



EFFECT OF SPACING OF TRANSVERSE REINFORCEMENT ON THE LAPPED SPLICED GFRP-RC COLUMNS SUBJECTED TO CYCLIC-REVERSED LOADS

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ABSTRACT

Recently, the non-corrodible fibre-reinforced polymer (FRP) materials have been used successfully as reinforcement for concrete structures. However, the behaviour of glass (G) FRP-reinforced concrete (RC) columns under seismic loading has not been explored yet. This paper presents the results of an experimental program that investigates the contribution of GFRP transverse reinforcement to the confinement of concrete core in lap-spliced GFRP-RC columns. Three full-scale column specimens were constructed and tested to failure under quasi-static cyclic-reversed loads. All specimens were reinforced with GFRP longitudinal bars and transverse stirrups. The columns had 350-mm square cross section and 1850-mm cantilever length. Each column was cast on heavily steel-RC footing measuring $1400 \times 1400 \times 600 \text{ mm}^3$, which was constructed to simulate rotational fixity of the column and to force the failure to occur in the column. The splice length for each column was equal to sixty times the diameter of the longitudinal column reinforcement. The test variable was the spacing between the transverse GFRP reinforcement (75, 100 and 150 mm). Test results are presented in terms of mode of failure, load-drift diagrams, energy dissipation, ultimate capacity and code comparison, if applicable. The results showed that, decreasing the spacing of spiral reinforcement improved both the strength and the deformability of the columns. Moreover, the requirement of the Canadian standard for FRP-RC buildings related to the amount of confinement provided by GFRP transverse reinforcement is adequate to ensure stability of the longitudinal bars.

Keywords: Reinforced Concrete (RC); Fibre Reinforced Polymer (FRP); Seismic; Lap Splice; Quasi-Static; Cyclic-Reversed Loads

1. INTRODUCTION

Considerable amount of research has been conducted on the behaviour of fibre-reinforced polymer (FRP) reinforced concrete (RC) members, such as beams and slabs. However, very limited studies are available on GFRP-RC columns subjected to cyclic-reversed loads (*Sharbatadar 2003; Tavassoli et al. 2015; Ali and El-Salakawy 2015*). These previous studies demonstrated that the energy dissipated by GFRP-RC columns is lower than their steel-RC counterparts. Due to significant ductility in steel, steel-RC members exhibit desirable energy consumption. This behaviour is not pertinent for GFRP-RC members, which behave in linear-elastic manner until failure. To overcome this issue, the use of hybrid seismic resistant dampers is a possible solution. Although, steel-RC columns showed better performance in dissipating seismic energy, the overall behaviour of GFRP-RC columns was adequate to consider their use in seismic regions (*Tavassoli et al. 2015; Ali and El-Salakawy 2015*). Moreover, GFRP-RC specimens experienced large deformation prior to failure, and were able to attain adequate drift capacity. The drift capacity far exceeded the 2.5% drift ratio recommended by the National Building Code of Canada (NRC 2010).

Lap splicing in RC columns cannot be avoided in moment resisting frames. Lap splices in framed buildings are required to: (1) suit concreting schedule; (2) satisfy the limited lengths of the reinforcing bars; (3) facilitate the

continuation to smaller diameter bars at upper storeys where reduction in loads is expected. In practice, a lap splice in columns of a building frame is provided directly above the floor, within the potential plastic hinge region. Since this plastic hinge zone experiences extensive damage, it adversely affects the anchorage of the lap spliced longitudinal reinforcement. Accordingly, proper design and detailing of GFRP-RC columns with lap splices is deemed necessary. However, to date, there has been no research conducted on the GFRP-RC columns with lap splices subjected to combine axial and cyclic-reversed loading.

