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Keywords

Ambient-cured, Columns, Geopolymer concrete, GFRP, Loading conditions

Experimental study of GFRP-reinforced geopolymer concrete columns under different loading conditions

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Abstract

This study discusses the behavior of ambient-cured circular geopolymer concrete specimens tested under different loading conditions. Twelve specimens were cast and tested to investigate the influence of the type of the reinforcement, that is, steel and glass fiber-reinforced polymer (GFRP), pitch of the transverse reinforcement and loading conditions (concentric axial load, 15 and 35 mm eccentric load, and four-point bending). The axial load carrying capacity, confinement efficiency and deformability of the specimens decreased for the replacement of the steel reinforcement with the same amount of GFRP reinforcement under concentric and eccentric loads. However, the deformability of the GFRP reinforced concrete (RC) specimen was higher than the deformability of its steel counterpart specimen under four-point bending. Overall, the reduction in the pitch of the GFRP helices in the GFRP-RC specimens resulted in a significant improvement in the confinement efficiency, deformability, post-peak behavior and the axial load-bending moment capacity of the specimens under different loading conditions. Moreover, theoretical equations based on the confinement pressure of transverse reinforcement provide accurate predictions of the theoretical load carrying capacity of the columns reinforced with GFRP bars and helices.

Keywords: Geopolymer concrete; ambient-cured; columns; glass fiber-reinforced polymer (GFRP); loading conditions

Introduction

Concrete is widely used as an important construction material for the development of infrastructure, with an estimated yearly consumption of 30 billion tonnes (Monteiro et al. 2017). The use of ordinary Portland cement (OPC) as a binder material in concrete is a traditional method to produce concrete. Therefore, the production of a huge volume of OPC is a substantial industrial activity, which contributes to 5-7% of CO₂ in the overall greenhouse gas emissions (Miller et al. 2016).

Geopolymer concrete is a new form of concrete produced with aluminosilicate binders rather than the cement binder. The production of geopolymer concrete involves a process of geopolymerization of industrial waste materials that are rich in alumina and silica (Davidovits 1991). The synthesis of commercially available binders, i.e., ground granulated blast furnace slag (GGBFS) and fly ash (FA), is carried out in the presence of an alkaline solution (sodium-hydroxide and sodium silicate). The studies reported that the production of geopolymer based pastes and concrete significantly reduced greenhouse gas emissions (McLellan et al. 2011; Turner et al. 2013; Nawaz et al. 2020). Several research studies indicated that geopolymer concrete is a durable construction material with mechanical properties as good as the OPC concrete. However, some of the challenges in the use of geopolymer concrete include the need for special handling, high cost of alkaline solutions and loss of uniformity of the mix. Although geopolymer concrete lacks in ductility, it offers higher fire resistance with low susceptibility to shrinkage and creep than OPC concrete. In addition, geopolymer concrete possesses comparable or even higher chemical resistance against chloride and sulphate attacks compared to OPC concrete (Sofi et al. 2007; Duxson et al. 2007; Kong and Sanjayan

2010). It is found that heat curing significantly improves the compressive strength of geopolymer concrete at an early age (Sindhunata et al. 2006). However, heat curing restricts the use of geopolymer concrete for precast structural members. The development of ambient-cured geopolymer concrete is a major breakthrough in the use of geopolymer concrete, as it offers economical solution and simplification for in-situ constructions (Hadi et al. 2017; Ahmad et al. 2021a).

The service life of reinforced concrete (RC) structures is currently a major concern of the construction industry. One of the main factors affecting the performance of RC structures is the corrosion and degradation of steel reinforcement. Therefore, the use of fiber-reinforced polymer (FRP) bars instead of conventional steel bars has become an innovative solution, particularly for marine structures (Hadi et al. 2020; Ali et al. 2021; Ahmad et al. 2021b). The FRP bars are lightweight (20–25% of the density of steel), corrosion resistant, and non-electromagnetic. In addition, FRP bars have high tensile strength and fatigue resistance compared to the conventional steel bars, making FRP bars suitable alternatives to steel bars in RC structures (Cheng and Karbhari 2006).

A number of studies investigated the shear and the flexural behavior of OPC and geopolymer concrete structural members reinforced with FRP composites (Barris et al. 2009; El-Nemr et al. 2013; Maranan et al. 2018; Maranan et al. 2019). The FRP bars have lower compressive strength compared to their tensile strength (Afifi et al. 2014). The American Concrete Institute guide ACI 440.1R-15 (ACI 2015) provides no guidelines for the use of the longitudinal FRP bars in compression members. The Canadian Standards CSA S806-12:R17 (CSA 2017) ignores the contribution of FRP bars in compression. However, the recent Canadian highway bridge design code CSA S6-19 (CSA 2019) permits to account the contribution of FRP bars in compression.

The behavior of columns reinforced with FRP bars has also been investigated. Alsayed et al. (1999) investigated the behavior of rectangular RC columns for the replacement of steel reinforcement with the glass fiber-reinforced polymer (GFRP) reinforcement. It was found that the axial load carrying capacity of GFRP-RC columns was 13% lower than that of the steel-RC columns. Choo et al. (2006) reported that neglecting the contribution of FRP bars under compression was a conservative approach. Tobbi et al. (2012) found that the contribution of GFRP bars in the axial load carrying capacity of the columns was about 10%. Hadhood et al. (2017) considered the contribution of GFRP bars under compression in the section analysis and developed load-moment characteristics of eccentrically loaded circular concrete columns reinforced with GFRP bars and helices, which reasonably agreed with the experimental results. Further, it was found that the use of the GFRP longitudinal reinforcement ratio up to 3.3% enhanced the stiffness of the columns (Hadhood et al. 2018). The deformability and the post-peak load-deformation behavior of the GFRP-RC columns improved significantly for the reduction in the pitch of the GFRP helices (Pantelides et al. 2013, Hadi et al. 2016, Hasan et al. 2017). Guérin et al. (2018) reported that the level of the axial load eccentricity significantly influenced the strength, failure mode and overall behavior of the GFRP-RC columns. The effect of slenderness ratio of GFRP-RC columns under concentric and eccentric loads was investigated in Abdelazim et al. (2020). The study recommended limiting the maximum slenderness ratio to 18 for short GFRP-RC columns. Elmesalami et al. (2021) investigated the behavior of FRP-RC columns under concentric and eccentric loads. It was found that the effect of reinforcement ratio on the load carrying capacity is more pronounced on the eccentrically loaded columns as compared to the concentrically loaded columns.

Relatively fewer studies investigated the behavior of ambient cured geopolymer concrete columns. Sumajouw et al. (2007) tested twelve geopolymer concrete columns of 175×175

mm cross-section and 1500 mm height reinforced with steel bars. The test results of the study agreed with the theoretical results calculated using the design provisions of Australian Standard AS 3600-01 (AS 2001) and American design code ACI 318-02 (ACI 2002). Sarker (2009) found that the analysis of steel-RC geopolymer concrete columns can be performed using the analytical method for OPC concrete columns with the appropriate stress-strain relationship of the geopolymer concrete. Farhan et al. (2018a) tested sixteen circular steel-RC geopolymer concrete columns under axial and flexural loads. It was found that the deformability of the columns increased significantly with the addition of steel fibers.

