls for new information about using these materials as reinforcement for structures in areas prone to earthquakes. Structural members (such as columns) in seismic regions require reinforcement to have inelastic behavior. Very little research has yet been conducted on the seismic behavior of FRP-reinforced moment-resisting frames and shear walls. Mady et al. (2011) studied the seismic behavior of beam–column joints reinforced totally with glass-fiber-reinforced-polymer (GFRP) bars and stirrups. Their seismic behavior was compared to that of a steel-reinforced specimen. Each specimen simulated a beam–column connection of an exterior bay in a multistory reinforced concrete moment-resisting plane frames. The span of the frame (bay length) was 4700 mm with a story height of 3650 mm. Each specimen represents an exterior connection between assumed contra-flexural points at mid-height of the columns and mid-span of the beams. The beams were 2350 mm long and 350×450 mm in cross section. The columns were 3650 mm high with a cross section of 350×500 mm. It was concluded that the GFRP-reinforced joints could successfully sustain a 4.0% drift ratio without significant damage. This indicates the feasibility of using GFRP bars and stirrups as reinforcement in beam–column joints subjected to seismic-type loading. Increasing the beam reinforcement ratio while satisfying the strong-column–weak-beam concept can enhance the ability of the joint to dissipate seismic energy. Mohamed et al. (2014) tested four reinforced-concrete shear walls, including one reference steel-reinforced specimen (ST15) and three GFRP-reinforced specimens (G15, G12, and G10) under quasi-static loading. The specimens represent a model of a single medium-rise shear wall. The minimum thickness and reinforcement details for the steel-reinforced wall were designed according to CSA A23.3 (2004) and ACI 318 (2008), whereas CSA S806 (2012) and ACI 440.1R (2006) were used for the GFRP-reinforced walls, where applicable. The wall specimens were designed with an adequate amount of distributed and concentrated reinforcement to ensure flexural domination and to prevent shear, sliding shear, and anchorage failures. The wall specimens were all 3500 mm high and 200 mm thick. ST15 and G15 were 1500 mm long. ST15 served as a reference for G15, since it had the same concrete dimensions and similar reinforcement axial stiffness. The lengths of G12 and G10 were 1200 mm and 1000 mm, respectively. It was concluded that properly designed and detailed GFRP-reinforced shear walls could reach their flexural capacities with no strength degradation and that the failure behavior could be effectively controlled. Paramanantham (1993) studied the behavior of 16 FRP-reinforced concrete columns under eccentric loading, testing the specimens under different axial loading and moment combinations. The longitudinal and transverse reinforcement were GFRP bars. All the specimens failed in compressive crushing mode. The column behavior was characterized by dividing the ratios of moment–deflection relationship slopes into three parts: the first and the second parts were linear; the third part nonlinear. When the ratio of the first to the second slope varied between 1.25 and 1.85, the failure was defined as compressive failure. When the ratio varied between 2.22 and 2.37, the failure was defined as compressive with tensile cracking. When the ratio varied between 5.5 and 6.0, the failure was defined as compressive–flexural failure. It was also concluded that the best results were produced with a transverse spacing of 100 mm. The maximum tensile stress measured in the FRP bars was about 70% of their ultimate tensile strength in direct tension. Tavassoli et al (2015) studied the seismic behavior of GFRP-reinforced circular concrete columns. The specimens were tested under lateral cyclic quasi-static loading and subjected to constant axial load. The parameters studied were the axial-load level, GFRP type, and the size and spacing of GFRP spirals. Two levels of column axial load were studied (0.28 Po and 0.42 Po), where Po is the column's nominal axial load capacity. It was concluded that the concrete columns reinforced with GFRP bars and spirals had a stable response and GFRP type had no significant impact on column behavior. Moreover, the columns with higher axial load sustained more damage and displayed lower levels of ductility and deformability. It was concluded that the transverse steel reinforcement provided more confinement for the column core before the steel yielded. Once the steel had started to yield, however, the confinement was less effective compared with the GFRP spirals, which continued to increase the confinement of the concrete core and delayed its crushing. Experimental studies are needed to verify the applicability of moment-resisting frame members reinforced with GFRP bars and stirrups under different loading types, especially seismic loading. The columns are very important members in theses frames and are considered as carrying the building's loads. GFRP bars and stirrups are needed for buildings to reduce the building deterioration due to steel corrosion. This poses serious concerns about their applicability for earthquake and wind-resistant structures, in which the inelastic reinforcement in these members is expected to dissipate energy. Our experimental study was conducted to investigate the seismic behavior of GFRP-reinforced concrete columns to develop design and detailing requirements for these columns under seismic loading. Four large-scale GFRP-reinforced concrete columns were constructed and tested under lateral cyclic quasi-static loading while being simultaneously subjected to constant axial load. The results of the experimental study are summarized in the following sections.