A study is being initiated at the University of Manitoba to investigate the behaviour of GFRP-RC columns with lap splices. The first phase of this study was to determine the adequate lap splice length required for GFRP-RC columns under combine axial load and seismic loading. The results showed that, the splice length equal to 60 times the diameter of the longitudinal column reinforcement was sufficient to provide anchorage and stable load-drift response. The other key parameter that affects the behaviour of columns with lap splices under seismic loading is the spacing of transverse reinforcement. Transverse confinement of concrete elements in compression can significantly improve their capacity and deformability. Lack of research limits the use of FRP transverse reinforcement in lapped spliced GFRP-RC columns. In the Canadian standard CSA S806-12 (CSA 2012), the use of transverse reinforcement is permitted in moment resisting frame members subjected to significant axial load. However, the standard does not specify any safe limit for the spacing of GFRP transverse reinforcement, in case of GFRP-RC columns with lap splices. The objective of this paper is to present the results of an experimental investigation that studies the effect of spacing of transverse reinforcement on the lapped spliced GFRP-RC columns subjected to cyclic-reversed loads.

2. EXPERIMENTAL PROGRAM

2.1 Test Specimens

In this study, three full scale square ($350 \times 350 \text{ mm}^2$) GFRP-RC column prototypes were constructed and tested. Each column prototype represents the part of a first story column between foundation and assumed point of contra-flexure at column mid-height. The overall dimensions are shown in Figure 1. Each column had 1850 mm cantilever length (shear span of 1650 mm). To provide rotational fixity to the column, $1400 \times 1400 \times 600 \text{ mm}^3$ steel-RC footing was provided. To simulate the field conditions, the steel-RC footing and the GFRP dowel bars were cast monolithically, then the column was cast a week later. A splice length, equal to 60 times the diameter of the GFRP longitudinal column reinforcement was kept constant for all three specimens. Each column was reinforced with GFRP longitudinal bars and stirrups. Concrete-crushing failure was achieved by providing longitudinal reinforcement ratio more than the balanced one. The design of GFRP-RC columns (P-M interaction curve) was based on the guidelines provided by recent research studies (Choo *et al.* 2006; Hasaballa and El-Salakawy 2015). For the spacing and amount of GFRP transverse reinforcement, recommendations of CSA/S806-12 (CSA 2012) standard were considered.

2.2 Material Properties

All specimens were cast using normal weight ready-mix concrete with a 28-day target compressive strength of 35 MPa and a maximum aggregate size of 20 mm. All test prototypes were wet cured for seven days after casting. The average compressive strength attained on the day of testing was in the range of 40 to 48 MPa, as shown in Table 1. Sand coated (Pultrall Inc. 2014) with high ultimate tensile strength GFRP bars and stirrups were used. Table 2 indicates the mechanical properties of GFRP reinforcement, measured according to CSA/S806-12 (CSA 2012) standards.

Table 1: Design Features of Specimens

Specimen ID	G-75S	G-100S	G-150S
Transverse Reinforcement	No. 10 @ 75 mm	No. 10 @ 100 mm	No. 10 @ 150 mm
Longitudinal Reinforcement		8-15M GFRP Bars	
Concrete Strength (MPa)	40	46	48
Splice Length		60 d_b (960 mm)	

Table 2: Properties of GFRP bars

GFRP Bar Type	Diameter (mm)	Area (mm ²)	Modulus of Elasticity (GPa)	Ultimate Tensile Strength (MPa)	Ultimate Strain (%)
No. 15M	15.9	198	62±0.6	1,184±32	1.89±0.1%
No. 10 (bent)	9.5	71	50±0.7	1,022±38 ^a	2.04±0.1%

*^a Strength of the straight portion of the bent bar.