In comparison with Aramid-FRP (AFRP) and Carbon-FRP (CFRP), Glass-FRP (GFRP) bars possess better mechanical characteristics, and they are economical and more desirable in the construction industry (Hu et al. 2020). A detailed review of the literature reveals that GFRP bars and geopolymer concrete are suitable materials for columns, particularly for marine structures and structures that require electromagnetic transparency. The use of GFRP bars leads to an overall reduction in the weight of the structure. The combination of GFRP bars and geopolymer concrete is an excellent option that can be used as an alternative of steel bars and OPC concrete for the columns. Maranan et al. (2016) investigated the behavior of ambient-cured circular geopolymer concrete columns reinforced with GFRP bars. However, the experimental testing presented in Maranan et al. (2016) was limited to the concentrically loaded columns. In reality, construction errors and moving loads can cause eccentric and flexural loads in the columns. In fact, most columns are exposed to a combination of axial loading and bending moment. Hence, this study investigates the effect of the concentric axial load, 15 and 35 mm eccentric loads and flexural load on the behavior of ambient-cured circular geopolymer concrete columns reinforced with GFRP bars and helices. The eccentricity to height ratio was 0.023 and 0.055 for 15 and 35 mm eccentrically loaded specimens, respectively. In addition, the effect of replacing steel reinforcement with the same

amount of GFRP reinforcement, the pitch of GFRP helices and the loading condition were investigated. Moreover, the theoretical axial load carrying capacities of the columns were calculated using various confinement based prediction equations proposed in the literature.

Experimental program

Details of the test matrix

The experimental program comprised testing of twelve ambient-cured circular geopolymer concrete column specimens. The specimens were distributed into three groups with four specimens in each group based on the type and the arrangement of the reinforcement. Three specimens in each group were of 160 mm diameter and 640 mm height. The fourth specimen in each group was of 160 mm diameter and 1500 mm height. The first group (S40) included the specimens reinforced longitudinally with six N10 (diameter = 10 mm) deformed steel bars and transversely with R8 (diameter = 8 mm) plain steel helices with 40 mm pitch. The second and third groups (G40, G75) included the specimens reinforced longitudinally with six G10 (diameter = 10 mm) GFRP bars. The specimens in the second and the third groups were reinforced transversely with G8 (diameter = 8 mm) GFRP helices with 40 mm and 75 mm pitch, respectively. The specimens in the first group (S40) were used as reference specimens. The specimens in the second group (G40) were prepared to study the influence of the type of reinforcement (steel and GFRP) on the behavior of circular geopolymer concrete columns. The specimens in the third group (G75) were prepared to investigate the effect of increasing the pitch of the transverse GFRP helices on the behavior of ambient-cured circular geopolymer concrete columns. The dimensions of the column specimens tested under concentric and eccentric loading were chosen to be suitable to the condition and capacity of the available testing equipment in the laboratory. In addition, the vertical support with l/h ratio greater than or equal to 2.5 is considered as a short column according to the provisions

of the Canadian codes CSA S806-12:R17 (CSA 2017) and CSA S6-19 (CSA 2019). The design of the helical reinforcement for the tested GFRP-RC specimens meets the minimum bar diameter, pitch and volumetric ratio of FRP helical reinforcement requirements outlined in CSA S806-12:R17 (CSA 2017). It is noted that the experimental program was designed before the release of CSA S6-19 (CSA 2019).

Table 1 shows the test matrix used in this study. The dimensions and reinforcement details of the specimens are presented in Fig. 1. The first specimen of each group was tested under concentric axial load. The second and the third specimens of each group were tested under 15 and 35 mm eccentric loads, respectively. The axial load eccentricities of 15 mm ($e/D = 0.09$) and 35 mm ($e/D = 0.22$) were selected considering the diameter of the columns and the available test facility in the structural engineering laboratory at the University of Wollongong, Australia. The letters “ e ” and “ D ” refer to the loading eccentricity and diameter of the specimen, respectively. The lower bound of the load eccentricity ($e/D = 0.09$) was taken approximately half of the upper bound eccentricity ($e/D = 0.22$). The fourth specimen of each group was tested as a beam under four-point bending to investigate the pure flexural behavior of the specimens. The size ($D \times H = 160 \times 1500$ mm) of the specimens tested under four-point bending was selected to minimize the effect of shear induced deflection at mid span.

The specimens were identified by specimen labels. Each label consists of letters and numbers based on the type of reinforcement, arrangement of the transverse helices and loading conditions (Table 1). The first part of each label represents the reinforcement material, where “S” and “G” refer to steel and GFRP reinforcement, respectively. Afterwards, the second part of each label refers to the pitch of the helices. The numbers 40 and 75 represent 40 mm and 75 mm pitch, respectively. The third part represents the loading condition, i.e., E0 refers to concentric axial load, E15 and E35 refer to 15 and 35 mm

eccentric loads, respectively, and B refers to four-point bending. For example, Specimen G40E35 represents the geopolymer concrete specimen reinforced longitudinally with 6G10 bars and transversely with GFRP helices at 40 mm pitch and tested under 35 mm eccentric load.

Materials and mix proportions of geopolymer concrete

All the specimens were cast using geopolymer concrete with a design compressive strength of 50 MPa at 28 days. The geopolymer concrete mix was prepared using ground granulated blast furnace slag (GGBFS), fly ash (FA), alkaline solution, coarse aggregate (up to 10 mm size), fine aggregates (river sand), water and superplasticizer. The GGBFS and FA were supplied by the local suppliers (ASA 2019; BORAL 2019). The alkaline solution of 14 molarity was prepared by mixing sodium hydroxide (NaOH) and sodium silicate (Na_2SiO_3) solutions. The Na_2SiO_3 solution was composed of $\text{Na}_2\text{O} = 14.7\%$ and $\text{SiO}_2 = 29.4\%$ with a modulus ratio (Ms) of 2 ($\text{SiO}_2/\text{Na}_2\text{O}$), as per the manufacturer (PQ 2019). Locally available superplasticizer (Viscocrete-10) was used to enhance the workability of the fresh geopolymer concrete mix (Sika 2019). The mix proportions used in this study were similar to those proposed in a previous study conducted by Ali et al. (2019) for ambient-cured geopolymer concrete (Table 2). In Ali et al. (2019), trial mixes were prepared and tested to modify the mix design of Hadi et al. (2017) for desired workability and compressive strength.

Preliminary testing

The preliminary testing included the chemical analysis of GGBFS and FA, the tensile and compressive strength of steel and GFRP reinforcement and the compressive strength of ambient cured geopolymer concrete. The chemical composition of GGBFS and FA was determined using X-Ray Fluorescent (XRF) test. The GGBFS contained 42.7% calcium

oxide (CaO) by mass and the FA contained 83.4% of alumino-silicates ($\text{Al}_2\text{O}_3\cdot\text{SiO}_2$) by mass. The fly ash was classified as Class F in accordance with ASTM C618-19 (ASTM 2019). The test results of the chemical analysis of GGBFS and FA are presented in Table 3.

The mechanical properties of both N10 and R8 straight steel bars were determined in accordance with AS 1391-17 (AS 2017). The yield tensile strength and elastic tensile modulus of N10 deformed steel bars were 552 MPa and 193 GPa, respectively. The yield tensile strength and elastic tensile modulus of R8 plain steel bars were 520 MPa and 190 GPa, respectively. The surface of the GFRP bars was ribbed for an effective bonding between the bars and the surrounding concrete. The root diameters of G10 and G8 GFRP bars were 9.2 mm and 7.4 mm, respectively, as per the specifications provided by the manufacturer (Pultron 2019). The root diameter of the GFRP bars refers to the diameter of an unprojected surface between the ribs. The diameter of G10 and G8 GFRP bars obtained using the immersion test was 9.8 mm and 7.7 mm, in accordance with ISO 10406-1-15 (ISO 2015). The mechanical properties of both G10 and G8 straight GFRP bars were determined based on the root diameters in accordance with ASTM D7205-16 (ASTM 2016) and ASTM D695-15 (ASTM 2015). The tensile strength and elastic tensile modulus of G10 GFRP bars were 1263 MPa and 63.8 GPa, respectively. The tensile strength and elastic tensile modulus of straight G8 GFRP bars were 1162 MPa and 61.9 GPa, respectively. The compressive strengths of G10 and G8 GFRP bar specimens were determined using the slenderness ratios of 16:1 and 13:1, respectively. The slenderness ratios of the tested GFRP bar specimens were within the permissible range specified in ASTM D695-15 (ASTM 2015). The compressive strengths of G10 and G8 GFRP bars were 827 MPa and 907 MPa, respectively. Table 4 shows the average properties of steel and GFRP reinforcement. The typical tensile stress-strain behavior of steel and GFRP longitudinal bars are presented in Fig. 2. Figure 3a and 3b show the GFRP bars and GFRP helices used in this study.

