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Abstract

This paper presents the results of an experimental study on the axial and flexural behavior of square concrete members reinforced with glass fiber-reinforced polymer (GFRP) bars and embedded with pultruded GFRP structural sections under different loading conditions. The main parameters investigated in this study were the influence of the type of internal reinforcement (steel bars, GFRP bars and pultruded GFRP structural I-sections and C-sections) and magnitude of load eccentricity on the flexural and compressive behavior of square concrete members. To fulfil the objectives of this study, 16 reinforced concrete specimens were tested, of which 12 were tested as columns under compression loading and 4 were tested as beams under flexural loading. The concrete specimens were square in cross section with a side dimension of 210 mm and a height of 800 mm. The experimental results have shown that the steel-reinforced specimens have a higher load-carrying capacity than specimens reinforced with GFRP bars for all loading conditions. In addition, for concentrically loaded specimens, steel-reinforced specimens have a better ductile performance than specimens reinforced with GFRP bars. In terms of eccentric loading, specimens reinforced with GFRP bars experienced similar ductility as compared to the corresponding steel-reinforced specimens. However, the eventual failure mode of specimens reinforced with GFRP bars was sudden and brittle in nature. However, specimens encased with GFRP structural sections have a higher load-carrying capacity but considerably lower ductility than the steel-reinforced and GFRP bar-reinforced specimens.

Keywords

load, concrete, specimens, under, axial, eccentric, experimental, four, investigation, bending, test, gfrp, reinforced, encased, point, square

Disciplines

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Experimental Investigation of GFRP Reinforced and GFRP Encased Square Concrete Specimens under Axial and Eccentric Load, and Four- Point Bending Test

Muhammad N.S. Hadi, F.ASCE¹ and Jim Youssef²

Abstract

This paper presents the results of an experimental study on the axial and flexural behavior of square concrete members reinforced with glass fiber reinforced polymer (GFRP) bars and embedded with pultruded GFRP structural sections under different loading conditions. The main parameters investigated in this study were the influence of the type of internal reinforcement (steel bars, GFRP bars and pultruded GFRP structural I-sections and C-sections) and magnitude of load eccentricity on the flexural and compressive behavior of square concrete members. In order to fulfil the objectives of this study, 16 reinforced concrete specimens were tested, of which 12 were tested as columns under compression loading and 4 were tested as beams under flexural loading. The concrete specimens were square in cross section with a side dimension of 210 mm and a height of 800 mm. The experimental results have shown that the steel reinforced specimens have a higher load carrying capacity than specimens reinforced with GFRP bars for all loading conditions. In addition for concentrically loaded specimens, steel reinforced specimens have a better ductile performance than specimens reinforced with GFRP bars. In terms of eccentric loading, specimens reinforced with GFRP bars experienced similar ductility as compared to the

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corresponding steel reinforced specimens. However, the eventual failure mode of specimens reinforced with GFRP bars was sudden and brittle in nature. On the other hand, specimens encased with GFRP structural sections have a higher load carrying capacity but considerably lower ductility than the steel reinforced and GFRP bar reinforced specimens.

CE Database subject headings: Columns; Application of FRPs; Composite systems; Hybrid structures; Pultruded shapes

Introduction

Based on the literature, understanding the behavior of concrete members reinforced with FRP bars has been the main objective of many researchers. In the last decade, there has been extensive research on the flexural and shear behavior of concrete members reinforced with FRP bars (Thériault and Benmokrane 1998; and Benmokrane et al. 1996). Therefore, the level of understanding of the flexural behavior of FRP-reinforced concrete beams has reached a stage where design standards and guidelines around the world have been developed for the design of these members. However, the structural behavior of columns reinforced with FRP bars has been examined by only a few limited studies (Mohamed et al. 2014; De Luca et al. 2010; and Hadi et al. 2015). Consequently, ACI 440.1R-06 (ACI 2006) does not recommend using FRP bars in columns as longitudinal reinforcement. On the other hand, CSA S806-12 (CSA 2012) neglects the compressive contribution of FRP longitudinal reinforcement. Given the lack of experimental data about FRP reinforcement in compression members, this study aims to expand the understanding of the compression behavior of concrete columns internally reinforced with GFRP bars.

Furthermore, most of the findings of studies investigating FRP reinforced concrete columns have been reported based on testing under concentric loading (Afifi et al. 2014; Tobbi et al. 2012; and De Luca et al. 2010), whereas only a few studies presented investigations of

columns subjected to eccentric loading, for example Xue et al. (2014) and Issa et al. (2011). In reality columns are not subjected to perfect concentric loading but are influenced by a combination of axial compression loads and bending moments (Hadi 2006). Even for columns nominally carrying only axial compression load, bending moments always exist. These bending moments are introduced by unintentional load-eccentricities and by out-of-straightness of the constructed column (Warner et al. 2007). Consequently, this study investigates the behavior and performance of FRP-reinforced concrete columns subjected to eccentric loading.

In recent times, GFRP pultruded profiles low self-weight, non-corrosive nature, low maintenance requirements and high durability have allowed them to become a competitive replacement as a primary structural material in place of steel and reinforced concrete. However, their application is still hindered by their sensitivity to buckling, high deformability and the lack of design codes (Correia et al. 2013). Having said this, there is an interesting potential for the use of GFRP pultruded profiles in hybrid GFRP-concrete structural elements, either for new constructions or for the rehabilitation of existing structures, as reported by many researchers. Kwan and Ramli (2013) conducted a study of completely encasing a pultruded GFRP I-section in a concrete beam for reinforcing purposes. Correia et al. (2009) developed a GFRP-concrete hybrid beam composed of pultruded GFRP I-sections connected to a concrete layer on the top flange. However, there have been no studies available on structural GFRP sections encased concrete columns.

Another focus of this study is the encasement of pultruded GFRP sections in concrete columns to achieve a hybrid GFRP-concrete member. The primary objective of encasing GFRP pultruded structural sections in concrete is to improve the load carrying capacity of FRP reinforced concrete members. The secondary objective of encasing the GFRP sections in

concrete is to prevent its buckling phenomenon and increase its flexural stiffness. In order to investigate the viability of encasing GFRP sections in concrete an experimental program has been designed and executed. This paper summarises the results of an experimental program investigating the behavior of GFRP reinforced and GFRP encased concrete columns and beams. Parameters investigated include the magnitude of load eccentricity and type of internal reinforcement with steel reinforced, GFRP reinforced, GFRP I-section encased and GFRP C-sections encased concrete specimens tested under compressive and flexural loading.

Design of Specimens

The test matrix was organized to investigate the influence of reinforcement type (steel bars, GFRP bars and encased pultruded GFRP structural sections) and magnitude of load eccentricity on the compressive and flexural behavior of square concrete specimens. Table 1 shows the test matrix. In this study the specimens were classified into four groups: reference steel reinforced (RS), reference GFRP reinforced (RF), GFRP I-section encased (I) and GFRP C-sections encased (C). Each group consisted of four specimens; one specimen was tested concentrically, one tested under 25 mm eccentricity, one tested under 50 mm eccentricity and the last specimen was tested as a beam under four point loading test. Each specimen had a square cross section with a side dimension of 210 mm and a height of 800 mm.

