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Abstract

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Performance Evaluation of High Strength Concrete and Steel Fibre High Strength Concrete Columns Reinforced with GFRP Bars and Helices

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Abstract

This study presents the results of an experimental investigation on high strength concrete (HSC) and steel fibre high strength concrete (SFHSC) circular column specimens reinforced longitudinally and transversely with Glass Fibre-Reinforced Polymer (GFRP) bars and helices, respectively. The Influence of the type of the reinforcement (steel and GFRP), the pitch of the transverse reinforcement, the addition of the steel fibres and the loading condition (concentric, eccentric and four-point loading) on the performance of the specimens was investigated. The study showed that the GFRP bar reinforced HSC (GFRP-HSC) specimen is as efficient as the steel bar reinforced HSC (steel-HSC) specimen in sustaining concentric axial load. However, the maximum load sustained by the GFRP-HSC specimens under eccentric axial load was 10-12% lower than the maximum load sustained by the steel-HSC specimens. GFRP bar reinforced SFHSC (GFRP-SFHSC) specimens sustained 3-13% higher axial load and 14-27% greater ductility than GFRP-HSC specimens under different loading

conditions. Furthermore, reducing the pitch of the GFRP helices in GFRP-SFHSC specimens resulted in a significant improvement in the ductility and the post-peak axial load-axial deformation behaviour of the specimens.

1. Introduction

Fibre-Reinforced Polymer (FRP) reinforcing bars feature many advantageous characteristics such as high tensile strength, high durability, light weight and resistance to harsh environmental conditions. These features make the FRP reinforcing bars ideal replacements for the conventional steel bars in reinforcing concrete structures that require such features. Investigation on the structural behaviour of FRP bar reinforced concrete members became the major objective of many recent studies. The flexural behaviour of FRP bar reinforced normal and high strength concrete members were extensively investigated in the last two decades [1, 2]. These studies significantly contributed in developing guidelines and standards for the design of FRP bar reinforced concrete flexural members. However, the behaviour of FRP bars under compression loads is considered complicated. This is because the nonhomogeneous and anisotropic nature of the FRP bars, which leads to micro-buckling of fibres in the FRP bars under axial compression [3]. Accordingly, The ACI 440.1R-06 [4] does not recommend reinforcing concrete columns longitudinally with FRP bars. The CAN/CSA S806-12 [5] ignores the contribution of FRP bars in the compression zone of both flexural and compression members. Moreover, the ACI 440.1R-15 [6] provides no guidelines for the use of FRP bars in reinforcing compression members. The structural behaviour of FRP reinforced compression members were investigated in few research studies [7-9]. However, these studies were limited to FRP bar reinforced concrete columns cast with normal strength concrete (NSC) with compressive strength lower than 50 MPa. Hence, the observations obtained from these studies may not be adequate for FRP bar reinforced HSC

columns, since the behaviour of HSC columns differs significantly from NSC columns [10-12]. Given the lack of experimental investigations on HSC compression members reinforced with FRP reinforcement, this study intends to expand the current state of knowledge through experimentally investigating the structural behaviour of HSC columns reinforced longitudinally and transversely with Glass Fibre-Reinforced Polymer (GFRP) bars and helices, respectively. Investigations on the behaviour of Carbon Fibre Reinforced Polymer (CFRP) and Aramid Fibre Reinforced Polymer (AFRP) bar reinforced concrete columns are considered beyond the scope of this paper.

The majority of the experimental results reported in the previous studies on the behaviour of FRP bar reinforced NSC columns [13-15] were based on columns tested under concentric axial load. Only few studies provided experimental data from columns tested under eccentric axial load [16-17]. In fact, concrete columns are usually subjected to a combination of concentric axial load and bending moment rather than a pure concentric axial load. Hence, this study investigates the effect of different loading conditions (concentric and eccentric axial load as well as four-point loading) on the behaviour of GFRP bar reinforced HSC columns (GFRP-HSC).

The other focus of this study is to investigate the effect of adding steel fibres to the GFRP bar reinforced HSC (GFRP-HSC) columns. The main objective of the addition of steel fibres is to overcome the lack of ductility that might be experienced by the GFRP-HSC columns, where both HSC and GFRP bars are brittle compared to the NSC and conventional steel bars, respectively. In addition, steel fibres may improve the post-peak behaviour of GFRP-HSC columns and thus providing adequate warning before the failure of GFRP-HSC columns.

Hence, the behaviour of GFRP bar reinforced steel fibre high strength concrete (GFRP-SFHSC) column is also investigated in this study.

2. Experimental Program

2.1 Specimen Design and Preparation

The experimental tests consisted of 16 circular column specimens of 210 mm diameter and 800 mm height. The specimens were divided into four groups with four specimens in each group. The specimens in the first group (Group S60) were prepared as reference specimens for comparison purposes. The Group S60 specimens were reinforced in the longitudinal direction with six N12 (deformed steel bars with 12 mm diameter) and transversely with R10 (rounded steel bars with 10 mm diameter) helices with 60 mm pitch. Group S60 specimens satisfy the requirements of ACI 318-14 [18]. The specimens in the second group (Group G60) were reinforced with six #4 (nominal diameter = 12.7 mm) GFRP bars in the longitudinal direction and transversely with #3 (nominal diameter = 9.5 mm) GFRP helices with a pitch of 60 mm. The specimens in this group were designed to investigate the effect of the direct replacement of steel reinforcement with the same amount of GFRP reinforcement on the behaviour of HSC columns. The specimens in the third group (Group G60F) were also reinforced with six #4 GFRP bars and with #3 GFRP helices with 60 mm pitch in the longitudinal and transverse directions, respectively. In addition, steel fibres with volumetric ratio (v_f) of 1% were added to the HSC mix used in casting the specimens in Group G60F. The specimens in this group were designed to investigate the effect of the addition of steel fibres on the behaviour of GFRP bar reinforced high strength concrete (GFRP-HSC) columns. The specimens in the fourth group (Group G30F) were reinforced longitudinally with six #4 GFRP bars and transversely with #3 GFRP helices with 30 mm pitch. As in Group G60F, steel fibres of 1% (by volume) were added to the HSC mix used in casting the specimens in

Group G30F. The specimens in this group were designed to study the combined effect of the pitch of GFRP transverse reinforcement and the addition of steel fibre on the strength and ductility of GFRP bar reinforced HSC columns. The test matrix of the specimens is presented in Table 1. The dimensions and reinforcement configurations of the specimens are shown in Fig. 1.