The compressive strength of ambient-cured geopolymer concrete was determined by testing three cylinder (100×200 mm) specimens in accordance with AS 1012.9-14 (AS 2014) and the average compressive strengths are reported. The average compressive strengths of ambient-cured geopolymer concrete at 7 days and 28 days were 35 MPa and 52 MPa, respectively. In addition, the compressive strength of geopolymer concrete on the day of testing for each specimen was also determined. The average compressive strength of geopolymer concrete on the day of testing was 54.4 MPa.

Fabrication and casting of specimens

All the specimens were fabricated and cast as per the design of each specimen. The longitudinal steel and GFRP bars were aligned in the vertical position by using two wooden circular templates having an outer diameter of 112 mm. Each template contained six uniformly distributed holes around the diameter to hold the longitudinal bars in place. One additional hole at the center of each template was used to hold a fixed threaded steel rod as per the required height of the specimens. Afterwards, the longitudinal reinforcement was assembled with the transverse reinforcement using steel wire ties of 150 mm length. The helices were positioned using aluminum spacer jigs to attain 40 mm and 75 mm pitch for the specimens of different groups (Fig. 3c).

Electrical strain gauges were attached to the longitudinal bars and helices to obtain strain readings during the test (Fig. 3d). The longitudinal bars were attached with 25 mm steel pins at the bottom to maintain a uniform concrete cover of 25 mm at the ends of all the specimens. The steel and GFRP reinforcement cages were assembled to attain an outer diameter of 128 mm (Fig. 3e). The steel wires were attached to the reinforcement cages to maintain the concrete cover (16 mm) at the sides of the specimens. Afterwards, the steel and the GFRP reinforcement cages were placed inside the molds. Polyvinyl chloride (PVC) pipes with an

internal diameter of 160 mm were used to cast the specimens. All PVC molds were placed on a flat wooden pallet to avoid unnecessary movements.

The mixing of the geopolymer concrete started with the mixing of the coarse aggregate and fine aggregate for 1 minute. The binders (slag and fly ash) were then added into the mixer and mixed for two minutes. Once the dry materials were mixed, the alkaline solution was slowly added into the mixer, followed by the water and the superplasticizer. The mixing continued for another 3-4 minutes and a homogenous mixture of geopolymer concrete was prepared. The fresh mix of the geopolymer concrete was directly poured into the PVC molds. Based on the capacity of the pan mixer, three batches were prepared to fill the molds for the specimens of each group (S40, G40, and G75). The specimens were poured in three layers and each layer was compacted using an electric vibrator to remove the entrapped air. Afterwards, all the specimens were kept under ambient-curing conditions until the testing day.

Testing setup and instrumentation

All concentrically and eccentrically loaded specimens were tested using a 5000-kN Denison compression testing machine. The top and the bottom ends of the concentric and eccentric loaded specimens were externally wrapped with two layers of carbon fiber-reinforced polymer (CFRP) sheet of 80 mm width and 0.5 mm thickness to avoid any premature failure during the testing due to the stress concentration at the ends. In addition, both ends of the specimens were capped with plaster to ensure a uniform distribution of load across the section during the test. A pair of loading heads were fabricated and used at the top and the bottom of the specimens to apply axial loads with the required eccentricity. The loading heads were composed of circular steel plates and steel ball joints. The inner diameter of each circular steel plate was greater than the diameter of the tested specimen. The gap between the

inner wall of the loading head and the specimen was filled with a 3 mm thick layer of plaster to confine the end region of the specimen. All concentrically loaded specimens were tested without using steel ball joints in the loading heads. Therefore, the specimens experienced concentric axial load directly from the testing machine through the circular steel plates. On the other hand, for the specimens tested under eccentric load, the steel ball joints were used to transfer the eccentric load from the testing machine at 15 mm eccentricity and 35 mm eccentricity. A similar test setup was adopted in Farhan et al. (2018b). Figure 4a shows the test setup for the specimens tested under eccentric loads.

The specimens tested as beams under four-point bending were tested using a 1000-kN loading frame. Two circular steel rigs were used at the top and the bottom of the beam specimens to transfer the load from the testing machine. The inner diameter of each steel rig was equal to the diameter of the tested specimen. All the specimens under four-point bending were tested over a clear span of 1300 mm. The spacing between the two point loads was 433.3 mm. Figure 4b shows the test setup for the specimens tested as beams under four-point bending.

The behavior of the test specimens was monitored through electrical strain gauges, linear variable displacement transducers (LVDTs) and laser triangulation. Two strain gauges were attached to the two opposite outermost longitudinal bars in the plane of bending at the mid-height of the specimens to observe the strains in the longitudinal reinforcement. In addition, two strain gauges were attached to the helices (180° apart) at the mid-height of the specimens to observe the strain development in the transverse reinforcement. All strain gauges were positioned on the reinforcement at the mid-height of the test region. Two LVDTs were attached vertically on the opposite corners in the plane of bending between the loading head and the supporting head of the Denison compression-testing machine to measure the axial

deformation in the specimens tested under concentric and eccentric loads. The lateral deformation of the specimens tested under the eccentric load was recorded by a laser triangulation attached at the mid-height of the tested specimen. The midspan deflection of the specimens tested as beams in the loading frame was recorded by a laser triangulation placed under the midspan of the tested specimens, as shown in Fig. 4b.

The testing began by applying force-controlled loads on the specimens up to 10% of their predicted load carrying capacity at a rate of 2 kN/s. Then the specimens were unloaded to 20 kN at the same rate. This process was adopted to prevent any movement that might have taken place during the test. Afterwards, the loading continued under the displacement-control system at a rate of 0.18 mm/min for the specimens tested in the Denison compression testing machine. A loading rate of 0.96 mm/min was used for the specimens tested as beams under four-point bending in the loading frame. All the specimens were tested until the failure of the specimen. The internal load cell of the testing machines (5000-kN Denison testing machine and 1000-kN loading frame) recorded the test results. The strain gauges, LVDTs and the laser triangulations were attached to a data logger to record the testing data at every 2 s. Two computers were attached to store the test results: one from the internal load cell of the individual testing machine and the other from the data logger.

Experimental results and discussions

Behavior of specimens tested under concentric axial load

Three specimens (one from each group) were tested under concentric axial load. The failure modes of the column specimens tested under concentric axial loads are shown in Fig. 5. The failure modes of the tested specimens under concentric axial loads were mainly influenced by the type of reinforcement (steel or GFRP) and the arrangement of the transverse helices. All specimens were tested to failure.