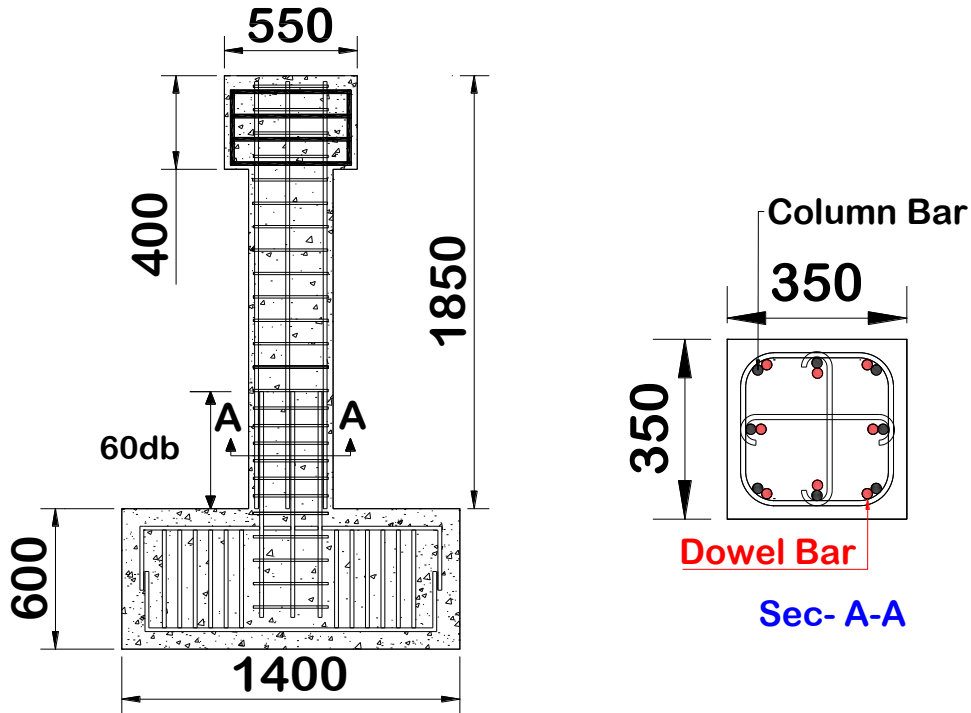


Figure 1: Details of test prototype (all dimensions in mm)

2.3 Test Setup

The test setup is illustrated in Figure 2. A hydraulic jack was used to pre-stress steel-RC footing with the strong floor, to ensure rotational fixity of the column. Quasi-static cyclic-reversed loads were applied using an actuator which had 1000kN capacity and ± 250 mm stroke. The actuator was centered on the column head and was supported on a laboratory strong wall. During the test, the columns were constantly loaded using a hinged frame, with an axial load of 10% of the concrete column capacity ($0.1 A_g f'_c$). The direction and the magnitude of the axial load were to simulate the service conditions.

2.4 Loading Scheme and Procedure

The loading scheme had two phases (Figure 3). The first was the service loading phase, which had two cycles. In the first cycle, columns were loaded until first crack appeared, while during the second cycle; service loading conditions were reached (25% of ultimate strain of GFRP longitudinal bars). The second was the cyclic loading phase, which was based on displacement controlled mode. This phase of loading consisted of variable displacement amplitudes (steps), which represented the increase in drift ratio (ACI 2015). Each cyclic loading step included three identical cycles (at same displacement amplitude), in order to have steady formation of cracking pattern. Once the cyclic loading step reached 2% drift ratio, one load-controlled cycle with the maximum load equal to service condition was applied, to assess the stiffness degradation of the columns, if any.