2. Experimental Program

2.1. Specimen Design

The column specimen represents a full-scale, first-floor columns to simulate the seismic resistance of FRP-reinforced columns. As illustrated in Fig. 1-a, the tested columns were 1850 mm high with a cross-section of 400 × 400 mm. The base stub thickness was 600 mm. The base stub was used to fasten the specimen to the lab rigid floor and serve as an anchorage length for the vertical bars. The axial compression force applied on the column was distributed along a rigid beam and taken as $0.2A_cf_c$' as suggested by Paulay and Priestley (1995). CSA A23.3 (2004) and ACI 318 (2008) provisions for minimum dimensions and reinforcement ratio were applied to the column specimens. The design for flexure, shear, and confinement followed the CSA S806-12 (2012) and ACI 440.1R-06 (2006) provisions.

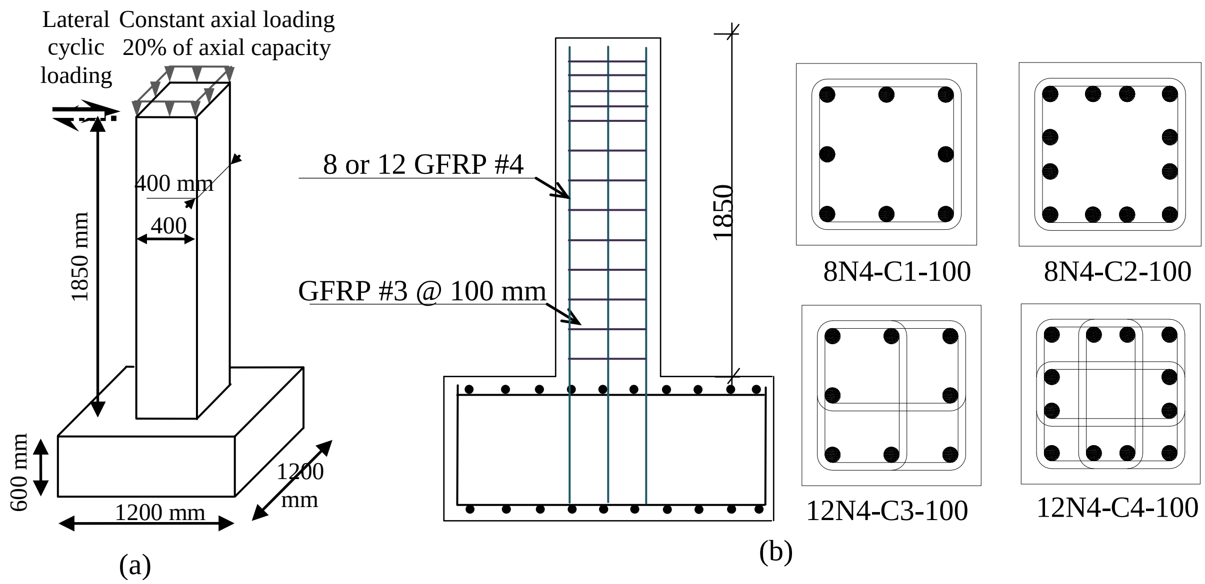

Fig. 1 – Test specimens: a) Dimensions, and b) reinforcement details

2.2. Materials

The columns were made with normal-weight, ready-mixed concrete with an average 28-day compressive strength of 40 MPa. The base stub was reinforced with 20M steel bars to assure its rigidity. The columns were reinforced with Grade III sand coated GFRP reinforcing bars manufactured by Pultrall (Thetford Mines, QC, Canada). Table 1 provides the reinforcement properties: d_b = bar nominal diameter, A_f = nominal cross-sectional area, E_f = modulus of elasticity, f_{fu} = guaranteed tensile strength, ε_{fu} = ultimate strain. For longitudinal reinforcement, either 8 or 12 #4 GFRP bars were used. The transverse reinforcement was #3 GFRP bent bars spaced at 100 mm.