The failure of Specimens S40E0, G40E0 and G75E0 occurred in the following mechanism. With an increase in the axial load, the longitudinal strain increased. Also, lateral expansion was observed in the geopolymer concrete due to Poisson's effect, which could be due to little or no contribution to the confinement of concrete core provided by helices. At the peak concentric axial load, the lateral expansion resulted in the formation of vertical hairline cracks at the mid-height of the tested specimens. In Specimen G75E0, wider and deeper cracks were observed as compared to Specimens S40E0 and G40E0. This could be due to the increased pitch of the helices in Specimen G75E0 compared to Specimens S40E0 and G40E0. After the peak axial load, the spalling of the concrete cover occurred, causing the formation of progressive cracks and an increase in the lateral expansion of the geopolymer concrete core. The total failure of Specimen S40E0 occurred due to the extensive buckling of the longitudinal steel bars and rupture of the steel helix (Fig. 6a.). Similar observations were reported for concrete columns in Pantelides et al. (2013). However, in Specimen G40E0, closely spaced GFRP helices resisted the lateral movement of the longitudinal GFRP bars. This led to the kinking of the glass fibers in the longitudinal GFRP bars, which was followed by the delamination and fracture of the longitudinal GFRP bars on the tension side, crushing of concrete core and the rupture of the GFRP helices (Fig. 6b). On the other hand, in Specimen G75E0, widely spaced GFRP helices provided less confinement, which resulted in the buckling and rupture of the longitudinal GFRP bars, as shown in Fig. 6c.

The experimental results of the specimens tested under concentric axial load are presented in Table 5. The load capacities are normalized against the compressive strength of geopolymer concrete on the day of testing, using Eq. (1).

$$P^* = \frac{P}{f_{co}A_g} \quad (1)$$

where P^* refers to the normalized load capacity. The f_{co} and A_g are the compressive strength of geopolymer concrete on the day of testing and gross area of the specimen, respectively. The f_{co} values for all the specimens are provided in Table 6. The normalized axial load-axial deformation behaviors of the concentrically loaded Specimens S40E0, G40E0 and G75E0 are shown in Fig. 7. It can be observed that the ascending slope of the curve of Specimen G75E0 was slightly lower than the ascending slope of the curve of Specimen G40E0, although the difference is not significant. The slight variation could be because of the non-homogeneity of the geopolymer concrete caused during mixing, placing and compaction, which might have affected the properties of geopolymer concrete. Neville (2011) reported similar observations for OPC concrete. It can also be observed that Specimens S40E0, G40E0 and G75E0 followed a linear ascending path until the peak load because of the low lateral expansion of the geopolymer concrete due to little or no contribution to the confinement of concrete core provided by helices. This could be because the Poisson's ratio of geopolymer concrete is less than the Poisson's ratio of normal strength OPC concrete (Hardjito 2005). Also, geopolymer concrete is more brittle than OPC concrete. Afterwards, a short non-linear reduced slope before the peak load represents the crushing and cracking of the geopolymer concrete. In general, at 95% of the peak load, hairline cracks started to appear at the mid-height surface of the specimens due to the lateral expansion of the geopolymer concrete. The confinement provided by the helices was partially activated at this stage. Later on, the cracks widened and formed major vertical cracks, followed by the complete spalling of the concrete cover. This led to complete activation of the transverse reinforcement, which provided confinement and resisted the lateral movement of the geopolymer concrete core. As a result, a slight increase in the post-peak behavior was observed, followed by the failure of reinforcement.

The replacement of steel reinforcement (in Specimens S40E0) directly with the same amount of GFRP reinforcement (in Specimens G40E0) resulted in an approximately 27% reduction in the normalized axial load carrying capacity. On the other hand, Specimen G40E0 sustained about 10% higher peak normalized load than Specimen G75E0. This was because of the reduction in the pitch of the GFRP helices from 75 mm to 40 mm. Karim et al. (2016) reported similar observations for circular concrete columns reinforced with GFRP bars and helices. After the peak axial load, all the specimens experienced a drop in the axial load caused by the spalling of the concrete cover, which initiated the lateral expansion of the geopolymer concrete core. Specimen S40E0 experienced a drop of about 49% in the peak normalized axial load before the rupture of the steel helix. The corresponding axial deformation in Specimen S40E0 before the rupture of steel helix was recorded as 17.9 mm. On the other hand, Specimens G40E0 and G75E0 experienced about 25% and 41% loss of their peak normalized axial loads, respectively, before the rupture of the GFRP reinforcement. The rupture of the GFRP helices in Specimens G40E0 and G75E0 occurred at axial deformations of 7.3 mm and 6.4 mm, respectively.

The contribution of longitudinal bars in compression (P_{bar}) is also presented in Table 5. In the reference Specimen S40E0, the value of P_{bar} was calculated by multiplying the nominal area of the longitudinal steel bars with the yield strength of the steel bar. The axial load contribution of GFRP bars was calculated by multiplying the nominal area of the longitudinal GFRP bar with the measured average strain in the longitudinal bars and the modulus of elasticity of the longitudinal bars. In Specimen S40E0, the axial load contribution of longitudinal steel bars in the peak normalized axial load was approximately 21%. On the other hand, the average axial load contribution of GFRP bars in Specimens G40E0 and G75E0 was approximately 6%. This could be due to the brittle nature of GFRP bars compared to steel bars.

Behavior of specimens tested under eccentric load

Six specimens (two from each group) were tested under the eccentric load. Specimens S40E15, G40E15 and G75E15 were tested under 15 mm eccentric load and Specimens S40E35, G40E35 and G75E35 were tested under 35 mm eccentric load. The failure modes of the specimens tested under 15 and 35 mm eccentric loads are shown in Fig. 5.

The failure of the eccentrically loaded specimens initiated with the crushing of the geopolymer concrete cover on the compression face. The crushing of the geopolymer concrete cover on the compression face initiated the transverse cracking on the tension face. This behavior was because of the combined axial-flexural load. It was observed that the width and the depth of the cracks on the tension side increased with an increase in the axial loading and were dependent on the pitch of the helices. The specimens reinforced with widely spaced helices experienced deeper cracks as compared to the specimens reinforced with closely spaced helices. Afterwards, Specimens S40E15 and S40E35 experienced buckling of the longitudinal steel bars on the compression side. The total failure of Specimens S40E15 and S40E35 occurred due to the extensive buckling of the longitudinal steel bars on the compression side and rupture of the longitudinal steel bars on the tension side, as shown in Fig. 6d. After the completion of the test, the remaining intact concrete cover was removed to inspect the failure of the reinforcement. Similar observations were reported for geopolymer concrete columns reinforced with steel bars and helices in Farhan et al. (2018a). On the other hand, in Specimens G40E15 and G75E15, the longitudinal GFRP bars fractured at the mid-height on the compression side of the specimen, causing the failure of the specimens. Specimens G40E35 and G75E35 experienced flexural-tension failure caused by spalling of the concrete cover on the compression side followed by the rupture of the GFRP bars at the upper third portion of the tension side of the specimens (Fig. 5).

The experimental test results of the specimens tested under 15 and 35 mm eccentric loads are presented in Table 7. Figure 8a shows the normalized axial load-axial deformation and normalized axial load-lateral deformation behavior of the specimens tested under 15 mm eccentric load. It can be observed that the ascending part of the normalized axial load-axial deformation curves of steel and GFRP-RC specimens were almost linear until the peak axial load.

The peak normalized axial load sustained by Specimens G40E15 and G75E15 were about 18% and 31%, respectively, lower than the peak normalized axial load sustained by reference Specimen S40E15. After the peak axial load, the concrete cover started to spall off on the compression side, which activated the transverse reinforcement. In comparison with the concentrically loaded specimens, the specimens of Groups S40, G40 and G75 experienced 31%, 22% and 28% reduction, respectively, in the normalized load carrying capacities for the increase in the loading eccentricity from zero to 15 mm.