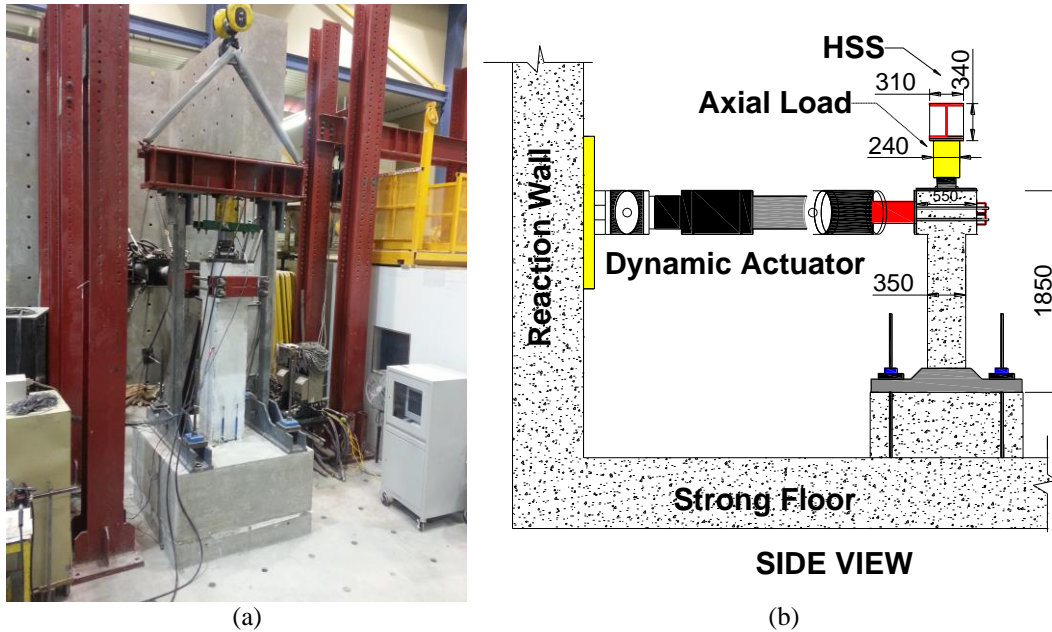


Figure 2: Test Setup (all dimensions in mm): (a) front photo of the test setup (b) schematic drawing of side view

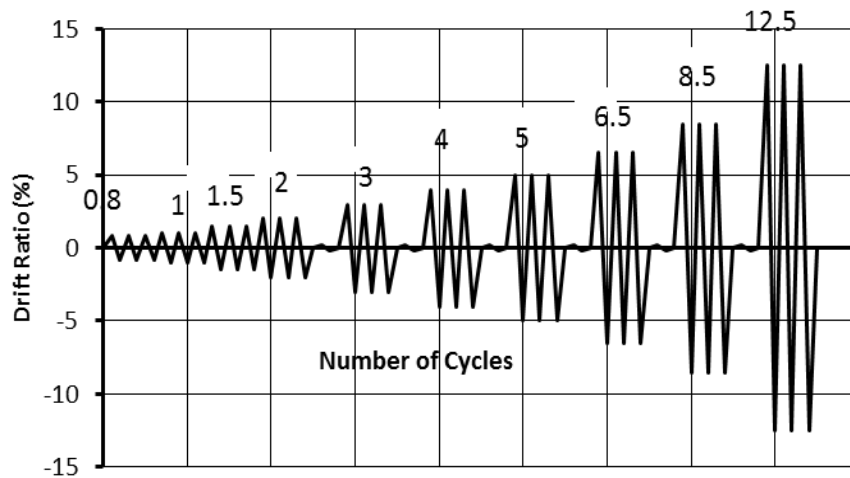


Figure 3: Loading Scheme

3. TEST RESULTS AND DISCUSSION

3.1 Load-Lateral Drift Response

The relationship between the applied lateral load and drift ratio of the column at the point of load application is indicated by the hysteresis loops (Figure 4). The drift ratio corresponds to the ratio of the column displacement at the point of load application to the distance between that point and the column footing interface i.e. 1650 mm. Specimen G-75S showed linear elastic response up to 5% drift ratio (as indicated in Figure 4a). However, it started to show non-linear behaviour after 5% drift ratio, due to significant concrete damage. Maximum lateral load resistance (equal to 158 kN) was recorded at 2.5% drift ratio. At 5.0% drift ratio, the lateral capacity dropped to 143 kN and then remained approximately constant up to 8.5% drift ratio. Significant pinching was observed starting at 8.5% drift ratio. The specimen failed at 12.5% drift ratio with 29% strength loss. Specimen G-100S showed linear-