The first specimen of each group was concentrically loaded. The second and the third specimens of each group were tested under eccentric axial load with eccentricities of 25 mm and 50 mm, respectively. The fourth specimen of each group was tested as beam under four-point loading in order to assess the pure flexural behaviour of the specimens. The loading conditions used in this study (including the 25 and 50 mm eccentric axial loads) were selected based on the testing facilities available at the University of Wollongong, Australia.

The specimens are labelled by a series of letters and numbers corresponding to the reinforcement type, configuration of the transverse reinforcement, loading conditions and the presence of the steel fibres (Table 1). The first letter in each specimen label refers to the reinforcement material, where “S” refers to steel reinforcement and “G” refers to GFRP reinforcement. The first number in each specimen label refers to the pitch of the helices. The second letter “E” and the second number in each specimen label stand for the loading condition: E0 refers to concentric load; E25 and E50 refer to axial loads with 25 mm and 50 mm eccentricity, respectively. The letter “B” refers to the four-point loading. The letter “F” stands for the presence of steel fibres. For example, Specimen G60E50F is reinforced longitudinally with GFRP bars (6#4) and transversely with GFRP helix with a pitch of 60 mm and tested under 50 mm eccentric axial load. Besides, 1% (by volume) steel fibres were added to the concrete mix of this column specimen.

2.2 Fabrication and Instrumentation of the tested specimens

Polyvinyl chloride (PVC) pipes with an inner diameter of 210 mm and a height of 800 mm were used as moulds to cast the specimens. Also, a wooden frame was used to hold the PVC pipes vertically and to prevent any movement during the casting of the specimens. Steel and GFRP reinforcement cages were assembled based on the reinforcement arrangement of each specimen. First, the longitudinal steel and GFRP bars were aligned vertically using two plastic templates with an outer diameter of 150 mm (Fig. 2a). The plastic templates have 12 holes distributed evenly around the perimeter of the templates: six of the holes fit the steel bars and the other six holes fit the GFRP bars. Afterwards, the longitudinal bars were assembled with the reinforcing helices using steel wire ties. The helices were adjusted to have the required pitch using two aluminium spacer jigs having groves at 30 mm centres (Fig. 2b). The groves were used for helices with 30 mm pitch and every second grove for helices with 60 mm pitch. Afterwards, the completed reinforcement cages (Fig. 2c) were placed inside the PVC moulds as shown in Fig. 2d. The steel and GFRP helices were fabricated to have an outer diameter of 170 mm. the concrete cover at the sides of the specimens was 20 mm. Also the steel and GFRP longitudinal bars were cut in lengths of 760 mm to ensure a constant concrete cover of 20 mm at the top and the bottom of the specimen.

All the specimens were cast on the same day at the laboratory of the School of Civil, Mining and Environmental Engineering, University of Wollongong, Australia. Ready mix high strength concrete with a maximum aggregate size of 10 mm provided by Hanson Company, Australia [19] was used. The HSC mix was poured directly from the truck mixer into the moulds prepared for Groups S60 and G60 specimens. For the rest of the specimens (Groups G60F and G30F specimens), steel fibres were added to the concrete mix using a concrete mixer. Firstly, the ready mix HSC was placed into the concrete mixer and then steel fibres

were added gradually and were dispersed uniformly using a sieve and were mixed for about 10 minutes. Afterwards, the concrete mix was poured into the moulds prepared for Group G60F and Group G30F specimens. The specimens were cast vertically in three stages. In every stage the concrete was internally vibrated to remove air voids and to ensure perfect compaction. During the following 28 days, the specimens were kept in the moulds and wet hessian was used to cure the specimens. Meanwhile, plastic sheets were used to cover the specimens and to maintain the moisture conditions.

2.3 Materials

The mix proportions of the high strength concrete (HSC) used in casting the specimens are presented in Table 2. The average 28-day compressive strength of the nonfibrous and fibrous concrete was 85 and 93 MPa, respectively. Two different sizes of steel bars were used in reinforcing Steel-HSC column specimens: 12 mm deformed steel bars N12 (longitudinal reinforcement), and 10 mm plain mild rounded steel bars R10 (transverse reinforcement). The mechanical properties of the N12 and R10 steel bars were determined according to AS 1391-2007 [20] as shown in Table 3. The GFRP bars and the GFRP helices used in reinforcing the GFRP bar reinforced specimens were sand-coated to improve the bond between the concrete and the embedded GFRP bars. Sand-coated #4 GFRP bars and sand-coated #3 GFRP helices were used as longitudinal reinforcement and transverse reinforcement, respectively. Both #4 GFRP longitudinal bars and #3 GFRP helices were provided by V-Rod Company, Australia [21]. In addition to the standard nominal diameter and the cross-sectional area of the GFRP bars provided by V-Rod company, the diameter and the cross-sectional area of the GFRP bars were also obtained using the immersion test according to ISO 104061-1:2015 [22], as presented in Table 4. The mechanical properties of the GFRP bars were determined according to ASTM D7205-11 [23] (Table 4). The steel fibres were provided by Ganzhou Daye

Metallic Fibres Company, China [24]. The steel fibres used in this study were straight in shape with brass coated surface. The steel fibres used were 13 mm in length (l) and 0.2 mm in diameter (d) with an aspect ratio (l/d) of 65. The ultimate tensile strength of the steel fibres was 2500 MPa [24].