Figure 8b shows the normalized axial load-axial deformation and normalized axial load-lateral deformation behavior of the specimens tested under 35 mm eccentric load. The behavior of Specimens S40E35, G40E35 and G75E35 in the ascending part of the normalized axial load-axial deformation curve was almost linear until the peak normalized axial load. Specimens G40E35 and G75E35 sustained 16% and 29% lower peak normalized axial load, respectively, than the reference Specimen S40E35. In comparison with the concentrically loaded specimens, the specimens of Groups S40, G40 and G75 sustained 64%, 58% and 61% lower peak normalized axial load, respectively, by increasing the eccentricity of the axial load from zero to 35 mm. It was also observed that Specimens G75E15 and G75E35 sustained lower peak axial loads compared to Specimens G40E15 and G40E35 due to the lower confinement pressure provided by the transverse reinforcement.

Behavior of specimens tested under four-point bending

Three specimens (one from each group) were tested as beams under four-point bending to investigate the behavior of the geopolymer concrete specimens under flexural load. Figure 9 presents the failure mode of the geopolymer concrete specimens tested under four-point bending.

The failure of the specimens tested as beams under four-point bending initiated with the formation of hairline cracks at the midspan on the tension side. This was followed by the crushing of concrete in the compression region at the midspan of the specimens. The width and number of the cracks on the tension side varied depending upon the pitch of the helices in the specimens. The specimens reinforced with closely spaced helices (S40B and G40B) experienced closely spaced hairline cracks. However, the specimen reinforced with widely spaced helices (G75B) exhibited wide but deep cracks. Similar observations were reported in Hasan et al. (2017). The crushing of concrete on the mid-span compression side resulted in an increase in the deflection of the specimens, which led to the development of new cracks as well as increased the depth of the existing cracks on the tension side. At this stage, the flexural load of the specimens was dropped to 80%. Afterwards, the flexural load continued to drop until the rupture of the reinforcing bars on the tension side, which led to the ultimate failure of the specimens. The rupture of the longitudinal steel bars in the extreme tension side at the midspan led to the total failure of Specimen S40B. On the other hand, the rupture of both GFRP bars and GFRP helices on the tension side at the midspan resulted in the complete failure of Specimens G40B and G75B.

The experimental test results of steel and GFRP-RC specimens tested as beams under four-point bending are presented in Table 8. Figure 10 shows the normalized load-midspan deflection behavior of the specimens tested under four-point bending. It can be observed that all the specimens showed an ascending behavior until the first peak load. The initial cracking

in Specimens S40B, G40B and G75B started at almost same normalized load. The reference Specimen S40B showed only one peak load. On the other hand, Specimens G40B and G75B continued to resist the flexural loads beyond the first peak loads and achieved the second peak loads. The normalized loads sustained by Specimens G40B and G75B at the first peaks were approximately 12% and 17% less, respectively, compared to the reference Specimen S40B. The midspan deflections of Specimens G40B and G75B at the first peak load were approximately 46% and 60% higher, respectively, than the midspan deflection of Specimen S40B. Afterwards, a drop in the flexural loads was observed due to the crushing of the geopolymer concrete cover on the compression face.

In the post-peak zone, Specimens G40B and G75B showed ascending post-peak behavior until the second peak load due to the high tensile strength and the elastic stress-strain relationship of GFRP bars and GFRP helices. The second peak normalized loads of Specimens G40B and G75B were about 27% and 13% higher, respectively, compared to their first peak loads. Afterwards, all the specimens experienced crushing of the concrete cover on the compression face, followed by a drop in the load. In general, Specimen G40B sustained approximately 12% and 19% higher normalized load than Specimens S40B and G75B, respectively.

Confinement Efficiency and Deformability

In this study, the experimental confinement efficiency and deformability of the specimens were determined to investigate the influence of the type of reinforcement (steel or GFRP) and the arrangement of reinforcement. The procedure for determining the confinement efficiency and deformability was based on the recommendations suggested in Afifi et al. (2014) and Hadi et al. (2016), respectively. The experimental confinement efficiency of the column specimens was calculated as the ratio of the compressive strength of the confined geopolymer

concrete column to the compressive strength of the unconfined geopolymer concrete column (f'_{cc} / f_{co}). The f'_{cc} represents the maximum stress sustained by the area of the confined geopolymer concrete core. On the other hand, the f_{co} is equivalent to $0.85 f'_c$. The area of the confined geopolymer concrete core was measured based on the outside to outside diameter of the helices. A similar approach was adopted in Pantelides et al. (2013) for the calculation of the area of the confined concrete core. The deformability of the specimens was computed as the area under the axial load-axial deformation curve up to the ultimate deformation divided by the area of the axial load-axial deformation curve up to the deformation at the end of the linear elastic stage. The ultimate deformation represents the smaller of the first fracture load of the reinforcement and 80% of the peak load, as adopted in Hadi et al. (2016). The point where the first crack occurred or the ascending branch of the load-deformation curve started flattening prior to the peak load was considered as the end of the linear elastic stage for all the specimens. The load and deformation values recorded at the end of the linear elastic stage of the concentrically and eccentrically loaded specimens are presented in Table 5 and Table 7, respectively. Table 6 shows the values of the confinement efficiency and deformability of the specimens.

The direct replacement of steel reinforcement by the same amount of GFRP reinforcement resulted in an approximately 28% reduction in the confinement efficiency in Specimen G40E0 compared to the confinement efficiency in Specimen S40E0. The confinement efficiency of Specimen G40E15 was approximately 19% less than that of Specimen S40E15. Specimen G40E35 achieved approximately 7% less confinement efficiency than that of Specimen S40E35. In GFRP-RC specimens, the reduction in the pitch of GFRP helices from 75 mm to 40 mm resulted in the improvement in confinement efficiency. Thus, Specimen G40E0 achieved about 5% higher confinement efficiency than that of Specimen G75E0. The confinement efficiency of Specimen G40E15 was about 17% higher compared to Specimen

G75E15. The confinement efficiency of Specimen G40E35 was 28% higher than the confinement efficiency of Specimen G75E35.

Figure 11a shows the effect of loading eccentricity on the confinement efficiency of the specimens. It was observed that the confinement efficiency of the specimens decreased with an increase in the loading eccentricity from zero to 15 and 35 mm (Fig. 11a). The confinement efficiency of Specimens S40E15 and S40E35 were about 32% and 64% less, respectively, compared to the confinement efficiency of Specimen S40E0. Similarly, the confinement efficiency of Specimens G40E15 and G40E35 were about 23% and 53% less, respectively, compared to the confinement efficiency of Specimen G40E0. The confinement efficiency of Specimens G75E15 and G75E35 were about 31% and 62% less, respectively, compared to the confinement efficiency of Specimen G75E0.

Figure 11b shows the deformability of the specimens tested under different loading conditions. It can be observed that the deformability of GFRP-RC Specimen G40E0 was approximately 42% less than the deformability of steel-RC Specimen S40E0. The deformability of Specimens G40E15 and G40E35 were approximately 4% and 8% less, respectively, compared to the deformability of Specimens S40E15 and S40E35. Overall, the specimens with closely spaced GFRP helices, experienced effective confinement, which resisted the lateral movement of the geopolymer concrete core in the specimens. Specimen G40E0 achieved about 14% higher deformability than Specimen G75E0. The deformability of Specimen G40E15 was approximately 15% higher than the deformability of Specimen G75E15. Specimen G40E35 achieved 25% higher deformability than Specimens G75E35.