The notation of the specimens consists of two parts: the first part is either a letter “RS”, “RF”, “I” and “C” stating the name of the group, and the second part is either “0”, “25”, “50” or “B” which indicates the eccentricity under which the specimens are subjected to with the letter “B” denoting a beam tested under flexural loading. For example, Specimen RS-50 is a concrete column reinforced with steel and tested with the load applied at an eccentricity of 50 mm.

The reinforcement details of all the groups of specimens are shown in Figure 1. The specimens of the first group (Group RS) were considered as a reference group designed with longitudinal and transverse steel reinforcement in accordance with AS3600-2009 (Standards Australia 2009). The concrete standard specifies a minimum longitudinal reinforcement ratio of 1%. For this study the internal longitudinal steel reinforcement ratio was designed as the lowest ratio required by the standard. Therefore, the specimens were designed with four N12 (12 mm deformed bars with 500 MPa nominal tensile strength) as longitudinal reinforcement with a reinforcement steel ratio of 1.03%. The shear reinforcement provided was R10 stirrups (10 mm diameter plain bars with 250 MPa nominal tensile strength) spaced at 50 mm center to center. Specimens of the second group (Group RF) were designed to have similar longitudinal and transverse reinforcement ratios as the specimens in the reference steel group (RS) but instead of steel, these specimens were reinforced with GFRP bars. These specimens were designed in accordance with ACI 440.1R-06 (ACI 2006). The spacing of the shear reinforcement for the two reference specimens was dictated by the design of the GFRP reinforced specimens. The amount of stirrups was chosen to ensure that the beam specimen reinforced with GFRP bars would fail in flexure rather than in shear and a consistent reinforcement arrangement was used for all the reference specimens. The nominal diameter of the longitudinal GFRP reinforcement and transverse reinforcement was 12.7 and 9.5 mm, respectively. The geometry of the stirrups used in the reference specimens is shown in Figure 2.

Specimens of the third group (Group I) were designed with an encased pultruded GFRP I-section. The encased I - section had a nominal height of 152.4 mm, nominal flange width of 152.4 mm and nominal thickness of 9.5 mm. The GFRP I-section was completely encased in the concrete specimens for reinforcing purposes. Similarly, specimens of the fourth group (Group C) were designed with encased GFRP C-sections. The C-sections had a nominal

height of 152.4 mm, nominal flange width of 42.9 mm and a nominal thickness of 9.5 mm, as shown in Figure 1. Two GFRP C-sections were positioned side by side in a box arrangement to serve as encased longitudinal reinforcement and to confine the internal concrete core. The GFRP reinforcement ratio of the Specimens in Group I and C were similar and at a value of approximately 9.44 %. Therefore, a direct comparison of the influence of the shape of encased structural GFRP section on the structural behavior could be analysed from these two groups of specimens. In the design for shear it was assumed that the I-section and C-section would provide some shear resistance in the beam specimens as opposed to the negligible shear resistance of the longitudinal bars in the reference beam specimens. Therefore, specimens in Group I and Group C were provided with steel R10 stirrups (10 mm diameter plain bars) at an increased center to center spacing of 100 mm, as compared to the reference specimens. The main purpose of the stirrups was to: increase the shear capacity of the specimen; confine the concrete core between the web of the I-section and extremities of the specimen; position the C-sections in place as a box arrangement; and to prevent the GFRP sections from outward lateral buckling. It should be noted that the stirrups for these specimens and that of the reference steel specimens were of the same batch but varied with the radius of the bend. The concrete cover at the top and bottom of the specimens was maintained at 20 mm, while the side covers for all the groups of specimens varied slightly depending on the tolerances in the internal reinforcement cages, as shown in Figure 1.

To ensure the failure mode occurred in the instrumented region of the column specimens (at mid-height) and to prevent premature failure, the ends of each specimen were strengthened and confined with two layers of CFRP sheets in the circumferential direction. However, wrapping CFRP sheets around the sharp edges of square specimens has been reported to result in stress concentration and premature failure at these locations (Ozbakkaloglu 2013). Therefore, the top and bottom of the specimens at a length of 100 mm were rounded to

provide a curved round finish to wrap the CFRP sheets. A corner radius of 20 mm was applied at these locations. In addition, for each eccentrically loaded column specimen, two layers of CFRP wrap were applied longitudinally on the tension zone in combination with the two layers wrapped circumferentially to ensure no premature tensile failure occurred at these regions. These longitudinal sheets were 100 mm in length. Furthermore, the four beam specimens were rounded at a radius of 20 mm throughout the length of the specimen. This was done as a precaution if the beam specimens were required to be wrapped by CFRP in the shear zones, at the outer thirds of the beam to ensure failure occurs due to bending rather than shear.

Specimen Preparation

The formwork used for moulding the concrete column and beam specimens was made from 17 mm thick plywood (formply). Before pouring of the concrete, the formwork was vertically fixed to a base and tied together laterally with timber, in order to prevent any movement while pouring and vibrating the concrete. Normal-strength ready-mix concrete with an intended compressive strength of 32 MPa was supplied by a local supplier.

The completed reinforcement cages for the four groups of specimens are shown in Figure 3. The GFRP square stirrups used in the Group RF specimens were manufactured by Pultrall Inc. (2012). GFRP bars cannot be bent after they have been cured (polymerized) and the only way to produce bends was during the manufacturing process. The radius of the bends of the square GFRP stirrups is 12.7 mm. The steel stirrups used in the Group RS specimens were fabricated having similar dimensions to that of the GFRP stirrups but varied in the radius of the bends, as shown in Figure 2. The radius of the bends of the steel stirrups was 40 mm. The difference in corner bend radius of stirrups between the two reference specimens meant that the center to center spacing of the longitudinal bars was slightly greater for the Group RF

specimens as compared to the Group RS specimens. On the other hand, the steel stirrups for Group I and Group C specimens were fabricated in the lab with a corner bend radius of 6.5 mm, to ensure the stirrups fit over the GFRP sections and maintain the required cover.