elastic behaviour up to 4% drift ratio, followed by non-linear behaviour up to failure (Figure 4b). Starting at 5% drift ratio, pinching was observed. The maximum lateral capacity of 148 kN was recorded at 4% drift ratio. The specimen failed at 8.5 % drift ratio load step, with 35% reduction in strength. Figure 4(c) shows the load-drift response for specimen G-150S. The specimen behaved linearly up to 3% drift ratio. Compared to specimens G-75S and G-100S, specimen G-150S showed relatively less maximum lateral resistance (139 kN), which was recorded at 3% drift ratio. The strength degradation was 38% when the specimen failed at 8.5% drift ratio. It is worth mentioning that, for specimen G-100S and specimen G-150S, selected spacing was 14% and 71% more than that recommended by CSA/S806-12, respectively. However, the maximum drift capacity equal to 8.5% of these specimens was considerably more than the 2.5% and the 4% drift ratio required by the National Building Code of Canada (NRCC 2010) and CSA/ S806-12 (CSA 2012), respectively.

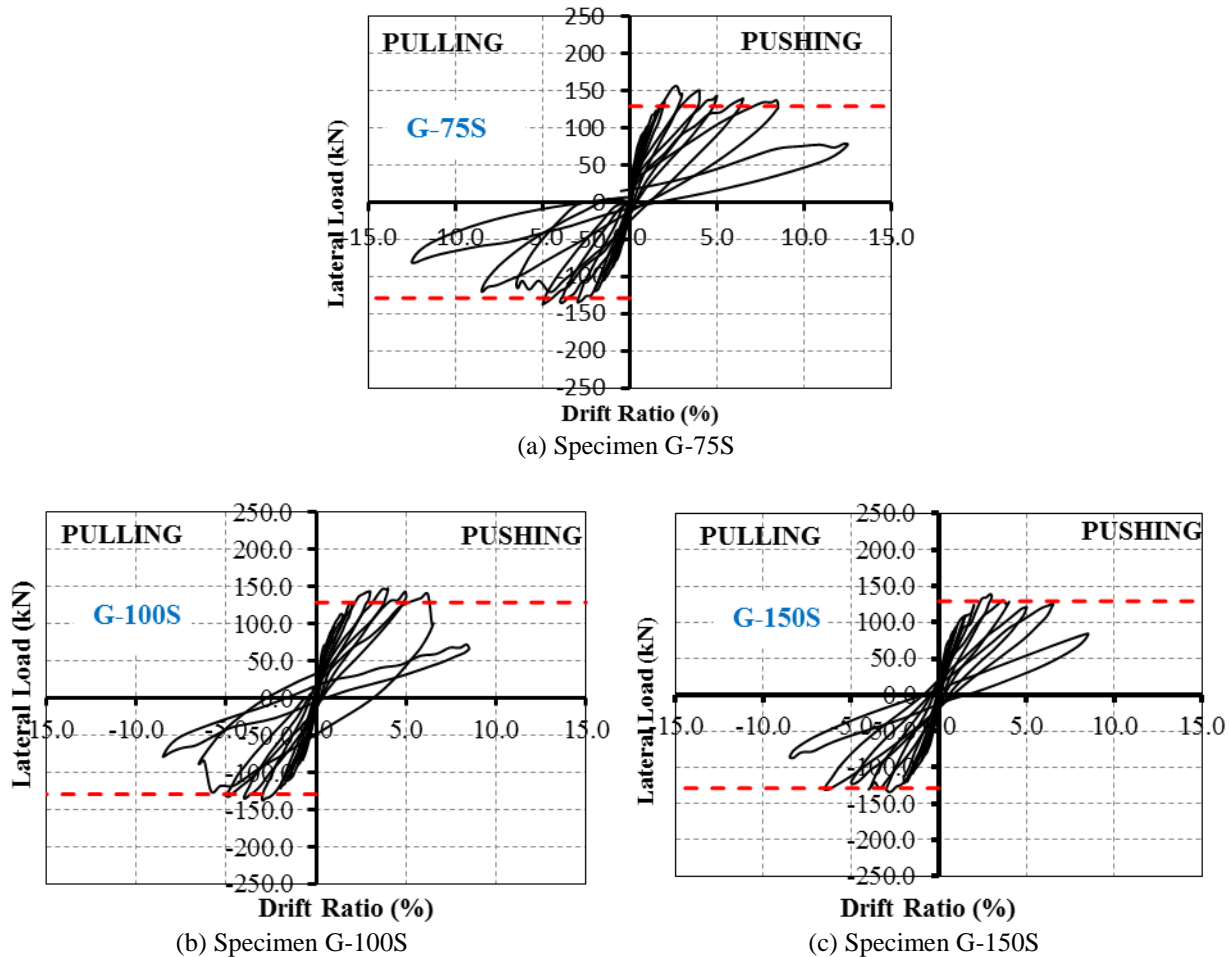


Figure 4: Load Lateral Drift Relationship

3.2 Mode of Failure

At the end of each loading step, cracks were marked after visual inspection. At higher drift ratios, the damage was recorded using digital video cameras. For all test specimens, flexural cracks were observed at the face of the column, perpendicular to the direction of load application at approximately 55 kN. During the 0.8% drift loading stage, flexural cracks further developed and spread on the column faces. At 2% drift ratio, the cracks were extended towards the sides of the columns, while the existing cracks on the faces got widened. However, intensity of the flexural cracks was lower in specimen G-150S compared to specimens G-100S and G-75S. This showed ineffective confinement at early stages of loading for this specimen. Spalling of concrete started at 3% drift ratio in all specimens, which was mostly limited to the hinging zone (approximately one column depth above footing).

Concrete damage in specimen G-150S was more severe, which exposed the reinforcement cage at 4% loading stage, whereas for specimens G-75S and G-100S, the reinforcement cage was exposed at 5% drift ratio.

The flexural failure in the GFRP-RC specimens occurred due to the crushing of concrete followed by buckling and compression failure of dowel bars. Failure in specimen G-75S occurred at 12.5% drift ratio (Figure 5a). The buckling and compression failure of specimen G-100S and specimen G-150S started at 6.5% drift ratio. These early indications of failure were due to the instability of the dowel bars caused by increased spacing and widened cracks, which ultimately led to the failure of the two specimens at 8.5% (Figure 5b and 5c).