Table 1 - Mechanical properties of GFRP bars as provided by the manufacturer

Bar		d_b (mm)	A_f (mm²)	E_f (GPa)	f_{fu} (MPa)	ε_{fu} (%)
#4		12.7	126.7	69.1 ± 2.5	1392	2
#3	Straight portion	9.5	71.3	51.9 ± 0.5	962 ± 19	2.11
	Bend portion			---	500 ± 52	---

2.3. Specimens

Four column specimens were built and tested to failure under reversed cyclic lateral loading and subjected to constant axial load. The specimens represented part of a first-story building column between the footing and point of inflection. The specimens were identified as illustrated in Fig. 2.

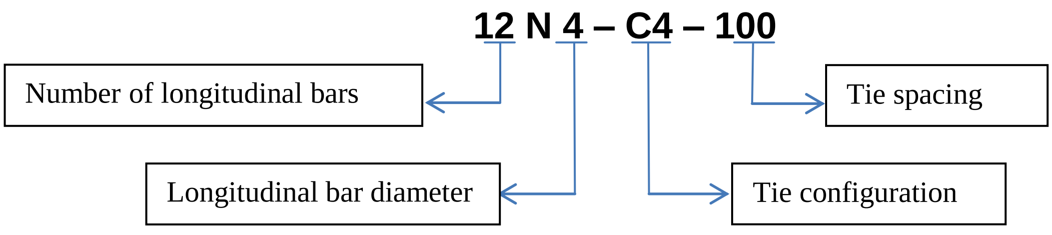

Fig. 2 – Identification key

The column specimens were chosen to represent a dominantly flexural behavior. Four different configurations; as shown in Fig. 1-b, were constructed to study the effect of the number of vertical bars and the effect of the horizontal ties on the seismic response of the FRP-reinforced columns.

2.4. Test Setup

Figure 3 shows the layout of the test setup. The column specimens were tested in an upright position. A specially fabricated steel load-transfer assembly was used to transfer both axial and lateral loads to the specimens. An axial load of approximately 20% of the columns' axial capacity was applied at the top of the columns and was maintained constant throughout each test.

The test setup consisted of three main divisions: (1) Fastening the base stub to the lab floor. In preparation for testing, the stub was leveled on the laboratory floor. The base was fastened to the laboratory floor with four prestressing 66 mm diameter Dywidag bars (high-strength steel bars) to prevent uplifting and/or horizontal sliding during the application of the lateral load. (2) Axial loading system. An axial load simulating gravity load was produced with two hydraulic jacks applying tension on two high-strength 34 mm Dywidag steel bars placed on both sides of the column. These steel bars were connected at the bottom to a stiff steel plate anchored with the base stub to the laboratory rigid floor. The upper sides of the bars were connected to the steel load-transfer beam, which rested on the top of the column in order to distribute the axial load. (3) Lateral loading system. Lateral load was applied to the specimen with a 500 kN MTS-hydraulic actuator.