For the Group I and C specimens, steel stirrups were bonded onto the GFRP sections with silicone at 100 mm center to center spacing. Due to the symmetry of the GFRP sections, hooks could not be made for the steel stirrups. Instead the overlap regions, which were approximately 80 mm long were continuously stitch welded together to ensure the stirrups would provide adequate confinement to the internal concrete core and GFRP sections. The welded overlap regions were positioned to be parallel to the web of the I-section. The encased section of the Group C specimens consisted of two C sections forming a box section as shown in Figure 1. To form this profile, a square wooden piece of the required dimension was temporarily glued with silicone onto two C-sections positioned side by side, at both the top and bottom. Once the GFRP C-sections were ready to be placed into the formwork, the wood pieces holding the two sections in place were removed with the steel stirrups holding the GFRP sections in place. To ensure that the concrete and GFRP sections had adequate bond and no bond-slip occurred, a layer of sand was coated onto the smooth sides of the GFRP I-sections and box C-sections. It was assumed that by coating the largest outside surface area of the sections, an adequate bond would occur between the sections and concrete. Therefore, only the outside faces of both the flanges of the I-section and the long dimension of the C-sections were sand coated along the full height as shown in Figure 4. This was done by first applying a thin layer of epoxy resin on the GFRP sections and then placing coarse sand on the resin.

As mentioned above, for the purpose of avoiding premature failure, the ends of each column specimen were strengthened and confined with two layers of CFRP sheets in the hoop

direction. To avoid stress concentrations from the sharp edges of the specimens, rounded styrofoam (polystyrene) sets of 20 mm radius and 100 mm in length were glued to the internal edges of the formwork with silicone at the top and bottom ends to provide a curved round finish to wrap the CFRP sheets. The connection between this corner radius and the sharp edges of the column specimen created a point of stress concentration which is discussed below. In addition, the four beam specimens were provided with 20 mm radius foam sets throughout the length.

After casting, a wet mesh was placed over all the specimens and was watered daily and covered with plastic sheets to maintain the moisture in the concrete and allow for adequate curing. After 28 days of this curing process, the specimens were removed from the formwork and were wrapped with CFRP as explained above. The adhesive used to bond the CFRP to the concrete was a mixture of epoxy resin and hardener at 5:1 ratio.

Material Properties

All the concrete specimens were cast on the same day with ready-mixed, normal strength concrete provided by a local supplier. The concrete compressive strength was carried out and determined in accordance with AS 1012.9-1999 (Standards Australia 1999). A total of five cylinders having a diameter of 100 mm and height of 200 mm were tested to obtain the 28 day concrete compression strength of 29.3 MPa. In addition, the concrete compression strength at the first day and last day of specimen testing was determined to be 31 MPa and 35.30 MPa, respectively.

Deformed steel N12 bars were used as longitudinal reinforcement in Group RS specimens and plane steel R10 bars were fabricated into square stirrups to serve as transverse reinforcement in Group RS, I and C specimens. Five samples of each diameter were tested in

214 accordance with AS 1391- 2007 (Standards Australia 2007) to determine the tensile
215 properties of the reinforcing steel bars and stirrups as shown in Table 2.

216 Sand coated GFRP bars and GFRP stirrups were used to reinforce the Group RF specimens.
217 No. 4 (#4) GFRP bars of 12.7 mm nominal diameter were used as longitudinal reinforcement
218 and No. 3 (#3) GFRP bars of 9.5 mm nominal diameter were used as square stirrups for the
219 transverse reinforcement. The GFRP reinforcement was supplied with a sand-coated surface
220 to improve the bond performance between the bars and surrounding concrete. Four samples
221 of each diameter were tested in accordance with ASTM D7205-06 (2011) to determine the
222 tensile properties of the GFRP bars, as shown in Table 3. It should be noted that the No. 3,
223 9.5 mm diameter stirrups made by bending GFRP bars have their own production lot number
224 and thus they were not from the same batch as the No. 3, 9.5 mm diameter straight bars that
225 were tensile tested. According to Ehsani et al. (1995) the minimum ratio of radius of bend to
226 the stirrup diameter is three. However, in this study the radius of bend was only 12.7 mm due
227 to manufacturing errors. In addition, according to Pultrall, Inc. (2012) the tensile strength of
228 the GFRP stirrups straight portions is lower than the tensile strength of the corresponding
229 straight bars with the same diameter. However, for the purpose of this study it assumed that
230 the properties of the tested 9.5 mm diameter straight bars are equal to that of the straight
231 portions of the 9.5 mm diameter stirrups. Furthermore, Nanni et al. (1998) reported that the
232 tensile strengths in the bend portion of FRP bars are 40 to 50% lower compared to that of a
233 straight bar due to stress concentrations or fibre bending. Lastly, due to the small dimensions
234 of the GFRP stirrups, the manufacturing process was by hand which resulted in the regions at
235 the bend radius being relatively square in cross section rather than circular which may have
236 posed a further region of stress concentration.

GFRP pultruded I-sections and C-sections were used in the specimens of Group I and C, respectively and they were supplied by GRP Australia (GRP, n.d). These GFRP sections are orthotropic materials with the properties varying in each direction. The fibres are laid mainly in the longitudinal direction, which makes these sections stronger in the longitudinal direction as compared with the transverse direction. The tensile properties of the GFRP pultruded sections were determined based on the test method ISO 527-4 (1997) and are shown in Table 4. Five coupon samples from each GFRP C-section, web of the I-section and flange of the I-section with nominal dimensions of 300 mm long and 25 mm wide were extracted and tested using a screw-driven material testing machine known as the 500 kN Instron 8033 machine. The compressive properties of the GFRP pultruded sections were determined based on the test method ASTM D 695 (2002) and are shown in Table 4. Eleven coupon samples from each GFRP C-section, web of the I-section and flange of the I-section with nominal dimensions of 37.6 mm long and 12.7 mm wide were extracted and tested under compression by direct end loading using the Instron 8033 machine. Five samples each from the C-section and web and flange of the I-section were instrumented with strain gauges to measure the compressive modulus. It should be noted that compression testing of pultruded samples is difficult due to the high longitudinal strength and low transverse strength of the material. As a result, the ends of the coupons failed by splitting and the property measured may not be the actual compressive strength but represent the composite bearing strength. Having said this, the average compressive strength and modulus obtained from testing were higher than those values provided by the supplier (compressive strength of 207 MPa and compressive modulus of 17.2 GPa) and were adopted in this study, although they may not represent the true compressive properties due to premature failure by bearing. Originally, based on the supplier's specifications it was assumed that the GFRP I-section and C-sections had similar

material properties. However, from material testing it can be seen that the two shapes of GFRP sections varied in tensile and compressive properties as shown in Table 4.

Instrumentation and Specimen Testing

All of the specimens were tested with the Denison 5000 kN compression testing machine until failure. The column specimens were capped with high strength plaster at the top and bottom ends to ensure the bearing surfaces were parallel and the load was distributed evenly during testing. The typical compression testing setup of the column specimen is shown in Figure 5. The eccentric load was applied to the column specimen by an eccentric loading system manufactured at the University of Wollongong, as illustrated by Hadi and Widiarsa (2012) and shown in Figure 5. The loading system comprised a set of high strength steel loading heads which were attached to both ends of the columns. The loading heads consisted of two parts: a 50 mm thick square plate, called the adaptor plate and a 25 mm thick bottom plate that had a ball joint, known as the bottom plate. The eccentric load was applied to the column by the interaction of the adaptor plate and bottom plate. The load generated by the testing machine was transferred to the adaptor plate by the bottom plate through the ball joint. The ball joint was offset from the center of the column by the amount of eccentricity required (25 or 50 mm). For columns tested under concentric loading, only the adaptor plate was used to apply the load. The line of application of eccentricity load for the column specimens is shown in Figure 1.