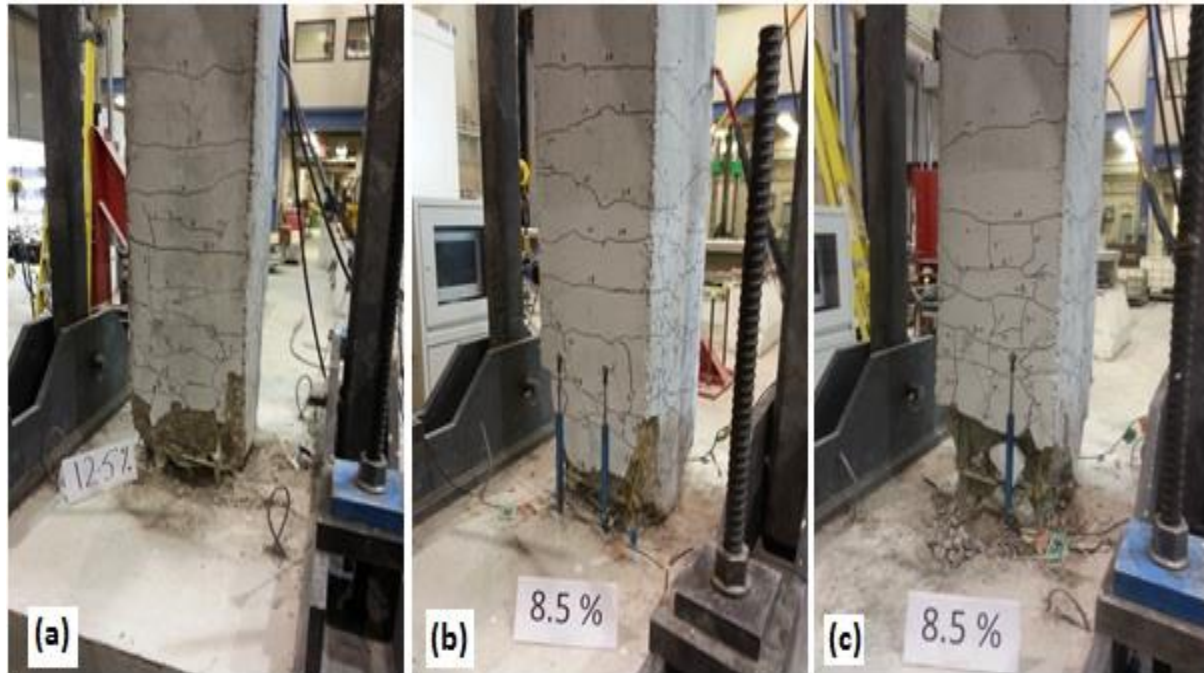


Figure 5: Condition of the specimens at failure: (a) G-75S; (b) G-100S; (c) G-150S

3.3 Cumulative Energy Dissipation

Figure 6 shows the cumulative energy dissipated by each specimen, at different drift ratio. Cumulative energy is calculated as the area enclosed by the hysteresis loops. Because of the linear elastic behaviour, all specimens showed same low level of energy dissipation up to 3 % drift ratio (> NBCC limit, 2.5%). During the later stages of loading, specimen G-75S experienced higher energy dissipation due to the fact that, it showed higher lateral resistance with significant pinching of the hysteresis loops. Specimen G-150S showed the least energy dissipation levels after 5% drift ratio. This explains that, well-confined columns have a better non-linear response under seismic excitations.

3.4 Developed Strains in Transverse Reinforcement

The maximum strain in column stirrups and drift ratio relationship is shown in Figure 7. The figure also includes the allowable strain limit, 6000 micro-strain, recommended by CSA/S806-12 (CSA 2012). Transverse reinforcement strain in specimens G-75S and G-100S increased as the applied drift ratio was increased, until it reached a maximum value; then dropped down when the lateral capacity of the column was reduced. Also, maximum recorded stirrup strains were significantly lower than the rupture strain, before initiation of strength degradation in both specimens. This behaviour indicates that closely spaced GFRP stirrups were able to provide effective confinement to the columns.

In contrast to the above mentioned specimens, higher stirrup strains were developed in specimen G-150S; despite the fact that it showed least lateral resistance. At 4% drift ratio, the recorded stirrup strain exceeded the allowable limit of 6000 micro-strain, specified by CSA/S806-12 (CSA 2012). This observation can be explained as follows.

First, the spalling of concrete cover caused a serious crushing of concrete core. Most of the crushed concrete gravitated towards the GFRP stirrups, which resulted in higher strains. Second, the unsupported length of the GFRP longitudinal bars was increased due to wide stirrup spacing. This caused the early buckling of the longitudinal bars. The buckling of longitudinal bars applied outward pressure on to the GFRP stirrups, which led to the increase in strains.

