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Numerical Investigations on the Flexural Behavior of GFRP-RC Beams under Static Loads

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Abstract

The behaviour of Glass Fibre-Reinforced Polymer (GFRP) bar reinforced concrete beams varies significantly from the behaviour of traditional steel bar reinforced concrete beams. This paper numerically investigates the response of GFRP bar reinforced concrete (GFRP-RC) beams under static loads. This paper also presents the details of a three-dimensional Finite Element (FE) model for GFRP-RC beams under static loads. The results of the numerical modelling have been validated against the experimental results of nine GFRP-RC beams. The results of the FE analysis have been found to be in very good agreement with the experimental results. Furthermore, an extensive parametric study is carried out to investigate the effects of the reinforcement ratio, compressive strength of concrete, and shear span to effective depth ratio on the response of GFRP-RC beams. The effects of these parameters on the load-midspan deflection behaviour, energy absorption capacity, and failure modes of GFRP-RC beams have been adequately discussed in this paper.

Keywords: reinforced concrete; beam; failure mode; GFRP; modelling

1 Introduction

Reinforced Concrete (RC) structures are prone to the corrosion of steel reinforcing bars caused by chloride and alkali ions present in aggressive marine and corrosive environments. Corrosion of steel reinforcing bars in RC structures decreases the lifespan of structures and increases the costs of repair and maintenance [1]. During the last few decades, Fibre-Reinforced Polymer (FRP) bar has emerged as a suitable replacement for the steel reinforcing bar in RC structures due to its corrosion resistance, chemical resistance, electromagnetic neutrality, high strength-to-weight ratio, competitive life cycle cost, and fatigue resistance [2-5]. Structures reinforced with FRP bars are particularly suitable in aggressive marine and corrosive environments.

Reinforcing bars of FRP include Glass FRP (GFRP), Basalt FRP (BFRP), Aramid FRP (AFRP), and Carbon FRP (CFRP) bars. The most popular type of reinforcing bar is GFRP bar due to its competitive price and abundance. In the last few decades, a significant amount of research has been carried out to investigate the behaviour of GFRP bar reinforced concrete (GFRP-RC) beams under static loads. However, the behaviour of GFRP-RC beams differs significantly from the behaviour of traditional steel bar reinforced concrete (Steel-RC) beams. GFRP materials are anisotropic, which affects the shear strength and bond performance of GFRP bars with concrete. In addition, unlike steel bars, GFRP bars do not yield. Hence, a change in the traditional design philosophy is adopted for GFRP-RC beams [6-8]. Furthermore, GFRP-RC beams experience larger deflections due to the lower modulus of elasticity of GFRP (40-70 GPa) than the modulus of elasticity of steel (200 GPa). The recommended failure mode for GFRP-RC beams is concrete crushing since it is less brittle than GFRP bar rupture and provides more ductility to the GFRP-RC beam [6-8]. The flexural responses of GFRP-RC beams have been investigated experimentally in the literature [9-22]. Experimental investigations focused mainly on the load-midspan deflection behaviour, failure modes, cracks propagation, cracks pattern, cracks width, and Energy Absorption Capacities (EAC) of GFRP-

RC beams. El-Nemr, et al. [16] investigated the behaviour of normal strength and high strength GFRP-RC beams and reported that an increase in the compressive strength of concrete leads to a decrease in the crack width and deflection and an increase in the ultimate load-carrying capacity of GFRP-RC beams. Adam, et al. [9] investigated the behaviour of GFRP-RC beams with different longitudinal reinforcement ratios and concluded that increasing the reinforcement ratio increases the ultimate capacity of the beam. Nonetheless, there have been no extensive experimental parametric studies in the literature that investigate the effects of a wide range of variables such as the compressive strength of concrete, longitudinal reinforcement ratio, and geometry of the GFRP-RC beams. The most significant barriers that hinder conducting extensive experimental parametric studies include the large cost and time associated with these parametric studies. Numerical simulations, on the other hand, are cost and time effective and have been effectively used in the literature to carry out parametric studies [23-25].

In the last few decades, Finite Element Method (FEM) has emerged as one of the most efficient tools to replicate the experimental response of RC beams under different loading conditions. One of the most significant advantages of using FEM is the ability to analyse failure modes, load-midspan deflection behaviours, cracks pattern, cracks width, and Energy Absorption Capacity (EAC) of RC beams. Another advantage of using FEM is that it can be used to provide guidelines prior to experimental investigations. The finite element code LS-DYNA [26, 27] is a very powerful and efficient tool due to its comprehensive material library and its ability to capture the non-linear response of RC beams under static loads. Yet, only a few numerical studies investigated the behaviour of GFRP-RC beams under static loads [9, 28-30]. The numerical studies available in the literature focused on the load-midspan deflection behaviour and damage of GFRP-RC beams. However, there is a need to analyse how various factors influence the behaviour of GFRP-RC beams. There are no systematic parametric study

available in the literature that investigates the influence of reinforcement ratios, compressive strengths of concrete, and shear span to effective depth ratios on the behaviour of GFRP-RC beams. This paper numerically investigates the influence of reinforcement ratios, compressive strengths of concrete, and shear span to effective depth ratios on the maximum midspan deflections, load-midspan deflection curves, failure modes, and EAC of GFRP-RC beams using the finite element code LS-DYNA.

2 Numerical investigations

The finite element code LS-DYNA has been used for numerical modelling of GFRP-RC beams. Due to its computational power and comprehensive material library, LS-DYNA has been extensively used in the literature to model the behaviour of RC beams [31-34].