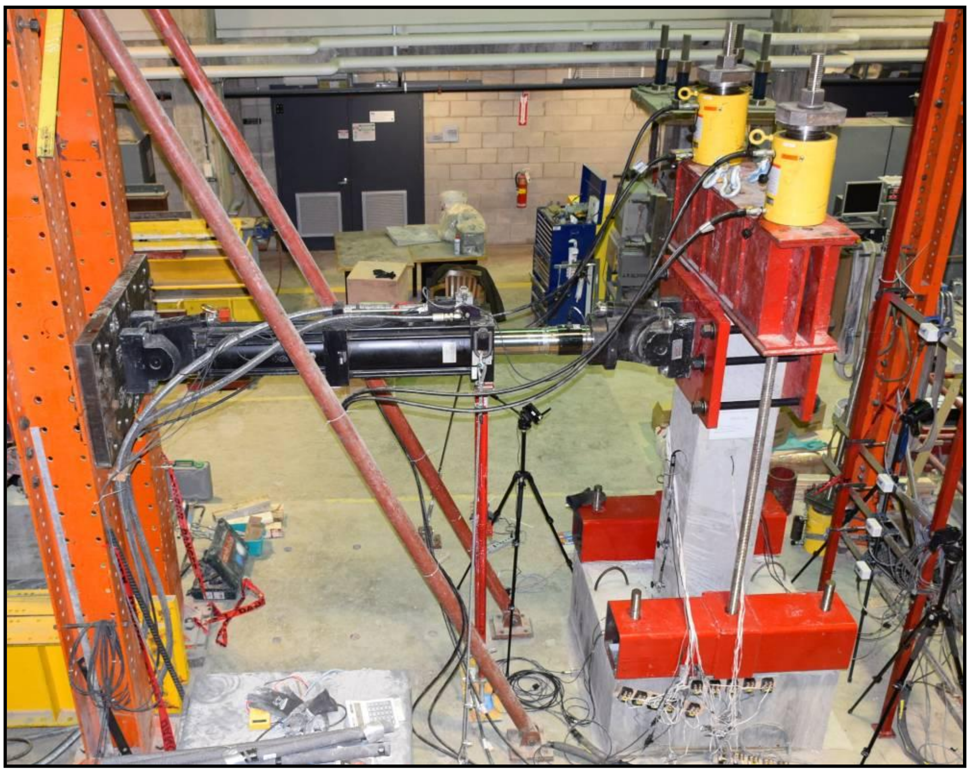

Fig. 3 – Test setup

2.5. Test Procedure

The axial load was slowly increased to the maximum value and held there throughout the testing. The value of the axial load was calculated according to compressive stress equaling 0.2 fc'. Since the effect of loading history was not a test variable, the typical procedure of applying reversed cyclic loading until failure was used. Displacement control loading was used throughout the test. The displacement was applied in two cycles at the same amplitude with increments of 0.5% drift up to 3% drift, followed by an increment of 1% drift up to failure. Figure 4 illustrates a typical sequence of displacement cycles.

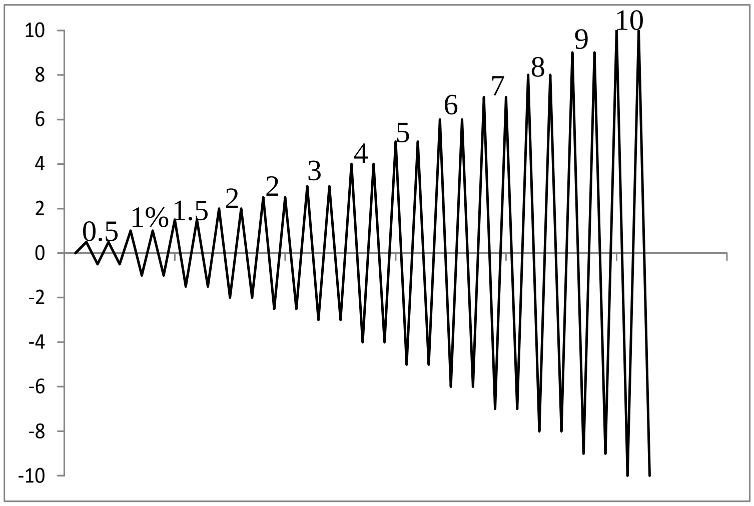

Fig. 4 – Sequence of loading displacement

2.6. Instrumentation

A series of linear variable differential transducers (LVDTs) and strain gauges were used to measure critical response quantities. Lateral displacement was measured at the top of the column. Two LVDTs were used to measure horizontal sliding between the column and base as well as between the base and rigid floor (unlikely to occur). Strain gauges were glued to the longitudinal GFRP bars to measure strains at the extreme compression and tension fibers at a height of 50 mm from the column base. Figures 5-a and b, respectively, show the configurations of the LVDTs and strain gauges.