2.4 Test Setup

Before testing, the top and the bottom parts of each specimen were externally wrapped with two layers of CFRP sheets to ensure that failure would occur at the mid-height of the specimen. The thickness and the width of CFRP sheets were 0.5 mm and 100 mm, respectively. Besides, the top and the bottom ends of each specimen were capped with a thin layer of high strength plaster to ensure a uniform distribution of the applied axial load during the test. All specimens were tested using the Denison testing machine having maximum compressive load capacity of 5 MN. Two loading heads fabricated at the University of Wollongong, Australia were used at the top and the bottom ends of each specimen to apply the axial loads at the required eccentricity. Each loading head consisted of circular steel plate and steel ball joint (Fig. 3a). For the eccentrically loaded specimens, the steel ball joints were used to transfer the applied load of the testing machine into 25 mm and 50 mm eccentric axial loads. For concentrically loaded specimens, the steel ball joints were not used and the applied load of the testing machine was transferred concentrically to the specimen directly through the circular steel plates. The circular steel plates were used to protect the ends of the specimens from the bearing failure (crushing of the ends of the specimens). The inner diameter of the circular steel plates was larger than the diameter of the tested specimens. Hence, the circular steel plates provided no restraint to the ends of the specimens during the test. For specimens tested as beams, a four-point loading system consisted of two steel circular rigs was used. The beam specimens were tested over a clear span of 700 mm and the

distance between the two-point loads was kept constant at 233.3 mm (Fig 3b). A typical test setup of the column and the beam specimens is shown in Fig. 4.

Two linear variable differential transducers (LVDTs) were attached vertically to the heads of the testing machine at two opposite corners to measure the axial deformation in the column specimens during the test. For specimens tested under eccentric loads, a laser triangulation was placed at the mid-height of the specimen to capture the lateral deformation. For specimens tested as beams, the laser triangulation was fixed underneath a hole at midspan of the testing rig to measure the midspan deflection of the tested specimens.

At the beginning of the test, the specimens were loaded (force controlled) at the rate of 2 kN/s to 100 kN and then the specimens were unloaded to 20 kN at the same rate to prevent any movements in the specimens that might occur during the test. Afterwards, the specimens were reloaded (displacement-control) at the rate of 0.005 mm/s until the failure (specimens experienced a substantial or total loss of the strength) of the specimens. The LVDTs and the laser triangulation were connected to a data logger to capture the data at every 2s. The applied axial load was recorded during the testing of the specimens via the internal load cell of the Denison testing machine.

3. Experimental Results and Analysis

3.1 General Observations

All column specimens were tested until failure. Two main points were noted in the load-deformation curve of the tested specimens: the first peak load ($P_{Peak\ 1}$) and the second peak load ($P_{Peak\ 2}$) as shown in Fig 5. The $P_{Peak\ 1}$ represents the maximum axial load sustained by the gross area of the specimen (the area of the reinforced concrete core plus the area of

concrete cover of the specimen, A_g), while the $P_{Peak\ 2}$ represents the maximum axial load sustained by the confined concrete core (A_{cc}) of the specimen after the loss of the concrete cover. Under concentric axial load, Specimens S60E0, G60E0 and G60E0F (reinforced transversely with steel or GFRP helices having a pitch of 60 mm) exhibited no second peak load due to the low confinement pressure provided by the transverse helices. In contrast, the well-confined Specimen G30E0F (reinforced transversely with GFRP helix having a pitch of 30 mm) exhibited a second peak load greater than the first peak load due to the adequate confinement pressure provided by the closely spaced GFRP helix. Besides, the 30 mm pitch GFRP helix in Specimen G30E0F contributed in delaying the crack propagation of the concrete core, restraining the GFRP longitudinal bars against buckling and allowing the specimen to fail progressively. On the other hand, all the eccentrically loaded specimens did not experience a well-defined second peak load, even specimens reinforced transversely with GFRP helices with a pitch of 30 mm due the effect of the combined loading (axial load and bending moment). The steel bar reinforced Specimen S60B tested under four-point loading also showed one peak load. However, all the GFRP bar reinforced specimens tested under four-point loading showed a second peak load due to the elastic linear stress-strain relationship and the high tensile strength of the GFRP bars and the GFRP helices compared to the steel bars and steel helices.

In general, the axial load-axial deformation and the axial load-lateral deformation behaviour of all tested specimens experienced three phases as shown in Fig. 5. The first phase (Phase 1) represents the ascending part of the load-deformation curve up to the first peak load ($P_{Peak\ 1}$). During this phase, the transverse reinforcement and the steel fibres had no or insignificant effects on the behaviour of the specimens. The second phase (Phase 2) represents the drop in the total axial load due to the spalling of the concrete cover after the $P_{Peak\ 1}$. The third phase

(Phase 3) represents the part of the load-deformation behaviour of the specimen that starts after the spalling of the concrete cover (activation of the transverse reinforcement) and ended with the total failure of the specimen. The load-deformation behaviour of the specimen during Phase 3 is governed by the type of the longitudinal and transverse reinforcement (steel or GFRP), the pitch of the transverse helices and the presence of the steel fibres.

The ductility (energy absorption capability) of the tested specimens was determined based on the area under the axial load-axial deformation curve of the specimens as outlined in ASTM C1018-97 [25]. Ductility index (I_5) was used as a measure for the ductility of the specimen (Fig. 5). The I_5 represents the ratio between the area ABDE (area under the axial load-axial deformation curve up to $3\delta_y$) to the area ABC (area under the axial load-axial deformation curve up to δ_y). Where δ_y is the yield deformation (the deformation at which the first crack occurs). The δ_y corresponds to the intersection point between the horizontal line drawn from the $P_{Peak\ 1}$ and the straight line passes the origin and the point representing the 0.75 times the $P_{Peak\ 1}$ [26], as shown in Fig. 5.