External instrumentation was used to obtain the displacement data of the column specimens. To measure the axial displacement of the column specimens, two linear variable differential transformers (LVDTs) were directly connected to the testing machine at opposite ends. In addition to this, for the eccentrically loaded columns a laser triangulation was positioned horizontally at mid-height of the columns on the tension side in order to measure the lateral

deflections (δ) as shown in Figure 5. The LVDTs and laser triangulation were connected to a data logger to record readings on a control computer at a user controlled time interval. Depending on the amount of data, the time interval for recording of the data varied from 2 to 5 seconds. The data read from the instrumentations were recorded at the same time as the load data were recorded by the testing machine. Prior to the start of testing, calibration was carried out to ensure the specimens were placed at the center of the testing machine and the instrumentations were positioned and operating adequately. The concentrically loaded specimens were initially preloaded to approximately 10 – 20 kN and then tested under displacement control with a loading rate of 0.3 mm/min until failure. On the other hand, the eccentrically loaded specimens were initially loaded to 100 kN (approximately 10 % of the anticipated ultimate capacity) under force control with a loading rate of 2.5 kN/s and then unloaded to a load of 20 kN to ensure adequate contact between the loading plates. The test was then resumed using displacement control with a loading rate of 0.3 mm/min until failure of the specimens.

As highlighted above, four of the specimens were tested as beams under flexural loading in order to determine the flexural capacity. The beam specimens were subjected to pure bending by a four point loading system manufactured at the University of Wollongong, as described by Hadi and Widiarsa (2012). The typical flexural testing setup of the beams is shown in Figure 6. The bottom rig was placed diagonally on the bottom plate of the testing machine with the beam specimen then placed on top of the bottom rig. The top rig was then placed on the top of the beam. The beams were loaded with a pin support at one end and a roller support at the other. In addition, one contact area on the top rig was provided with a roller. To measure the midspan deflection of the beam, a laser triangulation was placed vertically underneath the bottom rig through which the laser is shot through a slot in the bottom rig. The test for the beams initially started with force control at a loading rate of 2.5 kN/s up to a load

of 25 kN (approximately 10% of the anticipated ultimate capacity). This level of preload ensured the proper interaction of the steel rigs and specimen at the start of testing. The test was then continued using displacement control under a loading rate of 0.3 mm/min until failure of the specimens. During testing there were issues with the premature failure mechanism of the beams due to bearing failure. The sections below describe this problem and solutions in detail. For each of the column and beam specimens, the test was terminated either once the load dropped down to approximately 35% of the first maximum peak load or due to the failure of the GFRP reinforcements.

Behavior of Column Specimens

To analyse the structural behavior of the column specimens, the relationship of the applied axial load and the corresponding axial and lateral displacements were plotted. Based on these relationships, the strength and ductility of each group of specimens under different types of eccentric loading were analysed.

Ductility can be defined as the ability of structural material to deform plastically without fracturing. Ductility is commonly measured by the ratio of the ultimate displacement (δ_u) divided by the yield displacement (δ_y), which can be written as follows:

$$\lambda = \frac{\delta_u}{\delta_y} \quad (1)$$

The yield displacement is assumed to be the axial displacement at the yield load or at the limit of the elastic behavior, as defined by Pessiki and Pieroni (1997). Various authors have used different percentages of the peak load to calculate the corresponding ultimate displacement. Pessiki and Pieroni (1997) calculated the ultimate displacement as the axial displacement at an axial load equal to 85% of the peak load in the descending part of the axial

load - displacement curve. Other research studies calculated the ultimate displacement at 80% of the peak load (Sheikh and Legeron 2014).

In this study two methods were used to calculate the ductility of the columns. For the first method the ductility (λ) was calculated based on the ratio of the ultimate displacement (δ_u) divided by the yield displacement (δ_y), as shown in Equation 1. In terms of notation herein, P_{max} is the first maximum load achieved just after the initial linear region and P_{peak} is the peak load after concrete cover spalling and is obtained for columns experiencing a strength increase after concrete spalling ($P_{peak} > P_{max}$). For the steel reinforced columns, no increase in strength occurred after concrete spalling and the ultimate displacement was taken at 80% of P_{max} . However, for the GFRP reinforced and GFRP encased columns, the ultimate displacements was taken at either the first fracture load of the GFRP reinforcement ($P_{fracture}$), at the peak load (P_{peak}) or at 80% of P_{peak} , whichever gave the smallest axial displacement. For GFRP reinforced columns experiencing an increase in strength after concrete spalling it was safer to define the ultimate displacement at this peak load rather than at 80% of peak load considering the unpredictable and sudden brittle failure of the internal GFRP reinforcement after peak load. This is further discussed in the below sections.

For the second method, the ductility was calculated based on the ratio of the area under the axial load-displacement curve up to the ultimate displacement (A_2), divided by the area under the curve up to the yield displacement (A_1), which is written as follows:

$$\lambda = \frac{A_2}{A_1} \quad (2)$$

The same definitions of the ultimate and yield deflections were used for the two methods.

Behavior of Column Specimens under Concentric Loading

The first specimen from each group was tested under concentric loading. The experimental results of the concentrically loaded column specimens are summarized in Table 5 with the axial load-displacement curves shown in Figure 7. Initially, all the specimens experienced similar behavior, with the ascending region of the load-displacement curve being almost linear up to the beginning of concrete spalling. As mentioned above the column edges were rounded at the top and bottom at a width of 100 mm to allow for the wrapping of CFRP sheets. The connection between this corner radius and the sharp edge of the specimen created a point of stress concentration. This resulted in cracks first appearing at the top of the specimen at this weak transition zone. As the test continued the cracks started to propagate to all sides of the specimen and eventually along its instrumented region. After the maximum load (P_{max}) was achieved, the load dropped as a result of the sudden spalling of the concrete cover.

A decrease in maximum load of 4.8% relative to Specimen RS-0 was achieved for Specimen RF-0. Specimen I-0 achieved the highest maximum load of all the concentrically loaded specimens with an increase of maximum load of 5.6% and 10.9% relative to the RS-0 and RF-0 Specimens, respectively. Similarly, an increase of maximum load of 2.6% and 7.8% in comparison to the RS-0 and RF-0 specimens was achieved for Specimen C-0, respectively. After the maximum load was achieved spalling of the concrete cover occurred which resulted in a decrease in the cross-sectional area resisting the load with a corresponding drop in the load carrying capacity of the specimens. After this drop in the capacity the load then stabilised for Specimen RS-0 and RF-0. This meant that the passive confinement provided by the stirrups was activated to prevent the lateral expansion of the concrete core and the specimen was able to sustain the load up until failure.