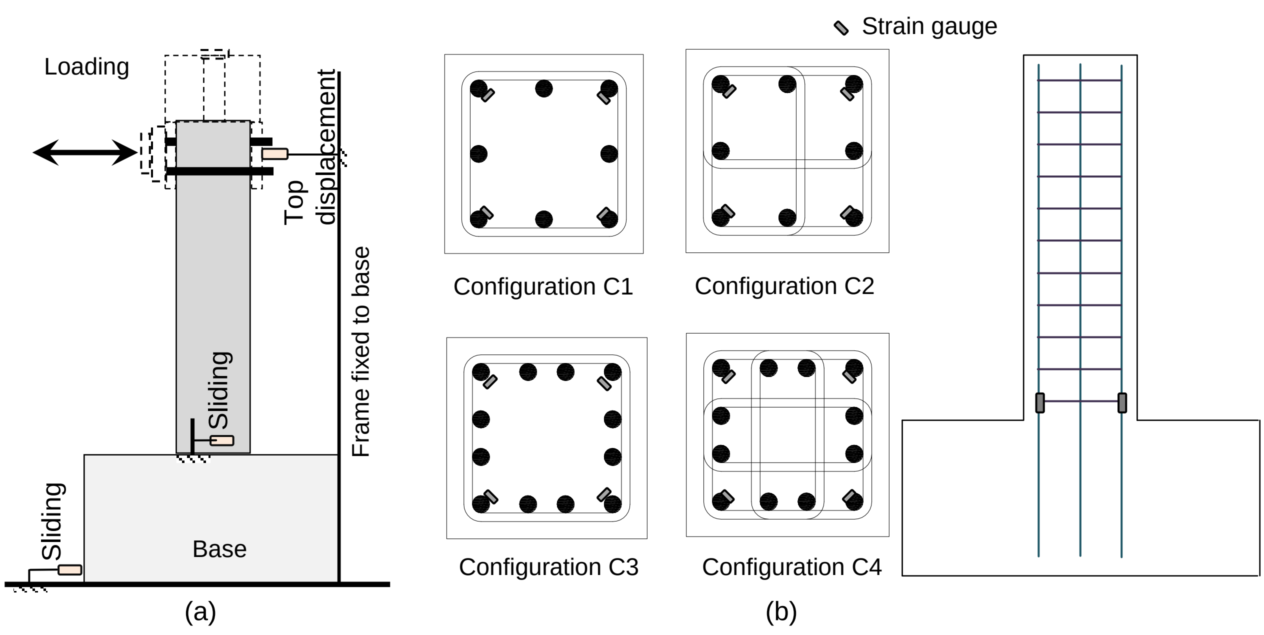

Fig. 5 – Instrumentation

3. Test Results and Discussion

3.1. Crack Pattern and Failure Mode

Figure 6 shows the cracking pattern at the end of the test. The specimens failed in the expected mode, beginning with flexural cracks, followed by shear cracks. The first flexural crack occurred in the lower part of the columns at approximately 40kN. The cracks were horizontal within the spacing of the horizontal stirrups and ties. Flexural cracks extended to about half of the effective height of the columns (700 mm). The flexural cracks were followed by shear cracks. The inclination of the shear cracks was quite higher in the top part than in the cracks in the bottom part. As cycling increased deformations, the rhomboidal pieces of concrete between the intersecting cracks gradually deteriorated; spalling of the concrete cover occurred on both sides of the columns.

8N4-C1-100 8N4-C2-100 12N4-C3-100 12N4-C4-100

Fig. 6 – Crack pattern

Fig. 7 – Concrete crushing

Fig. 8 – GFRP stirrup cut

Fig. 9 – GFRP longitudinal bar fracture

Thereafter, a significant loss of strength, leading to failure, was observed when the concrete deteriorated in the most heavily stressed parts of the compressed side of the columns (Fig. 7). As depicted in Figs. 8 and 9, stirrups were cut and longitudinal GFRP bars in compression fractured.