Figure 11b also shows that the increase in the loading eccentricity from zero to 15 and 35 mm resulted in the reduction of deformability for all the steel and the GFRP-RC geopolymer concrete specimens. The deformability of eccentrically loaded steel-RC Specimens S40E15 and S40E35 were about 43% and 45%, respectively, lower than the deformability of the

concentrically loaded Specimen S40E0. The deformability of eccentrically loaded GFRP-RC Specimens G40E15 and G40E35 were approximately 5% and 11%, respectively, lower than the deformability of the concentrically loaded Specimen G40E0. Similarly, the deformability of Specimens G75E15 and G75E35 were about 6% and 20%, respectively, lower than the deformability of Specimen G75E0.

Unlike the concentric and eccentric loaded specimens, the deformability of the GFRP-RC specimen tested under four-point bending was significantly higher than the deformability of the counterpart steel bar reinforced specimen, as shown in Fig. 11b. This could be because of the higher tensile strength of GFRP bars compared to the equivalent steel bars (Table 4). Therefore, the contribution of GFRP bars was higher than the steel bars in resisting the flexural load. Thus, the deformability of Specimen G40B was approximately 31% higher than the deformability of Specimen S40B. In GFRP-RC specimens, the reduction in the pitch of the GFRP helices increased the deformability. Specimen G40B achieved 15% higher deformability than Specimen G75B. Similar observations were reported for the GFRP-RC specimens in Hadi et al. (2017).

Experimental axial load-bending moment interaction diagrams

The experimental axial load-bending moment (P - M) interaction diagrams plotted for the specimens of Groups S40, G40 and G75 were based on four points. In the (P - M) interaction diagram, the first point represents the concentrically loaded specimens. The second and third points represent the 15 mm and 35 mm eccentrically loaded specimens, respectively. The fourth point represents the pure bending moment capacity of the specimen tested as a beam under four-point bending. For the specimens tested under concentric axial load, the bending moment was considered to be zero. For the specimens tested under 15 mm and 35 mm eccentric axial loads, the experimental bending moment capacity (M) at the mid-height was

calculated using Eq. (2). The experimental bending moment capacity (M) of the specimens tested as beams under four-point bending was calculated using Eq. (3).

$$M = P_{peak} (e + \delta) \quad (2)$$

$$M = \frac{P_{peak1} \times L}{6} \quad (3)$$

where; P_{peak} = the peak axial load, e = the load eccentricity, and δ = the lateral deformation of the specimen corresponding to the peak axial load, P_{peak1} = the first peak flexural load, and L = the length (1300 mm) between the end supports of the beam specimens.

The experimental results of the flexurally loaded specimens are presented in Table 9 and the experimental (P - M) interaction diagrams are shown in Fig. 12. It can be observed that the load-bending moment capacity of the steel-RC specimens of Group S40 under concentric axial load, eccentric axial load and four-point bending were higher than the GFRP-RC specimens of Group G40. This could be because of the high modulus of elasticity of the steel bars compared to the GFRP bars. On the other hand, the load bending moment capacity of the specimens of Group G40 was significantly higher compared to the load-bending moment capacity of the specimens of Group G75.

Experimental versus predicted load capacity of concentrically loaded specimens

The American design code ACI 318-19 (ACI 2019) recommends Eq. (3) to predict the axial load capacity of OPC concrete columns reinforced with the steel reinforcement. However, considering the lower compressive strength of FRP bars compared to their tensile strength, ACI 440.1R-15 (ACI 2015) does not recommend the use of FRP bars for the longitudinal reinforcement of concrete columns. In fact, ACI 440.1R-15 (ACI 2015) provides no design guidelines for the longitudinal reinforcement of concrete columns with FRP bars. The

Canadian standard CSA S806-12:R17 (CSA 2017) permits the use of FRP bars for the longitudinal reinforcement of concrete columns but neglects the contribution of FRP bars in the axial load carrying capacity of FRP bar RC columns. Based on CSA S806-12:R17 (CSA 2017), Eq. (4) can be used to predict the axial load capacity of FRP bar RC columns.

$$P_o = 0.85 f'_c (A_g - A_{st}) + f_y A_{st} \quad (4)$$

$$P_o = \alpha f'_c (A_g - A_f); \quad \alpha = 0.85 - 0.0015 f'_c \geq 0.67 \quad (5)$$

where P_o is the theoretical axial load carrying capacity, f'_c is the 28-day unconfined compressive strength of concrete; A_g , A_{st} and A_f are the gross area of the column, area of steel bars and area of FRP bars, respectively.

A number of studies reported that neglecting the contribution of longitudinal FRP bar in compression (as presented in Eq. 5) underestimated the axial load carrying capacity of FRP-RC columns (Tobi et al. 2012; Afifi et al. 2014; Hadi et al. 2016). Therefore, in the literature, two different approaches were adopted to account for the contribution of longitudinal FRP bars in the axial load carrying capacity of the columns. In the first approach, Afifi et al. (2014) calculated the stress in FRP bars by using the tensile strength of the FRP bars, as shown in Eq. 6.

$$P_o = 0.85 f'_c (A_g - A_f) + \alpha_f f_{fu} A_f \quad (6)$$

where α_f is a reduction factor to account the decrease in the compressive strength of GFRP bar as a function of the tensile strength of GFRP bar. The value of α_f was recommended to be assumed as 0.35 in Afifi et al. (2014). In Eq. (4), f_{fu} represents the ultimate tensile strength of the longitudinal FRP bar in tension.

In the second approach, the stress in the FRP bars was calculated using the axial strain (ϵ_f) in the FRP bars and the modulus of elasticity (E_f) of the FRP bars, as shown in Eq. (7).

$$P_o = 0.85 f'_c (A_g - A_f) + \varepsilon_f E_f A_f \quad (7)$$

Mohamed et al. (2014) used Eq. (7) to calculate P_o of the FRP-RC columns by including the FRP strength contribution until the development of micro-cracks in the concrete plastic stage. In Mohamed et al. (2014), the ε_f value was assumed as 0.002 corresponding to the initiation of plastic concrete deformation. Similarly, Hadi et al. (2016) and Hadhood et al. (2017) used Eq. (7) to calculate the P_o for the FRP-RC columns. However, in Hadi et al. (2016) and Hadhood et al. (2017), ε_f was taken as 0.003. Further, Hadhood et al. (2017) also used the modified factors $k_1 = 0.85 - 0.0015 f'_c$ and $k_2 = 0.0035$ in Eq. (7) and provided reasonable predictions for the P_o of the FRP-RC columns. It is noted that the recent Canadian highway bridge design code CSA S6-19 (CSA 2019) considers the contribution of the compressive strength of FRP bars by limiting the stress corresponding to a strain of 0.002 in the calculation of the axial load carrying capacity of FRP bar RC columns.

In this study, it was observed that the change in the pitch of the GFRP helices affected the confinement stress, which altered the axial load carrying capacities of the columns. Similar observations were reported by Pantelides et al. (2013), Karim et al. (2016), Maranan et al. (2016) and Hadhood et al. (2018). Thus, considering Eq. (7), f'_c was replaced with f'_{cc} (ultimate confined concrete strength) to include the effect of confinement in the theoretical axial load carrying capacity of the concentrically loaded GFRP-RC column specimens, as shown in Eq. (8).

$$P_o = 0.85 f'_{cc} (A_g - A_f) + \varepsilon_f E_f A_f \quad (8)$$

The value of f'_{cc} was calculated based on the lateral confining pressure (f_{FRP}) provided by the GFRP helices, using different prediction equations, including Pantelides et al. (2013), Karim et al. (2016), Maranan et al. (2016) and Hadhood et al. (2018), as shown in Table 10. Among the available equations for confined concrete strength (f'_{cc}) and lateral confining

pressure (f_{lFRP}), the equations proposed in Maranan et al. (2016) were developed to investigate especially the behavior of GFRP-RC geopolymer columns (Table 10). The predicted values of P_o were calculated using $\varepsilon_f = 0.003$ in the longitudinal GFRP bars.