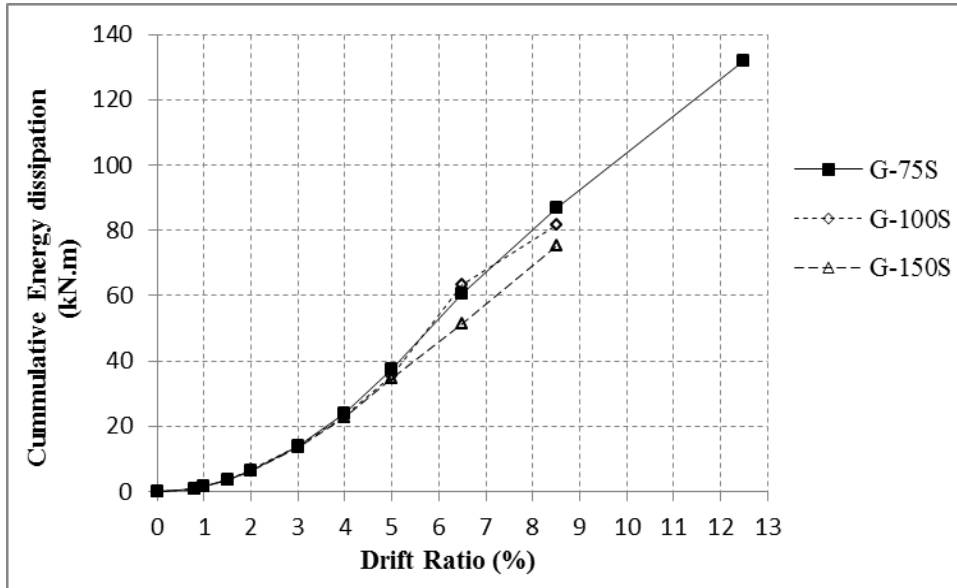


Figure 6: Cumulative Energy Dissipation

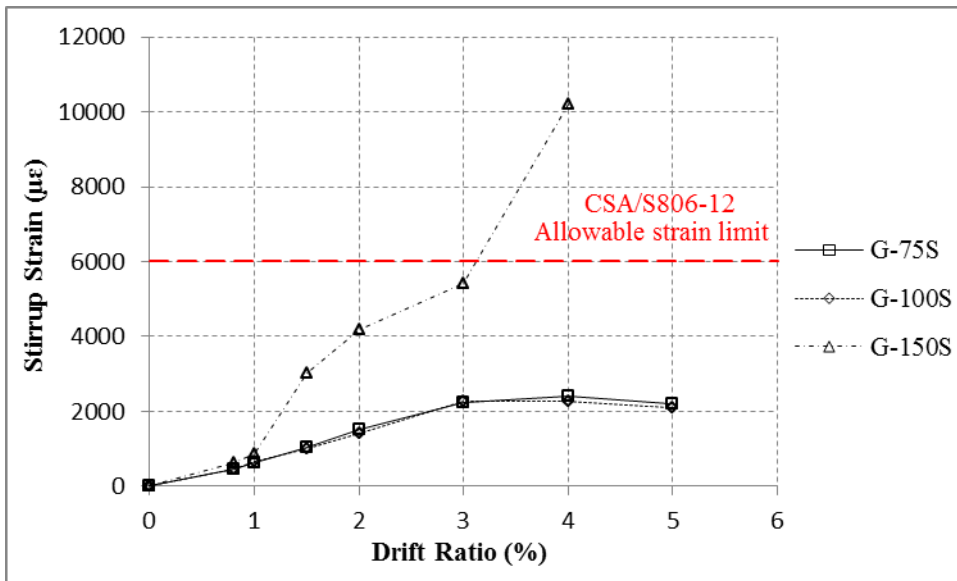


Figure 7: Maximum strain in column stirrups–drift ratio relationship

4. CONCLUSIONS

Based on the limited test results, the following conclusions can be derived:

- GFRP transverse reinforcement (stirrup) was effective in providing confinement to the concrete columns and provided lateral restraint up to failure, due to its linear elastic behaviour.
- The efficiency of widely spaced GFRP transverse reinforcement (more than 0.43 times column dimension) was found to be unreliable, and should be ignored in seismic design of GFRP columns.
- The results of specimen G-75S showed that, closely spaced transverse reinforcement (less than 0.22 times column dimension) enhanced the level of confinement, which increased the deformability and prevented the early crushing of concrete
- The requirement of the Canadian standard CSA/S806-12 (CSA 2012) for the amount of confinement provided by GFRP transverse reinforcement is adequate to ensure stability of the longitudinal bars.

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