3.2. Hysteretic Response

Lateral load–drift ratio (top displacement) results, as shown in Fig. 10, demonstrate a general similarity to the behavior of the steel-reinforced column. The unloading/reloading curves seem to demonstrate linearity depending on GFRP elastic behavior. The reloading branches followed a similar loading path but at a lower loading stiffness, resulting in lower peak strength. The unloading path shape seems to be dependent on the strain at the onset of unloading. The hysteretic response curves indicate that the first excursion of a new displacement level followed the loading path of the second excursion of the previous displacement amplitude. This suggests that additional cycles at a specific displacement level would produce negligible damage compared to that experienced in the first unloading/reloading cycle.

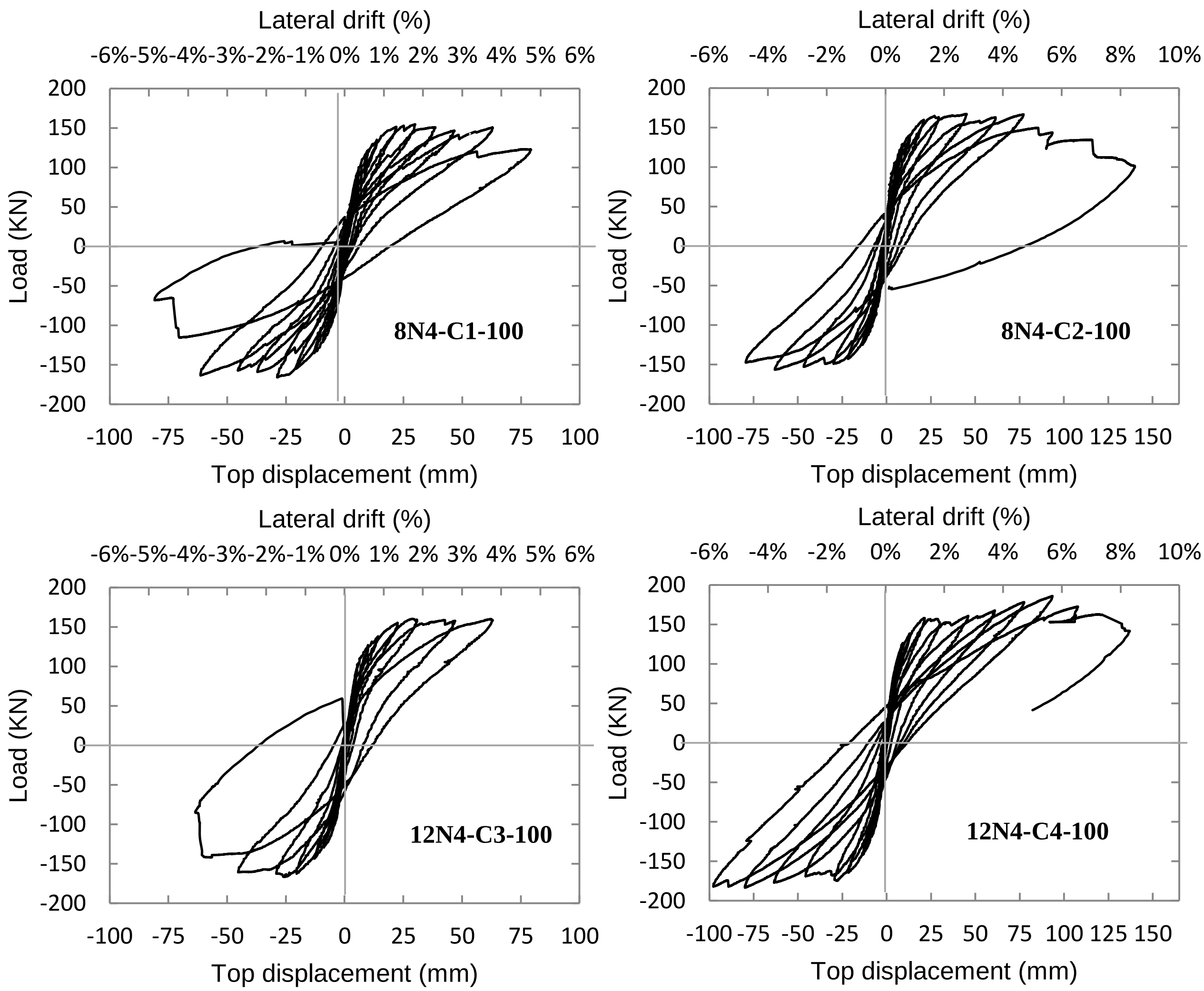

Fig. 10 – Hysteretic response

The behavior of the tested specimens remained linear elastic with no stiffness degradation associated with pinched hysteretic response up to almost 0.8% drift. With further application of the load, a gradual decrease in the overall stiffness combined with increase and expanding in the loops was observed up to failure. Desirable deformability behavior for the columns was observed starting from 1.3%, 1.8%, 1,2%, and 1.7% for 8N4-C1-100, 8N4-C2-100, 12N4-C3-100 and 12N4-C4-100, respectively, until failure. At this point, the load capacity was slightly decreased, indicating that more than 60% of the specimens' lateral deformation occurred with roughly no change in the load. This phenomenon is close to the yielding of steel bars, which is considered a desirable phenomenon for structural members subjected to seismic loads.