3.2 Failure modes of the tested specimens

The failure modes of the column specimens are shown in Fig 6. The reinforcement material (steel or GFRP), reinforcement arrangements, presence of the steel fibres and the loading condition were the main parameters that influenced the failure modes of the tested specimens. For concentrically loaded specimens, Specimens S60E0 and G60E0 exhibited spalling of the concrete cover immediately after reaching the $P_{Peak\ 1}$. The spalling of the concrete cover was mainly observed at the mid-height of the tested specimens and was attributed to the tendency of the concrete cover to buckle away from the concrete core when subjected to concentric axial load. Similar observations have been made in a number of experimental studies (Paultre

et al 1996 and Foster et al. 1998) [27-28]. Although Specimens G60E0F and G30E0F experienced cracks in the concrete cover at $P_{Peak\ 1}$, the concrete cover remained intact and attached to the concrete core throughout the test, even beyond the $P_{Peak\ 1}$. At the end of the test, the nonfibrous Specimens S60E0 and G60E0 experienced spalling of almost the entire concrete cover, whereas, only limited spalling of the concrete cover was observed in the fibrous Specimens G60E0F and G30E0F (Fig. 6). The failure of the Specimens S60E0 and G60E0 was initiated by the buckling of the longitudinal steel and GFRP bars, respectively, and failed by the rupture of the steel and GFRP helices, respectively. However, the failure of Specimens G60E0F and G30E0F was mainly due to the rupture of the GFRP helices which occurred after the crushing of the concrete core. Figs. 7a and 7b show the buckling of the steel and GFRP longitudinal bars and the rupture of the steel and GFRP helices of Specimens S60E0 and G60E0, respectively. Figs. 7c and 7d show the rupture of the GFRP helices of Specimens G60E0F and G30E0F at the end of the test after removing the concrete cover from the specimens by hand.

For eccentrically loaded specimens, the first sign of the failure of all specimens was the crushing of the concrete in the compression face of the specimens accompanied by transverse cracks in the tension face. This behaviour was due to the combined axial-flexural loading which was attributed to the change in the loading condition at the ends of the tested specimens from concentric axial load to 25 mm or 50 mm eccentric axial loads. Afterwards, the reference Specimens S60E25 and S60E50 exhibited buckling of the longitudinal steel bars located in the extreme compression layer. At the latter stage, the reference Specimens S60E25 and S60E50 failed due to the rupture of the longitudinal steel bars located in the extreme tension layer. On the other hand, Specimen G60E25 failed due to the rupture of the longitudinal GFRP bars and GFRP helices at the middle part of the compression face of the

specimen. It was observed that the rupture of the GFRP longitudinal bars located in the compression region of Specimen G60E25 could not be prevented due to the insufficient confinement provided by the GFRP helices. The failure of Specimen G60E50 was due to the rupture of the GFRP helices that occurred in the top third part of the specimen. Similarly, the failure of Specimens G60E25F, G30E25F, G60E50F and G30E50 was attributed to the rupture of the GFRP helices at the compression face of the specimens.

For specimens tested under four-point loading, the number and the width of the cracks experienced by the specimens at failure were depended mainly on the pitch of the transverse helices. Figure 8 presents a close-up view of the crushed region of the specimens tested as beams. Specimen G30BF exhibited a larger number of closely spaced cracks compared to Specimens S60B, G60B and G60BF. The crack width of the reference Specimen S60B at failure was about 22 mm which was about 13% smaller than the crack width of Specimen G60B and about 9% larger than the crack width of Specimen G60BF. The crack width of Specimen G30BF was about 5 mm. Similar to the eccentrically loaded specimens, the failure of all specimens tested under four-point loading started with the crushing of the concrete in the compression face at midspan of the specimens. Finally, the rupture of the longitudinal steel bars in the extreme tension layer at midspan resulted in the failure of Specimen S60B, whereas the rupture of the GFRP helices at midspan resulted in the total collapse of the Specimens G60B, G60BF and G30BF.

3.3 Behaviour of concentrically loaded specimens

Four specimens (the first specimen in each group) were tested under concentric axial load. Fig. 9 presents the axial load-axial deformation behaviour of the concentrically loaded specimens (S60E0, G60E0, G60E0F and G30E0F). The ascending part of the axial load-axial

deformation curves of Specimens S60E0, G60E0, G60E0F and G30E0F experienced similar patterns up to the first peak load $P_{Peak\ 1}$ and was mainly governed by the compressive strength of the concrete. This is because the lateral confinement provided by the transverse reinforcement (steel or GFRP helices) had little or no effect up to the first peak load due to the relatively low lateral dilation of the concrete. Similar observations were reported in Cusson and Paultre (1994) [10] and in Paultre et al. (2010) [29] for the steel bar reinforced HSC and SFHSC columns, respectively, and in Afifi et al. (2015) [15] for the GFRP bar reinforced concrete columns. The concrete cover of the concentrically loaded specimens did not crack until the specimens reached about 95% of the first peak load, where hairline cracks began to appear. With further loading, the hairline cracks widened and developed into vertical cracks. The maximum axial load sustained by the reference Specimen S60E0 was 2735 kN, which was about 0.5% higher than the maximum axial load of Specimen G60E0. Although the direct replacement of the steel reinforcement with the same amount of GFRP reinforcement resulted in a reduction in the maximum axial load carrying capacity of the columns [17, 30], Specimen G60E0F sustained about 2% higher axial load than the reference Specimen S60E0. The higher axial load sustained by Specimen G60E0F was attributed to the presence of the steel fibre which led to an increase in the compressive strength of the concrete by restraining the formation of the cracks and thereby increasing the axial load of the specimen. Specimen G30E0F sustained about 9% higher first peak load ($P_{Peak\ 1}$) than the reference Specimen S60E0.