The cracking appearance and failure modes of the concentrically loaded specimens after failure are shown in Figure 8 and Figure 9. The axial – displacement curve of Specimen RS-0 shows the behavior up until an axial displacement of 30 mm. This was the point at which the data limit of the data logger was exhausted. At this point, the specimen was still able to carry approximately 63% of the maximum load. It should be noted that the test was continued to an axial displacement of 50 mm, as obtained from the machine readings rather than the average readings of the LVDTs. Interestingly at this displacement the specimen was still able to carry approximately 46% of the maximum load, with the load gradually decreasing. After this point the test was terminated and it was observed that after the concrete spalled off, all four longitudinal bars had substantially buckled and some of the ties were deformed and distorted as a result of the concrete core dilating, as shown in Figure 9a. Specimen RS-0 experienced the most ductile behavior of the four specimens. Due to stress concentrations at the rounded corners as mentioned above, the buckling of the bars was predominately at the top third of the specimen.

Unlike Specimen RS-0, Specimen RF-0 failed in a brittle manner as result of the explosive failure of the internal reinforcement at a load of 1125.7 kN and an axial ultimate displacement of 7.72 mm (δ_u). After this failure point the load dropped down drastically. It was unclear whether the first failure was due to the rupture of a stirrup or due to the crushing or buckling and explosive fracture of a longitudinal bar. The rupture of the tie occurred at the bend portion at the connection with a longitudinal bar at mid-height of the specimen. This was anticipated as the bend radius of the ties was 12.7 mm, which was well below the recommended radius based on the required ratio of the bend radius and tie diameter, as reported by others and mentioned above. Furthermore, it was seen that one tie experienced slippage (at the splice locations) of the overlap regions resulting in a potentially inadequate confinement to the concrete core. It should be noted that the GFRP ties were spliced by steel

ties at the overlap regions. The exact timing of the rupture and slippage of the tie could not be established. The test was continued until all four longitudinal bars had failed in compression by complete crushing, with each drop in the load-displacement curve representing the explosive fracture of each bar. At the end of testing there was considerable crushing of the concrete core. As a result during the removal of the steel loading caps by force, the specimen separated into two regions with the four bars split into two, although Figure 9b shows the two regions placed on top of each other for completeness.

The GFRP encased specimens showed little ductile behaviour after the maximum load was reached as compared to the other two specimens. Not long after the maximum load was achieved for Specimen I-0, a small cracking noise was heard on one of the flanges of the I-section at a load of 1257 kN and corresponding axial ultimate displacement of 4.65 mm (δ_u). This load was assumed to be the first failure load of the specimen. At this point, the load dropped with a few more cracking noises being heard as the concrete cover started to completely spall off. Eventually, as the test progressed a larger cracking noise occurred on the opposite flange at a load of 965 kN and axial displacement of 8 mm with a large drop in load occurring. This point is assumed to be the second fracture load. Therefore, on observation it is clearly seen that the two flanges of the I-section at mid-height failed in compression due to the material crushing and delamination of the fibers at the different loads. The crushing occurred along the flange with the combination of axial load and pressure from the concrete core resulting in the crushed area delaminating outwards as shown in Figure 9c. Similarly, Specimen C-0 experienced a brittle failure manner as a result of the progressive crushing of both the GFRP C-sections at different locations after the first maximum load. Just before the first fracture of the GFRP sections, a small cracking noise could be heard, followed by a loud noise. Two other explosions occurred which resulted in the load suddenly dropping each time due to the subsequent damage of the material. Crushing and splitting of the C-sections

occurred on the flange, at the bend radius and on all four short sides as shown in Figure 9d. Again due to the pressure from the concrete core, the crushed areas were pushed outwards. Having said this, data was lost for Specimen C-0 and the exact fracture load and ultimate displacement at this load could not be accurately obtained. Therefore, for the ductility calculations the ultimate displacement was based on 80% of the maximum load as done for the steel reinforced specimen. Upon observation of the GFRP encased specimens after testing, the stirrups at the welded locations were still intact and there were no signs of strength reductions at the overlap region. In fact, no failure of the stirrups was evident and the GFRP sections failed first. Based on this, the welded steel stirrups in the encased specimens served their purpose and did not affect the behavior of the encased specimens as failure of the GFRP sections occurred first.

Behavior of Column Specimens under 25 mm Eccentric Loading

A specimen from each group was subjected to 25 mm of eccentric loading. The experimental results of the 25 mm eccentricity loaded column specimens are summarized in Table 6 with the axial and lateral load-displacement curves shown in Figure 10. Initially Specimen RS-25, I-25 and C-25 experienced similar behavior before reaching maximum load. However, the slope of the load-displacement curve of Specimen RF-25 was lower than that of the other specimens. This could be due to errors in aligning the specimen resulting in load not being applied exactly at 25 mm eccentricity. The first maximum load (P_{max}) of the Specimens RS-25, I-25 and C-25 were approximately equal. However, a decrease in first maximum load of 19.3% relative to Specimen RS-25 was achieved for Specimen RF-25.

The cracking appearance of the specimens at failure in the tension and compression zones is shown in Figure 11. Similar to the concentrically loaded specimens, stress concentrations on the top of the specimens at the transition between the rounded corners and square edges

produced stress concentrations resulting in cracks first appearing in those locations. As testing progressed, the cracks propagated to all four sides of the specimen and to the instrumented region. The application of the eccentric load resulted in the bending of the specimens with one side of the specimen under compression and the other under tension. After the maximum load the concrete in compression started to spall, with horizontal tension cracks originating on the tension side of the specimens. As the load increased the concrete in compression completely spalled off, tension cracks increased and all the specimens failed in compression.

After the maximum load was achieved, the specimens lost a percentage of their maximum capacity due to the sudden spalling of the concrete cover in the compression zone. Specimen RS-25 lost approximately 13% of its maximum value, whereas the Specimen RF-25 lost about 9% of the maximum load. After this drop in the capacity the load carrying capacity of Specimen RS-25 gradually decreased until the eventual termination of the test occurred when the load reached 35% of the maximum load. Specimen RS-25 displaced both axially and laterally the most out of all the specimens, with no sudden failure in the steel occurring providing a good ductile behavior. On termination of the test it was realized that the two longitudinal steel bars on the compression side had substantially buckled as shown in Figure 11b depicting the failure of the specimens in the compression side. Furthermore, the tension cracks at mid-height of the specimen had grown substantially as shown in Figure 11a.