The effect of the horizontal ties on the hysteretic response could be clarified by comparing the results of 8N4-C1-100 and 8N4-C2-100. It can be deduced that the confinement of the columns' core with cross ties played an effective role in increasing the deformability and strength of the GFRP-reinforced concrete columns. This enhancement was gained by increasing the reinforcement with internal cross ties (nominal cross-sectional area of 71.3 mm^2). This increased reinforcement resulted in a 27% increase in the drift ratio (deformability) and about a 7% increase in strength.

Moreover, comparing the hysteretic response of 12N4-C3-100 and 12N4-C4-100 would indicate that increasing the area of internal stirrups enhanced the lateral drift ratio (deformability) and column strength. Using closed stirrups with a total nominal cross-sectional area about 142.6 mm^2 resulted in a 116% increase in lateral drift and about a 12% increase in column strength.

It is clear that the GFRP-reinforced columns could achieve an acceptable drift ratio (deformability) of higher than the 2.5% stated by CSA S806-12 even without horizontal ties (8N4-C1-100 and 12N4-C3-100). Adding horizontal ties significantly enhanced the confinement of the concrete core, resulting in high deformability with more than 7% drift. Therefore, GFRP-reinforced columns can be accepted as resisting lateral forces.

3.3. Reinforcement Strain Measurements

Figure 11 shows the measured strain in the longitudinal bars close to the column–footing interface vs. load for all four tested columns.