After the first peak load ($P_{Peak\ 1}$), all specimens exhibited a drop in the axial load carrying capacity varied between 5 to 20 % of the load at the $P_{Peak\ 1}$ due to the spalling of the concrete cover. After the concrete cover spalled off, the concrete core experienced a lateral expansion, which activated the passive confining pressure of the steel and GFRP helices.

Afterwards, the concrete core started gaining strength whilst the concrete cover gradually spalled off for nonfibrous Specimens (S60E0 and G60E0) and disintegrates for the fibrous Specimens (G60E0F and G30E0F). Specimens S60E0 and G60E0 showed only $P_{Peak\ 1}$. Besides, the nonfibrous Specimens S60E0 and G60E0 experienced a significant loss of about 45% and 50% of their total axial load carrying capacity no longer after the spalling of the concrete due to the rupture of the steel and the GFRP helices, respectively. The rupture of the steel helix of Specimen S60E0 occurred at an axial deformation of about 4.4 mm, whereas the rupture of the GFRP helix of Specimen G60E0 occurred at an axial deformation of about 3.5 mm. Similarly, Specimen G60E0F showed only $P_{Peak\ 1}$, however, due to the presence of the steel fibers Specimen G60E0F showed a gradual decrease in the total axial load carrying capacity until the specimen failed at an axial deformation of about 7.8 mm. On the other hand, Specimen G30E0F reached a second peak load ($P_{Peak\ 2}$) of about 10% higher than the $P_{Peak\ 1}$. The second peak load ($P_{Peak\ 2}$) was an indication of the effectively combined confinement provided by both closely spaced GFRP helix and steel fibres. Specimen G30E0F failed due to the rupture of the GFRP helix at an axial deformation of about 12.6 mm.

The direct replacement of the steel reinforcement in (Specimen S60E0) by same amount of GFRP reinforcement in (Specimen G60E0) resulted in about 30% less ductility in the HSC column. Despite the brittle nature of both HSC and GFRP bars, the ductility of the Specimens G60E0F was only 10% lower than the reference specimen S60E0. Reducing the pitch of the GFRP helix from 60 mm to 30 mm in Specimen G30E0F resulted in about 38% higher ductility compared to Specimen S60E0, as shown in Table 5.

3.4 Behaviour of eccentrically loaded specimens

Eight specimens were tested under eccentric axial load: four specimens (S60E25, G60E25, G60E25F and G30E25F) were tested under 25 mm eccentric axial load and four specimens (S60E50, G60E50, G60E50F and G30E50F) were tested under 50 mm eccentric axial load. In general, all specimens tested under eccentric axial load showed one peak load (P_{Peak1}), even for specimens reinforced transversely with 30 mm GFRP helices. The decrease in the confinement efficiency of the GFRP helices in the GFRP bar reinforced specimens was attributed to the change in the loading condition at the ends of the specimens from concentric axial load to 25 mm or 50 mm eccentric axial loads.

Figure 10a presents the axial load versus axial deformation behaviour of the specimens tested under 25 mm eccentric axial load. The axial load versus lateral deformation behaviour of the specimens is also presented in Fig 10a. The ascending part of the load deformation curve of the specimens tested under 25 mm eccentric axial load was almost linear until the concrete cover started to spall off. This was an indication that the confinement provided by the transverse reinforcement and the steel fibres had insignificant effect on the axial load-axial deformation behaviour of the Specimens S60E25, G60E25, G60E25F and G30E25F up to the peak load. Similar observation was also reported in Paultre et al. [29] and in Hsu and Hsu [31]. Specimen S60E25 sustained maximum axial load of 1771 kN. The maximum axial load sustained by Specimen G60E25 was 1599, which was approximately 10% less than the axial load sustained by the reference Specimen S60E25. The ductility of Specimen G60E25 was only 3% lower than the ductility of the reference Specimen S60E25 due to the high tensile strength of the longitudinal GFRP bars which contributed in increasing the ductility of Specimen G60E25 as the load eccentricity increased to 25 mm. Similar to the concentrically loaded specimens, Specimen G60E25F sustained a slightly higher axial load (about 1.25%)

than the reference Specimen S60E25. The ductility of the Specimens G60E25F was about 20 and 24% higher than the ductility of Specimen S60E25 and G60E25, respectively. This was an indication on the effect of the steel fibres on the ductility of the specimens. Reducing the pitch of the GFRP helix in Specimens G30E25F did not increase the axial load sustained by the specimen. This is because the closely spaced GFRP helix caused a separation plane between the concrete core and the surrounding concrete cover, which led to early spalling (instability failure) of the concrete cover. Similar observations were also reported in Razvi and Saatcioglu [32] and in Pessiki and Pieroni [33]. However, reducing the pitch of the GFRP helix in Specimen G30E25F enhanced the post-peak behaviour, where specimen G30E25F sustained an almost constant axial load of about 94% of the maximum axial load until failure. Moreover, the ductility of Specimen G30E25F increased by about 40%, 44% and 17% compared to Specimens S60E25, G60E25 and G60E25F, respectively.

In comparison with the concentrically loaded specimens, GFRP bar reinforced HSC specimens in Group G60 experienced a reduction of 41% in the axial load carrying capacity due to increasing the eccentricity of the applied load from zero (concentric axial load) to 25 mm eccentric axial load. This reduction was about 6% greater than the reduction in the axial load carrying capacity experienced by the steel bar reinforced HSC specimens in Group S60. However, the reduction in the axial load carrying capacity of the GFRP bar reinforced SFHSC specimens in group G60F was almost similar to the reduction in the axial load carrying capacity of the steel bar reinforced HSC specimens (Group S60).