On the other hand, Specimen RF-25 was able to sustain an increase in load after the sudden concrete spalling and eventually a second peak load (P_{peak}) of 823 kN was achieved at an axial ultimate displacement of 8.21 mm (δ_u). After this point, the load decreased at a high rate until the eventual brittle and explosive failure of the internal reinforcement in compression at a load of 353 kN and axial displacement of 19.4 mm. On observation, after this failure, both a

longitudinal bar had fractured and a tie at the bend portion had ruptured on the compression region at the top third of the specimen. It was unclear whether the bar or tie failed first and whether the fracture of the bar was due to crushing or buckling (see Figure 12). Quickly followed after, another tie at the bend portion and the other longitudinal bar failed in a similar manner on the compression side with another drop in the load occurring. Unlike Specimen RS-25, the failure region of Specimen RF-25 was at the top of the specimen which is not in the instrumented region as shown in Figure 12. In addition, the large tension cracks also developed at the top of the specimen rather than at mid-height, with concrete spalling also occurring in tension as seen in Figure 11a. The reason for this as mentioned above may be due to the stress concentrations at the top of the specimen due to the transition of the rounded and sharp edges. Furthermore, another reason is the fracture of the compression bars resulted in these bars not being able to carry anymore load resulting in the top of the specimen bowing towards the compression region. The longitudinal bars in tension did not fail. After termination of the test it was evident that the crushing of concrete core occurred at the top of the specimen. Considering, the unpredictable failure of the GFRP reinforcement after peak load, the ultimate displacement for the ductility definition was taken as the axial displacement at peak load rather than at a higher displacement corresponding to 80% of the peak load.

Although the peak load of Specimen RF-25 (803 kN) was substantially lower than that of Specimen RS-25 (995 kN), the ductility of both specimens was calculated to be approximately similar based on the ductility definition in the above section with the steel reinforced column obtaining a slightly higher ductility. Having said this, the eventual failure of Specimen RF-25 was brittle and explosive with the load dropping substantially after peak load unlike that of Specimen RS-25 which did not fail abruptly but continued to displace and sustain the load until the termination of the test.

Specimens I-25 and C-25 did not displace laterally or axially as much as Specimens RS-25 and RF-25. After the maximum load, the load of Specimen I-25 slightly dropped and then increased up until a second peak load of 1024 kN at which sudden failure occurred (P_{peak} equals $P_{fracture}$). The failure was marked by the crushing and delamination of the compression flange of the I-section at mid-height. The failure mode of the I-sections was similar to that of the concentrically loaded specimen but only occurred in the compression flange. The concrete had not completely spalled at this point but just after the fracture load the concrete in compression was broken apart explosively at mid-height. Similarly, Specimen C-25 failed due to the crushing and rupture of the C-sections in the compression region. Interestingly though after the maximum load was achieved, the load did not decrease suddenly as experienced by the other specimens but instead the load stabilised until a load of 948 kN at which the failure of the C-sections occurred in compression due to material crushing and delaminating. After the fracture point there was a sudden drop in the load with a second drop occurring not long after due to the failure of another section of the C-section. The ultimate displacement of the GFRP encased specimens was taken at the fracture load. After the failure load of the GFRP encased specimens was achieved, testing continued until the load reached 35% of the first maximum load. After observation, it was realised that the tension cracking patterns of the GFRP encased specimens was different to the reference specimens as shown in Figure 11a. These vertical cracks seem to originate at the edges of the sections which could potentially mean inadequate confinement to the concrete core which results in the concrete cover spalling similar to that of concentrically loaded columns. In addition to tensile cracks it is observed that the concrete cover in the tensile region spalled off, which was marked by the vertical cracks. Similar to the concentrically loaded specimens, the GFRP encased specimens showed little ductile behavior after the maximum load was reached as compared to the other two specimens. Specimen C-25 experienced a slightly higher ductility than that of Specimen

I-25, which may be as a result of the confinement provided by the box arrangement of the internal C-sections

Behavior of Column Specimens under 50 mm Eccentric Loading

A specimen from each group was subjected to 50 mm of eccentric loading. The experimental results of the 50 mm eccentrically loaded specimens are summarized in Table 7 with the axial and lateral load-displacement curves shown in Figure 13. Similar to the 25 mm eccentrically loaded specimens, the specimens experienced similar behavior before reaching the maximum load. The maximum load for Specimen RS-50 was obtained to be 747 kN and experienced good ductility. A decrease of first maximum load of 17.7% relative to Specimen RS-50 was achieved for Specimen RF-50. Unlike the other loading cases, Specimen C-50 obtained a lower maximum load as compared to Specimen RS-50. On the other hand, Specimen I-50 experienced a slight increase in maximum load with reference to Specimen RS-0 but experienced low ductility with failure occurring slightly after the maximum load. In addition, Specimen RF-50 experienced a second peak load, with the load carrying capacity increasing after the sudden spalling of the concrete cover. After this point the load substantially decreased until the eventual failure of the GFRP bars in compression at a load of 374 kN and axial displacement of 13.61 mm. After this failure point the specimen could not carry any more load. In terms of ductility and based on the definitions in the above section, the Specimen RF-50 shows a slight improvement in ductility as compared to Specimen RS-50. Furthermore, although the ductility values of the GFRP encased specimens show a reasonable value for ductility, failure was sudden and brittle as opposed to that of Specimen RS-50. Similar to the 25 mm eccentric loading condition, Specimens I-50 and C-50 did not displace laterally or axially as much as the reference specimens.

The failure of the specimens was also observed to be in compression, similar to the failure mechanisms of the 25 mm eccentricity loaded specimens. During or after concrete spalling in the compression side, the steel reinforcement on the compression side buckled and the two GFRP longitudinal bars crushed and explosively fractured, while the reinforcement in tension did not fail. Again due to stress concentrations at the transition of the sharp and round edges, the failure of Specimen RF-50 was at the top of the specimen with the two longitudinal bars completely crushing and separating with no notable rupture in the stirrups (see Figure 14). The tension cracks were larger on the top of the specimen (Figure 14a) with severe crushing of concrete occurring on the top of the specimen at the fracture location of the bars throughout the cross-section in both compression and tension, as shown in Figure 14b and 14c. The CFRP tension strengthening strips were also debonded from the concrete (see Figure 14a).

Sudden failure of the I-section occurred for Specimen I-50 at a second peak load of 769 kN after the maximum load was achieved. The failure mode of the I-sections was similar to that of the 25 mm eccentrically loaded specimen which was marked by the crushing and delamination of the compression flange at mid-height. Similarly, Specimen C-25 failed due to the crushing and rupture of the C-sections in the compression region. Although the failure load for Specimen C-25 of 695 kN was lower than that for Specimen I-50 of 769 kN, the ductility of the C-section encased specimen (3.29) was greater than that of the I-section encased specimen (2.30) which proves the benefits of the confinement mechanism of the C-sections. Furthermore, the cracking appearance on the tension side of the specimens at the end of testing was similar to that of the 25 mm eccentrically loaded specimen with concrete cover appearing to be spalled on the GFRP encased specimens. The GFRP sections in the tension side did not appear to fail during testing.