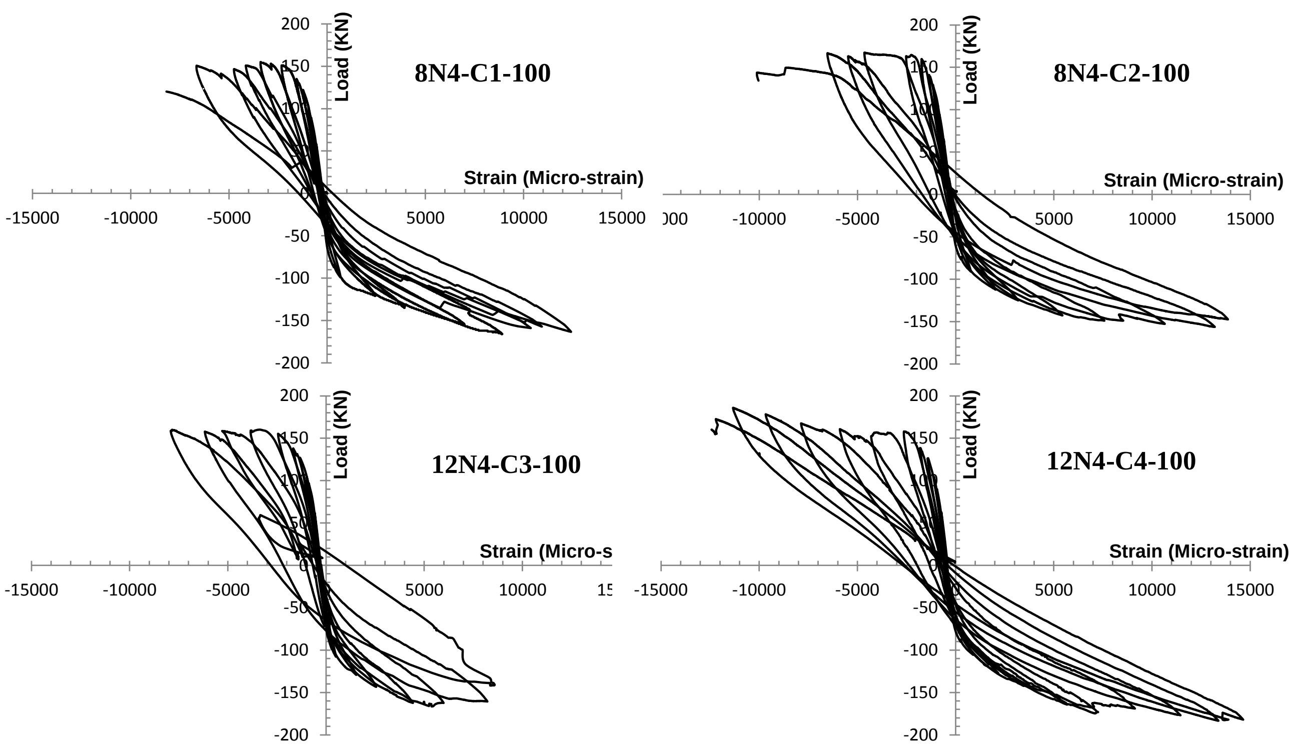

Fig. 11 – Strain in longitudinal reinforcement

The strain in the GFRP bars remained linear elastic up to 1000 and 500 microstrains in compression and tension, respectively. After that, a nonlinear curve was observed reaching the maximum strain followed by a descending part. A sort of load stability was observed, combined with increasing strain values. Finally, load degradation occurred until failure. Higher values were obtained when the shear reinforcement was increased with ties or internal closed stirrups. The strain in column longitudinal reinforcement strain reached 12,426 and 14,645 microstrains in compression and tension, respectively, in the case of column 12N4-100-C3 compared to 7,906 and 8,587 microstrains in compression and tension, respectively, in the case of 12N4-100-C1. Moreover, 8N4-100-C2 reached 10,123 and 13,852 microstrains in compression and tension, respectively, and the longitudinal reinforcement in 8N4-100-C1 was 8,180 and 12,422 microstrains in compression and tension, respectively. These numbers clearly show the significant effect of shear reinforcement in increasing strain values and deformability. Increasing the longitudinal reinforcement ratio had no significant impact, however, on column strain and deformability. Based on these observed large stain values, GFRP bars can be used in this type of column to counter seismic loads by given the large strain and deformation values, which can serve to offset the fact that GFRP bars to not yield.

4. Conclusion

This study was carried out on concrete columns in order to assess the validity of using GFRP bars to resist lateral loads in reinforcing columns. Four GFRP-reinforced columns with different reinforcement configurations were investigated. Steps were taken in building, casting, and testing the specimens to represent a real model of a column in an actual RC building site. The findings are promising with respect to applications for GFRP bars.

- All specimens achieved their flexural strength with no sign of premature shear, sliding shear, bond and anchorage failure, or instability failure.
- The GFRP-reinforced columns had a pinched hysteretic behavior without strength degradation, indicating acceptable behavior of such specimens.
- All the specimens showed a stable hysteretic behavior with a drift capacity of over 3.5%.
- The horizontal ties had a clear effect in increasing the drift ratio to more than 7%, which is well beyond the required deformability in design codes.
- The moderate damage (cover splitting) occurred at relatively high drift levels (2.5%).
- The GFRP-reinforced columns behaved elastically with realigned cracks and recoverable deformation of up to a lateral drift of 3%.
- Increased deformation at a roughly constant load was observed. This phenomenon is close to the yielding of steel bars, which is considered desirable a phenomenon for structural members subjected to seismic loads.

Therefore, since the GFRP-reinforced columns attained good strength and deformation capacity, GFRP reinforcement could be used in lateral resisting systems, although further research is needed to implement adequate design guidelines and recommendations for such structural elements.

5. Acknowledgements

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