Figure 10b shows the axial load-axial deformation and axial load-lateral deformation behaviour of the specimens tested under 50 mm eccentric axial load. Similar to the specimens tested under 25 mm eccentric axial load, the behaviour of Specimens S60E50, G60E50,

G60E50F and G30E50F throughout the ascending part of their axial load-axial deformation curves was slightly influenced by the confinement provided by the helices and the steel fibres. The axial load sustained by the reference Specimen S60E50 was 1158 kN. Specimen G60E50 sustained about 12% lower axial load than the reference Specimen S60E50. However, Specimen G60E50 achieved about 11% higher ductility than Specimen S60E50, as the load eccentricity increased to 50 mm. Specimens G60E50F achieved about 0.6% and 14% higher axial load and 25% and 13% higher ductility in comparison with the axial load and the ductility of the Specimen S60E50 and G60E50, respectively. Similar to Specimen G30E25F, Specimen G30E50F achieved 10% lower axial load compared to the reference Specimen S60E50 due to the early spalling of the concrete cover. However, due to the combined effect of the closely spaced transverse GFRP helix and the steel fibres, the ductility of the Specimen G30E50F was about 35% higher than the reference Specimen S60E50.

It was found that the reduction in the axial load carrying capacity experienced by the steel bar reinforced HSC specimens of Group S60 due to increasing the loading eccentricity to 50 mm was about 58%, whereas the reduction in the axial load carrying capacity exhibited by the GFRP bar reinforced HSC specimens in Groups G60, under the same loading eccentricity, was about 62%. The GFRP bar reinforced SFHSC specimens in Group G60F and Group G30F experienced about 58% and 65% reduction in the axial load carrying capacity, respectively.

It was observed that under concentric axial load, the axial load carrying capacity of the GFRP bar reinforced HSC Specimen G60E0 in Group G60 was almost similar to the axial load of the reference Specimen S60E0 in Group S60, which was reinforced with the same amount of steel longitudinal bars and helices. However, the efficiency of the GFRP bar reinforced HSC

specimens in sustaining axial load decreased with increasing the loading eccentricity, where under 25 mm and 50 mm eccentric axial load, Specimens G60E25 and G60E50 in Group G60 sustained 10 and 12% lower axial load compared to the reference steel bar reinforced HSC Specimens S60E25 and S60E50 in Group S60. On the other hand, the axial load carrying capacity of the specimen in Group G60F was slightly greater than the axial load carrying capacity of the specimen in Group S60 under concentric axial loads. Under eccentric axial loads (combined axial load and bending moment), the specimens in Group G60F experienced a reduction in the axial load carrying capacity due to the combined stresses in the cross-section of the specimens. However, the axial load carrying capacity of the eccentrically loaded specimens in Group G60F was still greater than the axial load carrying capacity of the eccentrically loaded specimens in Group S60. Table 6 reports the experimental results (peak loads, corresponding deformations and ductility) of the specimens tested under 25 mm and 50 mm eccentric axial loads.

3.5 Behaviour of specimens tested under four-point loading

Four specimens (S60B, G60B, G60BF and G30BF) were tested as beam under four-point loading to explore the behaviour of the specimens under pure flexural load. Fig. 11 shows the load-midspan deflection behaviour of the tested specimens. Table 7 presents the experimental results of the tested specimens. Two layers of CFRP sheets were used to wrap the shear span of the GFRP bar reinforced Specimens G60B, G60BF and G30BF to reduce the effect of the shear-induced deflection at midspan and to prevent the shear failure, which might occur because of the small span-to-depth ratio of the tested specimens as well as the high tensile strength of the longitudinal GFRP bars. The shear span of the reference Specimen S60B was also wrapped with CFRP sheets to achieve a consistent comparison. The steel bar reinforced specimen S60B experienced only first peak load, whereas all the GFRP bar reinforced

specimens experienced two peak loads. All specimens tested under four-point loading experienced a linear ascending behaviour up to the first peak load. The reference Specimen S60B sustained load of 309 kN at the first peak. Specimens G60B, G60BF and G30BF sustained about 4, 17 and 19% higher load, respectively, than the reference specimen S60B. Afterwards, Specimens S60B and G60B experienced a drop in the load carrying capacity of about 13% and 6%, respectively, due to the crushing of the concrete cover at the compression face of the specimens. However, Specimens G60BF and G30BF experienced no drop in the load carrying capacity due to the presence of the steel fibres. In the post-peak part of the load-midspan deflection behaviour, the reference Specimen S60B showed no clear second peak load, as mentioned above, and carried an almost constant load of about 75% of the first peak load until failure. But, Specimens G60B, G60BF and G30BF showed a linear ascending post-peak behaviour until failure reaching a second peak load of about 61%, 65% and 88% higher than the first peak load, respectively. The ductility of Specimens G60BF and G30BF was about 12, 9% and 40% higher than the ductility of the reference specimen S60B, respectively.

4. Axial load-bending moment interaction diagrams

For designing of the concrete members subjected to different loading conditions (concentric, eccentric and flexural loads), interaction diagrams are plotted for the tested specimens. In this study, four points were used to establish the axial load-bending moment (P - M) diagrams for the experimentally tested specimens in the Groups S60, G60, G60F and G30F. The first point on each P - M diagram represents the concentrically loaded specimens, the second and the third points represent the specimens tested under 25 mm and 50 mm eccentric axial loads, respectively, whereas the fourth point represents the specimens tested as beam under four-point loading. As most specimens in this study showed no second peak load, the first peak load will be considered the maximum axial load carrying capacity to use for the design

purposes. Consequently, the first peak load ($P_{Peak\ 1}$) experienced by the specimens was used in establishing the interaction diagrams. For the concentrically loaded specimens, the value of the bending moment was taken equal to zero. The bending moment, including the secondary moment for specimens tested under 25 and 50 mm eccentric axial loads was calculated using Eq. 1, while the bending moment of the specimen tested under four-point loading were calculated using Eq. 2.

$$M = P_{Peak\ 1} (e + \delta) \quad (1)$$

$$M = P_{Peak\ 1} L/6 \quad (2)$$

where $P_{Peak\ 1}$ = the first peak load of the tested specimens; δ = the corresponding lateral deformation; e = the load eccentricity and L = the length between the supports of the beam specimens (Fig. 3).