Table 11 presents a comparison between the normalized experimental and theoretical axial load carrying capacities of all the concentrically loaded geopolymer concrete columns. The $P_o / P_{exp.}$ refer to the ratio of the predicted load to the experimental load carrying capacity of the columns. It can be observed that, as expected, based on ACI 318-19 (ACI 2019), Eq. (2) reasonably predicts the P_o of steel bar reinforced geopolymer concrete column specimen with $P_o / P_{exp.}$ of 0.94. The P_o of the concentrically loaded GFRP-RC column calculated using CSA S806-12:R17 (CSA 2017) and CSA S6-19 (CSA 2019) underestimated the axial load capacity of the specimen with $P_o / P_{exp.}$ of 0.77 and 0.82, respectively. The approaches adopted in Mohammed et al. (2014), Hadi et al. (2016) and Hadhood et al. (2017) predicted the P_o for the concentrically loaded GFRP-RC column with $P_o / P_{exp.}$ ranged between 0.85 - 0.93. The approach adopted by Afifi et al. (2014) overestimated the axial load carrying capacity of the GFRP-RC column with $P_o / P_{exp.}$ of 1.02.

It was also observed that, to consider the effect of the confinement, the prediction equations proposed in Pantelides et al. (2013), Karim et al. (2016), Maranan et al. (2016) and Hadhood et al. (2018) provided a good correlation between the experimental and theoretical P_o of all the GFRP-RC columns. Pantelides et al. (2013) provided accurate predictions for all the GFRP-RC column specimens with $P_o / P_{exp.}$ ranging 0.97 - 0.99. Based on Karim et al. (2016) and Maranan et al. (2016), the $P_o / P_{exp.}$ for all GFRP-RC columns ranged 0.93 - 0.99 and 0.75 - 0.86, respectively. The predictions based on Hadhood et al. (2018) for all GFRP-RC columns ranged between 0.85 - 0.90. Hence, the prediction equations proposed by Pantelides et al. (2013) and Hadi et al. (2016) and Hadhood et al. (2018) can be used for the precise predictions of the P_o for the geopolymer concrete columns reinforced with GFRP bars

and GFRP helices. Also, the contribution of longitudinal GFRP bars should not be neglected and need to be taken into account in predicting the axial load capacity of FRP bar reinforced geopolymer concrete columns.

Conclusions

This study investigated the behavior of ambient cured geopolymer concrete circular column specimens under different loading conditions. The effect of replacing steel reinforcement with the same amount of GFRP reinforcement was investigated. Based on the experimental results, the following conclusions are drawn:

1. The failure of the steel RC specimens initiated with the buckling of the longitudinal steel bars and then rupture of the longitudinal steel bar or fracture of steel helix, which caused a complete failure of the specimens. On the other hand, the failure of the GFRP-RC specimens initiated with the kinking of glass fibers in GFRP bars. The complete failure of the specimens occurred due to the rupture of the longitudinal GFRP bars and GFRP helices.
2. Replacing the steel reinforcement with the same amount of GFRP reinforcement in the geopolymer concrete column specimens resulted in the reduction of the normalized axial load carrying capacity by 27%, 18% and 16% tested under concentric, 15 mm eccentric and 35 mm eccentric loads, respectively.
3. The replacement of steel reinforcement with the same amount of GFRP reinforcement in the geopolymer concrete column specimens resulted in the reduction of the deformability by 42%, 4% and 8% under concentric, 15 mm eccentric and 35 mm eccentric loads, respectively.
4. Reducing the pitch of the GFRP helices from 75 mm to 40 mm improved the normalized load carrying capacity by 10%, 19% and 19% for specimens tested under concentric, 15

mm eccentric and 35 mm eccentric loads, respectively. The improvement in the deformability due to the reduction in the pitch of the GFRP helices was 5%, 17% and 28% for specimens tested under concentric, 15 mm eccentric and 35 mm eccentric loads, respectively.

5. The contribution of the longitudinal GFRP bars was on average 6% in the axial load carrying capacity of the GFRP-RC specimens.

6. Under four-point bending, all GFRP-RC specimens achieved second peak axial loads, unlike the steel bar reinforced specimen. For the same pitch of the helices, GFRP-RC specimen achieved approximately 12% and 31% higher flexural load and deformability, respectively, compared to its steel bar reinforced counterpart specimen.

7. The theoretical axial load carrying capacity of the geopolymer concrete columns reinforced with GFRP bars and GFRP helices can be reasonably predicted by considering the contribution of the longitudinal FRP bars and the effect of the FRP confinement provided by the helices.

Based on the results of this study, it can be concluded that GFRP reinforcement is a promising material that can be used for reinforcing circular geopolymer concrete columns. The above conclusions were drawn based on the experimental study conducted on twelve circular geopolymer concrete columns tested under different loading conditions. Considering that the size of the tested concentrically and eccentrically loaded specimens is small, further research investigations are required on full-scale GFRP-RC geopolymer concrete columns for their wide practical applications.

Data Availability Statement

All data used during the study appear in the submitted article.

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Table 1. Test matrix.

Group	Specimen	Longitudinal reinforcement	Transverse reinforcement	Axial Load eccentricity (mm)
S40	S40E0	Steel 6N10	Steel R8 @ 40 mm	0
	S40E15			15
	S40E35			35
	S40B			Flexural
G40	G40E0	GFRP 6G10	GFRP G8 @ 40 mm	0
	G40E15			15
	G40E35			35
	G40B			Flexural
G75	G75E0	GFRP 6G10	GFRP G8 @ 75 mm	0
	G75E15			15
	G75E35			35
	G75B			Flexural

Table 2. Mix proportions of the geopolymer concrete (Based on Ali et al. 2019)

Material	Quantity
FA (kg/m ³)	270
GGBFS (kg/m ³)	180
Aggregate (kg/m ³)	1295
Sand (kg/m ³)	552
Na ₂ SiO ₃ (kg/m ³)	112.5
NaOH (kg/m ³)	45
NaOH (mole/l)	14
Al/Bi ^a	0.35
Na ₂ SiO ₃ /NaOH	2.5
Superplasticizer (kg/m ³)	34.7
Water (kg/m ³)	86.4

^a Al/Bi = alkaline activator to binder mass ratio.

Table 3. Chemical composition (mass %) of ground granulated blast furnace slag (GGBFS) and fly ash (FA).

Material	Components										
	CaO	Al ₂ O ₃	SiO ₂	MgO	Fe ₂ O ₃	K ₂ O	Na ₂ O	TiO ₂	P ₂ O ₅	Mn ₂ O ₃	SO ₃
GGBFS	42.7	12.8	34.5	5.3	0.4	0.3	0.8	0.8	0.1	0.4	1.7
FA	2.9	26.2	62.4	0.4	2.6	1.1	0.6	0.9	0.2	0.1	0.3

Note: Loss on ignition for GGBFS and FA were 0.1% and 2.0% by mass, respectively.

Table 4. Average properties of the bars.

Average properties of the bars	R8 (Steel)	N10 (Steel)	G8 (GFRP)	G10 (GFRP)
Nominal diameter (mm)	8	10	8(7.7 ^a , 7.4 ^b)	10(9.8 ^a , 9.2 ^b)
Tensile strength (MPa)	520 ^c	552 ^c	1162 ^d	1263 ^d
Tensile strain (mm/mm)	0.0029 ^c	0.0032 ^c	0.0185 ^d	0.0207 ^d
Elastic tensile modulus (GPa)	190	193	61.9	63.8
Compressive Strength (MPa)	-	-	907 ^d	827 ^d

Note: Tensile strength, elastic tensile modulus and compressive strength of GFRP bars are calculated based on the root diameter.