Behavior of Beam Specimens

The last specimen from each group was tested as a beam under four-point loading. The load versus midspan deflection of the beam specimens are shown in Figure 15. The first specimen tested was Specimen RS-B. Due to a malfunction in the data logger, the midspan deflections obtained from the laser triangulation were lost. Therefore, the midspan deflection for Specimen RS-B was recorded from the readings of the testing machine, with a maximum load of 232 kN achieved at a midspan deflection of 8.1 mm. After the maximum load was reached, the load was maintained until sudden failure occurred in the tension region of the specimen, at a midspan deflection of 20.1 mm. The failure was typical flexural failure with large tension cracks evident at the midspan of the specimen with considerable crushing of the concrete in the compression zone occurring.

The second beam specimen tested was Specimen I-B. At the early stages of loading, bearing failure occurred at the two ends of the specimen with concrete crushing at the two supports, as shown in Figure 16 after testing was terminated. This can be seen from the slight drop and then increase in load on the load versus midspan deflection curve. As a result of this initial bearing failure mode, a lower than expected maximum load of 215.7 kN was obtained at a midspan deflection of 13.47 mm. The test was terminated prematurely due to human errors in setting up the safety precautions of the test. The third specimen tested was Specimen C-B. To ensure no bearing failure occurred, a 50 mm wide by 8 mm thick steel plate was placed as a bearing plate on the two supports. In addition, as added precaution the two ends of the specimen, at 100 mm length, were wrapped with two layers of CFRP wrap as shown in Figure 17. Specimen C-B achieved the highest maximum load of all the specimens, with a load of 370 kN obtained at a midspan deflection of 12.78 mm. Therefore, Specimen C-B achieved an increase of 59.5% in maximum load as compared to Specimen RS-B. At the maximum load there was a drop in the load-deflection curve. After this point, small cracking

noises were heard and subsequent drops in the curve occurred due to possible failure of the C-sections at different areas, as shown in Figure 15.

The last beam specimen tested was Specimen RF-B. Two beams reinforced with GFRP bars were prepared, with one acting as spare. The first beam tested failed in shear and errors in testing occurred with unreliable results. To increase the shear capacity the second beam specimen reinforced with GFRP bars was wrapped with two layers of CFRP at the shear zones at the outer thirds of the specimen with the specimen unwrapped between the two point loads. Again steel bearing plates were provided at the supports. The load-midspan deflection response was almost linear up until a maximum load of 340.3 kN at a midspan deflection of 12.13 mm. At the maximum load the first rupture of the CFRP sheets resulted in a sudden decrease in load. The specimen then still resisted the load under increasing deflection until the second peak load was achieved. Another rupture of the CFRP sheet resulted in a drop of the load. After several ruptures, the bearing plate had shifted as a result of the roller on the supports. The slippage of the bearing plates resulted in the plate being wedged under the specimen and explains the behavior after midspan deflections of 20 mm where no eventual failure occurred in the internal reinforcement of the specimen and the load plateaued out with the test terminated not long after. The addition of the bearing plates would have resulted in slippage at the connection with the pin and roller supports which would have had an impact on the mid-span deflections and load carried. For the purposes of this study these slippages were ignored. By observation, the failure of the steel reinforced specimen was a typical flexural failure. However, the GFRP reinforced and GFRP encased specimens appear to have failed in shear rather than in flexure even though the GFRP reinforced specimen (RF-B) was wrapped with two layers of CFRP in the shear zone. It is unclear whether the steel reinforced specimen would have experienced an increase in load carrying capacity if the steel bearing

pads and ends of the specimen were wrapped with CFRP, which was done for the other beam specimens.

The initial stiffnesses of all the beams were different as shown in Figure 15. Possible reasons for these differences are that some beams were provided with a bearing pad (Group C and RF) while others were not. Other reasons may be the wrapping of the Group RF specimen resulted in a change in the load-midspan behavior with respect to the other specimens or the small ratio of the shear span to depth ratio of the specimens resulted in inconsistency in testing. Therefore, the ductility of the beam specimens were not determined and analysed.

Conclusion

A total of 16 square specimens, twelve as columns and four as beams, were tested in this study to investigate the type of internal reinforcement (steel bars, GFRP bars and encased GFRP structural sections) and magnitude of load eccentricity on the compressive and flexural behavior of concrete specimens. Based on the experimental results of this study, the following conclusions were drawn:

1. The column specimens of Group RS achieved a higher load carrying capacity as compared to the column specimens of Group RF for all loading conditions. The load carrying capacity of the Group RF column specimen loaded concentrically (RF-0) was 4.8% lower than its steel counterpart. On the other hand, the load carrying capacity of the Group RF column specimens loaded eccentrically (RF-25 and RF-50) were on average 18.5% lower than their steel counterparts. Therefore, a higher drop in load carrying capacity was experienced for the eccentrically loaded GFRP reinforced column specimens with respect to the equivalent steel reinforced column specimens.

2. For concentrically loaded columns, the column specimens of Group RS achieved a better ductile performance compared to the column specimens of Group RF. For eccentric loading conditions, column specimens of Group RF achieved similar ductility as compared to the column specimens of Group RS, based on the ductility definition in the paper. However, the eventual failure mechanism of the GFRP reinforced column specimens was brittle and sudden in nature, whereas the steel reinforced column specimens did not fail abruptly but continued to displace until the termination of the test.

3. The GFRP encased column specimens achieved a higher load carrying capacity but lower ductility as compared to the other reference column specimens. Group C column specimens experienced better ductility to that of Group I column specimens which is attributed to the confinement effect of the C-sections box arrangement. Having said this, the Group I column specimens achieved a slightly higher load carrying capacity as compared to the Group C column specimens.

4. Based on the results of the beam specimens the use of encased GFRP structural sections can provide a significant improvement in the load carrying capacity when comparing conventional beams reinforced with steel and GFRP bars. There is potential in encasing structural GFRP sections in concrete beams although further research elaboration is necessary to investigate this considering some of the errors and premature failure mechanisms experienced in the experimental program of the beam specimens.

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771 **Table 1.** Experimental Test Matrix

Group	Test Specimen	Longitudinal Reinforcement				Transverse Reinforcement			Encased Sections		Test Eccentricity (mm)
		Material	Number of bars	Diameter (mm)	Reinforcement Ratio (%)	Material	Diameter (mm)	Spacing (mm)	Material	Type	
1	RS-0	Steel	4	12	1.03	Steel	10	50	-	-	0
	RS-25										25
	RS-50										50
	RS-B										Bending
2	RF-0	GFRP	4	12.7	1.15	GFRP	9.5	50	-	-	0
	RF-25										25
	RF-50										50
	RF-B										Bending
3	I-0	-	-	-	-	Steel	10	100	GFRP	I-section	0
	I-25										25
	I-50										50
	I-B										Bending
4	C-0	-	-	-	-	Steel	10	100	GFRP	C-sections	0
	C-25										25
	C-50										50
	C-B										Bending

772 **Table 2.** Tensile Properties of the Steel Bars

Bar	Reinforcement	Diameter (mm)	Average tensile	Average tensile	Average tensile
			yield strength f_y (MPa)	yield strain ϵ_y (%)	modulus, E (GPa) ^a
R10	Transverse	10	326 ^b	0.370 ^b	191.7
N12	Longitudinal	12	540	0.324	199.8

773 ^a Calculated as the slope of the elastic linear region of the stress-strain relationship.