The experimental axial load-bending moment (P – M) diagrams of the Groups S60, G60, G60F and G30F are shown in Fig. 12. It was observed that the axial load and the corresponding bending moment achieved by steel bar reinforced specimens of Group S60 under concentric and eccentric axial load were higher than the axial load and corresponding bending moment of the specimens reinforced with same amount of GFRP reinforcement in Group G60. This is because the elastic modulus of the GFRP bars is lower than the elastic modulus of the steel bars. However, Group G60F specimens experienced higher axial load and moment capacity under concentric, eccentric and flexural loads compared to the Group S60 specimens, which was an indication on the effect of the addition of steel fibres in HSC. The axial load-bending moment diagram of Group G30F was lower than Groups S60 and G60F under eccentric axial load because of the early spalling of the concrete cover that

resulted in lower than expected axial load carrying capacity. However, Specimens G30E0F and G30BF experienced higher axial load and higher bending moment capacity under concentric and pure flexural loads, respectively compared to Groups S60, G60 and G60F specimens.

The analytical axial load-bending moment interaction diagrams for the GFRP bar reinforced HSC and SFHSC specimens were established to complement the experimental results. The analytical P - M interaction diagrams of the GFRP bar reinforced specimens were developed based on the strain compatibility and the force equilibrium principles adopted for the conventional steel bar reinforced specimens. The CSA A23.3-2014 [34] equivalent rectangular stress block, developed for the steel bar reinforced concrete specimens, was used to predict the axial load carrying capacity and the corresponding bending moment resistances for the GFRP bar reinforced specimens. Two parameters α_1 and β_1 were used to define the CSA A23.3-2014 [34] equivalent rectangular stress block. The parameters α_1 and β_1 were calculated using Eq. (3) and Eq. (4), respectively. The GFRP bars were assumed to have a linear elastic stress-strain relationship. Besides, the limiting strain ε_u at the extreme concrete compression layer was taken equal to 0.0035, as prescribed in the CSA A23.3-2014 [34].

$$\alpha_1 = 0.85 - 0.0015f'_c \geq 0.67 \quad (3)$$

$$\beta_1 = 0.97 - 0.0025f'_c \geq 0.67 \quad (4)$$

Figure 13 compares the analytical interaction diagrams obtained using the CSA A23.3-2014 [34] equivalent rectangular stress block with the experimental data. The comparison indicates that using the equivalent rectangular stress block defined in the CSA A23.3-2014 [34] yielded

reasonable conservative correlations between the computed and the experimentally obtained results. The conservative predictions were attributed to the conservative parameters of the CSA A23.3-2014 [34] equivalent rectangular stress block. Similar observations were also reported in Canbay et al. [12] and in Ozbakkaloglu and Saatcioglu [35] for steel bar reinforced concrete columns under concentric and eccentric axial loads. This was an indication that the response of the GFRP bar reinforced concrete specimens under different loading condition can be reasonably estimated using the same methods adopted for the steel bar reinforced concrete specimens.

5. Conclusions

In this study, 16 specimens were tested under different loading conditions: four specimens under concentric axial load, eight specimens under eccentric axial load and four specimens under four-point loading. The behaviour of the GFRP bar reinforced HSC and SFHSC specimens in regarding to the axial load carrying capacity, failure modes and ductility. Based on the test findings, the following conclusion could be drawn:

1. For HSC specimens, the direct replacement of the longitudinal and transverse steel reinforcement with the same amount of GFRP reinforcement did not influence the axial load carrying capacity of the specimen under concentric axial load. However, GFRP bar reinforced HSC specimens experienced about 10% and 12% lower axial load carrying capacity than the steel bar reinforced HSC specimens as a result of changing the loading condition from concentric axial load to 25 and 50 mm eccentric axial load, respectively.
2. For SFHSC, it was observed that Group G60F specimens sustained similar or slightly greater axial load than Group S60 specimens under concentric axial loads. The specimens in Group G60F experienced a reduction in the axial load carrying capacity under eccentric axial load (combined axial load and bending moment) due to the combined stresses in the

cross-section of the specimens. However, the axial load carrying capacity of the eccentrically loaded specimens in Group G60F was still greater than the axial load carrying capacity of the eccentrically loaded specimens in Group S60.

3. Under concentric axial load, only Specimen G30E0F (reinforced transversely with 30 mm GFRP helix) experienced a second peak load, which was higher than the first peak load. However, all the eccentrically loaded GFRP bar reinforced specimen showed no second peak load even specimens reinforced transversely with 30 mm GFRP helices due to the change in the loading condition from concentric axial load to 25 mm or 50 mm eccentric axial loads. This was an indication that the efficiency of the GFRP transverse reinforcement in confining HSC columns decreases with an increase in the eccentricity of the applied axial load.

4. The failure of the steel bar reinforced specimens was initiated by the buckling of the longitudinal steel bars and then the rupture of the longitudinal steel bars or the steel helix resulted in the total failure of the specimens. The failure of GFRP-HSC was controlled by the rupture of both the longitudinal GFRP bars and the GFRP helices, whereas the failure of the GFRP-SFHSC specimens was mainly attributed to the rupture of the GFRP helices.

5. Under concentric axial load, replacing the steel reinforcement with the same amount of GFRP reinforcement in HSC specimens resulted in about 30% reduction in the ductility of the specimen. However, under the same loading condition (concentric axial load), GFRP bar reinforced SFHSC specimens experienced almost similar ductility compared to the conventional steel bar reinforced HSC specimen.

6. Despite the non-ductile behaviour of HSC and GFRP bars, reducing the pitch of the GFRP helices with the addition of 1% by volume steel fibres resulted in about 35-40% higher ductility of Group G30F specimens compared to the reference specimens of group S60

under different loading conditions. However, closely spaced GFRP helices might lead to an early spalling of the concrete cover.