^a Obtained using immersion test in accordance with ISO 104061.1-15 (2015).

^b Root diameter provided by the manufacturer (Pultron 2019).

^c Yielding.

^d Ultimate.

Table 5. Experimental results of geopolymer concrete specimens tested under concentric axial load.

Specimen	S40E0	G40E0	G75E0
P_l^a (kN)	1175	1027	863
P_l^*	1.16	0.87	0.80
Axial deformation corresponding to P_l^* (mm)	3.5	3.4	3.0
Peak axial load, P_{peak} (kN)	1244	1062	877
P_{peak}^*	1.24	0.90	0.82
Axial deformation corresponding to P_{peak}^* (mm)	3.8	3.7	3.1
Average strain in the helices at peak load, ϵ_{helix} ($\mu\epsilon$)	745	625	521
P_{bar}^b (kN)	259	64	53

^a P_l = Load at the end of the linear elastic stage.

^b P_{bar} = Average axial load contribution by longitudinal bars at peak load.

Note: * represents the normalized load capacities of the test specimens calculated using Eq. (1).

Table 6. Experimental peak load, compressive strength, normalized load, confinement efficiency and deformability of geopolymer concrete specimens tested under different loading conditions.

Specimen	P_{peak} (kN)	f_{co} (MPa)	$P_{peak}^* = \frac{P_{peak}}{f_{co}A_g}$	Confinement efficiency	Deformability
S40E0	1244	50	1.24	1.9	2.6
S40E15	926	54	0.85	1.3	1.5
S40E35	494	55	0.45	0.7	1.4
S40B	72	46	0.07	-	3.9
G40E0	1062	59	0.90	1.4	1.5
G40E15	778	55	0.70	1.1	1.4
G40E35	470	61	0.38	0.6	1.3
G40B	82	47	0.08	-	5.0
G75E0	879	53	0.82	1.3	1.3
G75E15	734	62	0.59	0.9	1.2
G75E35	416	64	0.32	0.5	1.1

G75B 69 47 0.07 - 4.4

Table 7. Experimental results of geopolymer concrete specimens tested under eccentric loads.

Specimen	15 mm eccentric load			35 mm eccentric load		
	S40E15	G40E15	G75E15	S40E35	G40E35	G75E35
P_l (kN)	905	766	711	476	452	396
P_l^*	0.83	0.69	0.57	0.43	0.36	0.30
Axial deform. at P_l^* (mm)	2.8	2.3	2.4	2.3	2.0	2.5
Peak axial load, P_{peak} (kN)	926	778	734	494	470	416
P_{peak}^*	0.85	0.70	0.59	0.45	0.38	0.32
Axial deform. at P_{peak}^* (mm)	2.9	2.4	2.5	2.5	2.1	2.7
Lateral deform. at P_{peak}^* (mm)	2.7	2.1	1.6	3.1	2.1	3.6

Table 8. Experimental results of the geopolymer concrete specimens tested under four-point bending.

Specimen	S40B	G40B	G75B
First peak load, P_{peak1} (kN)	72	65	61
P_{peak1}^*	0.077	0.068	0.064
Midspan deflection at P_{peak1}^* (mm)	22.2	32.5	35.5
Second peak load, P_{peak2} (kN)	-	82	69
P_{peak2}^*	-	0.086	0.072
Midspan deflection at P_{peak2}^* (mm)	-	68.3	72.5

Note: * represents the normalized load capacities.

Table 9. Experimental load vs. bending moment capacity of geopolymer concrete specimens tested under different loading conditions.

Group	Specimen	P_{peak} (kN)	$P_{peak}^* = \frac{P_{peak}}{f_{co}A_g}$	Lateral deformation corresponding to P_{peak} (mm)	Bending moment capacity (kN.m)
S40	S40E0	1244	1.24	-	-
	S40E15	926	0.85	2.7	16.4
	S40E35	494	0.45	3.1	18.8
	S40B	72	0.077	22.2	15.6
G40	G40E0	1062	0.90	-	-
	G40E15	778	0.70	2.1	13.3
	G40E35	470	0.38	2.1	17.4
	G40B	65	0.068	32.5	14.0
G75	G75E0	879	0.82	-	-
	G75E15	734	0.59	1.6	12.2
	G75E35	416	0.32	3.6	16.1
	G75B	61	0.064	35.5	13.2

Note: The values for the specimens tested under four-point bending are corresponding to P_{peak1} .

Table 10. Equations for confined concrete strength (f'_{cc}) and lateral confining pressure (f_{lFRP}) in GFRP-RC columns.

Sr. No.	Author and Reference	Equations
1	Pantelides et al. (2013)	$f'_{cc} = f'_c + \psi_f 3.3 k_a f_{lFRP}$ $f_{lFRP} = \frac{2E_{FRP}A_{spFRP}\epsilon_{fe}}{sd_c}$
2	Karim et al. (2016)	$f'_{cc} = k_c f'_{co}$ $f_{lFRP} = \frac{2 A_h k_e f_{bent}}{d_c s}$
3	Maranan et al. (2016)	$f'_{cc} = f'_{co} + a(\rho_{ft} f_{lFRP})^b$ $f_{lFRP} = \frac{\rho_{ft}}{2} E_f \epsilon_{ftcc}$
4	Hadhood et al. (2018)	$f'_{cc} = f'_c [-1.254 + 2.254 \sqrt{1 + \frac{7.94 f_{lFRP}}{f'_c}} - 2 \frac{f_{lFRP}}{f'_c}]$ $f_{lFRP} = 0.5 k_e f_{fb} \frac{4 A_t}{s' d_s}$

Table 11. Normalized experimental and predicted axial load capacities of geopolymer concrete specimens.

Specimen	S40E0 ($P_{exp.}^* = 1.24$)			
Study	P_o^*	$P_o^* / P_{exp.}^*$		
ACI 318-19 (ACI 2019)	1.16	0.94		

Specimen	G40E0 ($P_{exp.}^* = 0.90$)		G75E0 ($P_{exp.}^* = 0.82$)	
Study	P_o^*	$P_o^* / P_{exp.}^*$	P_o^*	$P_o^* / P_{exp.}^*$
CSA S806-12:R17 (CSA 2017)	0.69	0.77	-	-
CSA S6-19 (CSA 2019)	0.74	0.82	-	-
Mohammed et al. (2014)	0.81	0.90	-	-
Afifi et al. (2014)	0.92	1.02	-	-
Hadi et al. (2016)	0.83	0.93	-	-
Hadhood et al. (2017)	0.83 ⁺ / 0.77 ⁺⁺	0.93 ⁺ / 0.85 ⁺⁺	-	-
Pantelides et al. (2013)	0.87	0.97	0.81	0.99
Karim et al. (2016)	0.84	0.93	0.82	0.99
Maranan et al. (2016)	0.67	0.75	0.71	0.86
Hadhood et al. (2018)	0.77	0.85	0.74	0.90

$P_{exp.}^*$ is the maximum normalized experimental axial load carrying capacity of the column.

P_o^* is the maximum normalized theoretical axial load carrying capacity of the column.

$P_o^* / P_{exp.}^*$ is the ratio of the normalized theoretical and experimental axial load carrying capacity of the column.

⁺ Calculated using $k_1 = 0.85$ and $k_2 = 0.0035$.

⁺⁺ Calculated using $k_1 = 0.85 - 0.0015 f_c'$ and $k_2 = 0.0036$.

Fig 1

Fig 2

Fig 3

Fig 4

Fig 5

Fig 6

Fig 7

Fig 8(a)

Fig 8(b)

Fig 9

Fig 10

Fig 11(a)

Fig 11(b)

Fig 12