774 ^b Determined by the 0.2% Offset Method

775 **Table 3.** Tensile Properties of the GFRP Bars (averages and sample standard deviations)

Bar Size	Nominal Diameter (mm)	Standard cross- sectional Area (mm ²)	Cross-sectional area by Immersion Test (mm ²) ^a	Tensile strength, f_{tu} (MPa) ^b	Tensile rupture strain, ϵ_f (%) ^b	Tensile modulus, E (GPa) ^{b,c}
#3	9.5	70.9	134.9	1855 ± 60	2.39 ± 0.12	77.6 ± 1.1
#4	12.7	126.7	190.7	1641 ± 73	2.41 ± 0.10	67.9 ± 1.3

776 ^a Determined in accordance with ASTM D7205-06 (2011)

777 ^b The material properties calculated are based on the bars standard cross-sectional area determined by the nominal diameter.

778 ^c Calculated as the slope of the elastic linear region of the stress-strain relationship.

779 **Table 4.** Tensile and compressive properties of the GFRP pultruded structural sections (averages and sample standard deviations)

Property	I-section – WEB	I-section – FLANGE	C-section
	Longitudinal	Longitudinal	Longitudinal
Tensile Strength (MPa)	386.5 ± 17.4	430.2 ± 32.7	318.1 ± 31.8
Tensile Modulus (GPa)	20.7 ± 0.7	26.0 ± 2.2	26.2 ± 2.0
Tensile Rupture Strain (%)	1.94 ± 0.08	1.75 ± 0.17	1.22 ± 0.11
Compressive Strength (MPa) ^a	213.1 ± 27.6	236.9 ± 28.5	304.9 ± 80.1
Compressive Modulus (GPa)	22.3 ± 2.2	25.1 ± 3.1	25.0 ± 2.7
Compressive Rupture Strain (%) ^a	-	-	-

780 ^a Bearing failure occurred at top and bottom of coupon tested in compression resulting in potential premature failure. See Material Properties
781 section.

782 **Table 5.** Experiment results of column specimens tested under concentric loading

Test Specimen	Max. Load	Axial displ. at	Yield Load	Axial displ. At	Fracture Load	Ultimate Displ.	Ductility Method	Ductility Method
	$P_{\max}$ (kN)	$P_{\max}$ (mm)	P_{yield} (kN)	P_{yield} (δ_y (mm))	P_{fracture} (kN)	δ_u (mm)	1 ^d	2 ^d
RS-0	1350	2.87	1122	1.68	-	15.24 ^a	9.07	16.80
RF-0	1285	2.59	1089	1.58	1126	7.72 ^b	4.89	8.37
I-0	1425	3.13	1173	1.86	1258	4.65 ^b	2.50	3.98
C-0	1385	3.24	1199	2.27	- ^c	7.33 ^c	3.23	5.19

783 ^a The displacement at the 80% of $P_{\max}$

784 ^b The displacement at the fracture of the GFRP reinforcement

785 ^c Data was lost and the fracture load of the GFRP C-section could not be obtained. Therefore the ultimate displacement was based on 80% of
786 $P_{\max}$, although failure did occur before this point but could not be accurately determined.

787 ^d Refer to section “Behavior of Column Specimens” for definitions of the methods

788

789 **Table 6.** Results of column specimens tested under 25 mm eccentric loading

Test Specimen	1 st Max. Load $P_{\max}$ (kN)	Axial displ. at $P_{\max}$ (mm)	Lateral displ. At $P_{\max}$ (mm)	Yield Load P_{yield} (kN)	Axial displ. At P_{yield} δ_y (mm)	Second Peak Load, P_{peak} (kN)	Fracture Load P_{fracture} (kN)	Ultimate Displ. δ_u (mm)	Ductility Method 1 ^d	Ductility Method 2 ^d
RS-25	995	2.72	2.11	904	2.13	-	-	8.04 ^a	3.77	5.96
RF-25	803	3.00	2.27	701	2.20	823	353	8.21 ^b	3.62	6.10
I-25	1008	2.51	2.05	905	2.00	1024	1024	4.97 ^c	2.49	3.96
C-25	985	2.86	2.96	866	2.03	-	948	5.68 ^c	2.80	4.65

790 ^a The displacement at the 80% of $P_{\max}$

791 ^b The displacement at P_{peak}

792 ^c The displacement at the first fracture of the GFRP reinforcement

793 ^d Refer to section “Behavior of Columns Specimens” for definitions of the methods

794 **Table 7.** Results of column specimens tested under 50 mm eccentric loading

Test Specimen	1 st Max. Load $P_{\max}$ (kN)	Axial displ. at $P_{\max}$ (mm)	Lateral displ. At $P_{\max}$ (mm)	Yield Load P_{yield} (kN)	Axial displ. At P_{yield} δ_y (mm)	Second Peak Load, P_{peak} (kN)	Fracture Load P_{fracture} (kN)	Ultimate Displ. δ_u (mm)	Ductility Method 1^d	Ductility Method 2^d
RS-50	747	2.65	2.66	672	2.02	-	-	7.55 ^a	3.74	5.94
RF-50	615	2.33	2.46	558	1.77	626	374	9.44 ^b	5.33	9.44
I-50	765	2.88	3.18	688	2.19	769	769	5.04 ^c	2.30	3.56
C-50	679	3.04	3.69	607	2.08	695	695	6.84 ^c	3.29	5.67

795 ^a The displacement at the 80% of $P_{\max}$

796 ^b The displacement at P_{peak}

797 ^c The displacement at the first fracture of the GFRP reinforcement

798 ^d Refer to section “Behavior of Column Specimens” for definitions of the methods

799

Fig.1

Fig. 2

a) Steel stirrups in Group RS Specimens

b) GFRP stirrups in Group RF Specimens

Fig. 3

(a) Side - View

(b) Cross - sections

Fig. 4

Fig.5

(a) Loading system

(b) Eccentric loading heads

(c) Interaction of the eccentric loading heads

Fig. 6

Fig. 7

Fig. 8

Fig. 9

(a) Specimen RS-0

(b) Specimen RF-0

(c) Specimen I-0

(d) Specimen C-0

Fig. 10

Fig. 11

(a) Tension side

(b) Compression side

Fig. 12

Fig. 13

Fig. 14

(a) Tension side

(b) Side- View

(c) Crushing of compression bars

Fig 15.

Fig 16.

Fig. 17