7. The axial carrying capacity and the bending moment resistances of the GFRP bar reinforced concrete specimens can be reasonably calculated using the equivalent rectangular stress block defined in the CSA A23.3-2014 [34]. This indicates that the response of the GFRP bar reinforced concrete specimens under different loading condition can be predicted using the same analytical procedures used for the steel bar reinforced concrete specimens.

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Table 1: Test Matrix

Group	Specimen	Longitudinal reinforcement	Transverse reinforcement	Steel fibres ratio, v_f (%)	Loading eccentricity (mm)
S60	S60E0 S60E25 S60E50 S60B	Steel 6N12	Steel R10 @ 60-mm Pitch	----	0 25 50 Four-point loading
G60	G60E0 G60E25 G60E50 G60B	GFRP 6 #4	GFRP #3 @ 60-mm Pitch	----	0 25 50 Four-point loading
G60F	G60E0 G60E25F G60E50F G60BF	GFRP 6 #4	GFRP #3 @ 60-mm Pitch	1.0	0 25 50 Four-point loading
G30F	G30E0F G30E25F G30E50F G30BF	GFRP 6 #4	GFRP #3 @ 30-mm Pitch	1.0	0 25 50 Four-point loading

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Table 2: Mix proportions of the high strength concrete (HSC) [19]

Material	Quantity
Cement (kg/m ³)	576
Fine aggregate (kg/m ³)	540
Coarse aggregate (kg/m ³)*	990
Silica fume (kg/m ³)	30
Fly ash (kg/m ³)	64
Water (kg/m ³)	197
Mid-range water reducing admixture (l/m ³)	6

* Maximum size of the aggregate used was 10 mm

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Table 3: Mechanical properties of the steel bars

Bar size	Diameter of the bar (mm)	Area of the bar (mm ²)	Yield tensile strength (MPa)	Strain corresponding to tensile strength (mm/mm)	Elastic modulus (GPa)
R10	10	78.5	420	0.0022	190
N12	12	113	550	0.0027	200

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Table 4: Mechanical properties of the GFRP bars

Bar size	Diameter of the bar (mm)		Cross-sectional area of the bar (mm ²)		Tensile strength (MPa)	Tensile rupture strain (mm/mm)	Tensile modulus (GPa)
	Nominal	By Immersion test ^a	Nominal	By Immersion test ^b			
#3	9.5	11	70.9	95	1770	0.0231	76.8
#4	12.7	14.5	126.7	165	1548	0.0228	67.8

^a Determined in accordance with the immersion test (ISO 2015) [22]

^b Calculated based on the diameter of the GFRP bars obtained from the immersion test

842 Table 5: Experimental results of the specimens tested under concentric axial load

Specimens	S60E0	G60E0	G60E0F	G30E0F
Yield load (kN)	2596	2603	2624	2844
Axial deformation at yield load (mm)	2.7	2.9	3.1	4.2
First peak load (kN)	2735	2721	2791	2983
Axial deformation at first peak load (mm)	2.9	3.1	3.5	4.5
Second peak load (kN)	----	----	----	3272
Axial deformation at second peak load (mm)	----	----	----	12.6
Ductility	3.7	2.6	3.3	5.1

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Table 6: Experimental results of the specimens tested under eccentric axial load

Specimens	Loaded under 25 mm eccentric axial load				Loaded under 50 mm eccentric axial load			
	S60E25	G60E25	G60E25F	G30E25F	S60E50	G60E50	G60E50F	G30E5F
Yield load (kN)	1728	1551	1728	1626	1143	990	1121	994
Axial deformation at yield load (mm)	2.7	2.5	2.8	2.7	2.8	2.5	2.8	2.4
First peak load (kN)	1771	1599	1793	1686	1158	1023	1165	1048
Axial deformation at first peak load (mm)	2.8	2.7	3.1	2.8	2.9	2.6	3.0	2.6
Second peak load (kN)	----	----	----	----	----	----	----	----
Axial deformation at second peak load (mm)	----	----	----	----	----	----	----	----
Ductility	3.5	3.4	4.2	4.9	3.4	3.8	4.3	4.6

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871 Table 7: Experimental results of the specimens tested under four-point loading

Specimen	S60B	G60B	G60BF	G30BF
First peak load (kN)	309	321	361	369
Axial deformation at first peak load (mm)	7.5	6.8	7.3	7.2
Second peak load (kN)	----	517	597	696
Axial deformation at second peak load (mm)	----	16.9	16.7	18.9
Ductility	4.9	5.5	5.3	7.0

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Fig. 1: Geometry and reinforcement details of the tested specimens

Fig. 2: Fabrication of the tested specimens: (a) Alignment of the longitudinal bars; (b) assembling of the reinforcement cages; (c) completed reinforcement cages and (d) completed formwork of the specimens.

(a)

(b)

Fig. 3: Specimen test setup: (a) testing of the column specimens and (b) testing of the beam specimens

Fig. 4: Typical test setup of: (a) column specimen and (b) beam specimen

Fig. 5: General behaviour and ductility calculations of the tested specimens

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Fig. 6: Failure Modes of the column specimens

Fig. 7: Close-up view of the buckling of the steel and GFRP longitudinal bars and the rupture of the steel and GFRP helices

Fig. 8: Close-up view of the crashed region of the beam specimens

Fig. 9: Axial load-axial deformation behaviour of the specimens tested under concentric axial load

Fig. 10: Axial load-axial deformation and axial load-lateral deformation behaviour of the specimens tested under: (a) 25 mm eccentric axial load and (b) 50 mm eccentric axial load

Fig. 11: Load-midspan deflection behaviour of the specimens tested under four-point loading

Fig. 12: Experimental axial load-bending moment (P - M) interaction diagrams

Fig. 13: Comparison of the experimental and analytical axial load-bending moment ($P-M$) interaction diagrams: (a) Group S60; (b) Group G60; (c) Group G60F and (d) Group G30F