2.1 Structural geometry

In order to replicate the experimental conditions, a three-dimensional (3D) FE model was created accounting for boundary and loading conditions. Figure 1 shows a 3D model of the GFRP-RC beam. To represent concrete and the experimental setup including the supports and the steel I-beam (used in the experimental program to apply the four-point load), eight-node solid hexahedron elements with single point integration were used. The single point integration saves computational time in complex problems. However, one of the disadvantages of the use of single point integration is the presence of hourglass modes. Hourglass modes are nonphysical, zero-energy modes of deformation that produce zero strain and no stress. To control and minimize the hourglass energy, Belytschklo-Bindeman hourglass control was chosen for implicit analysis. To represent GFRP and steel reinforcement, 2D Hughes-Liu beam elements with 2x2 Gauss quadrature integration were used. The 3D model was divided into three main parts: the beam, the steel I-beam, and the supports. For selecting a suitable mesh size, a separate convergence study was carried out considering a mesh aspect ratio of 1.

Different mesh sizes including 20, 15, 10, and 5 mm were modelled and analysed. It was found that with the reduction of the size of the mesh below 10 mm increased the computational time significantly for a minor increase in the accuracy of the results. Hence, a mesh size of 10 mm was chosen for this study, which provided both efficiency and accuracy. Furthermore, the reinforcement was modelled using a slide line one-dimensional model in LS-DYNA.

2.2 Contact and boundary conditions

To model the contact between the steel I-beam (used in the experimental program to apply the load), concrete, and supports, several formulations are available in LS-DYNA. Automatic contact [26, 27] between surfaces was used in this study. In order to replicate the actual boundary conditions of the experiment, the boundary conditions for this study were defined as a pinned and roller support. The pinned support was restrained from all translations and allowed to rotate about its major axis, whereas the roller support was restrained from all translations except along the major axis and allowed to rotate about its major axis. Moreover, to model the bond-slip between the reinforcement and concrete, a one-dimensional contact model (Contact_1D) was used [26]. This approach was successfully used in previous research studies [35-38]. The constitutive relationship between the bond shear stress and the slip is given by Eq. (1) [26, 27]

$$\tau = \begin{cases} G_s s, & s \leq s_{ma:} \\ \tau_{max} e^{-hD}, & s > s_{ma:} \end{cases} \quad (1)$$

where, τ is the bond shear stress, G_s is the bond shear modulus, s_{max} is the maximum elastic slip, h is the damage curve exponential coefficient, and D is the damage parameter. The values of these parameters were taken according to the experimental investigation results. The loading condition imposed on the GFRP-RC beams was deflection controlled.

2.3 Control options for solver

The advantage of implicit analysis, adopted in this study, is the unconditionally stability of the solution allowing larger values of time steps. The code LS-DYNA offers options for choosing automatic control over the time step. In this study, the maximum time step and optimum equilibrium iteration count were restricted to allow for convergence of the results. Here, if the number of iterations required to achieve equilibrium is larger than the optimum count (suggesting that non-linearity is increasing), the solver reduces the current time step. Whereas, if the number of iterations required to achieve equilibrium is smaller than the optimum count, the solver increases the current time step to reach the maximum.

2.4 Materials

Concrete

Winfrith Concrete Model [26] is used in this study to model the concrete. Winfrith Concrete Model is a smeared crack, smeared rebar model, implemented in the eight-node single integration point continuum element [26]. This model has been developed over many years by Broadhouse and Neilson [39] and Broadhouse [40]. Winfrith Concrete Model was initially developed to model structures subjected to impacts and blasts. Nonetheless, it has the ability to capture the behaviour of concrete under static loads. The Winfrith model has been validated with extensive experimental investigations and has been proven successful in capturing the complex behaviour of concrete [31, 32, 41, 42]. The input card of this model consists of the mass density, initial tangent modulus, Poisson's ratio, uniaxial compressive and tensile strengths, and aggregate size of concrete. The tangent modulus of concrete was calculated as specified by ACI [6] ($E_c = 4700\sqrt{f'_c}$). The Winfrith model has the ability to generate crack algorithms. Winfrith model does not include a criterion for erosion where elements can be deleted. Hence, erosion was added to concrete using an independent function to delete the

elements after satisfying the erosion condition. Elements were deleted by specifying a minimum principal strain of 0.35% for concrete at failure. The principal strain of 0.35% is considered failure strain for concrete in compression by CSA [7]. Although Winfrith offers a smeared rebar model, the reinforcement was modelled using Piecewise Linear Plasticity [26], as explained in the next section.

Reinforcement

The material model used to capture the behaviour of the reinforcement was Piecewise Linear Plasticity. This material model input card requires a minimum entry of density, modulus of elasticity, Poisson's ratio, yield stress, tangent modulus, and failure plastic strain. The Piecewise Linear Plasticity was used to model both steel and GFRP reinforcement in this study. This model is usually used to model steel reinforcement due to the ability to enter the yield stress and failure strain values. Figure 2 shows the stress-strain curve of the GFRP bar using Piecewise Linear Plasticity model. GFRP bars do not yield, instead they rupture. To simulate this behaviour numerically, a tangent modulus of zero ($E_{TAN}=0$ in Figure 2) is specified, creating an elastic-perfectly-plastic behaviour for GFRP bars. Then, a pseudo-plastic strain, with a value equal to the rupture strain of the GFRP bar, was chosen so that the GFRP bar fails as soon as it enters the plastic phase. Specifying a very small value for the plastic strain failure ensures that the bars fail as soon as the yield stress is reached. This approach ensures that a linear behaviour of GFRP bars takes place up till failure. Moreover, once the failure strain is reached, the beam element, representing the reinforcement, is deleted from the calculation.

Supports and loading plates

The Rigid material model was used to replicate the behaviour of the supports and steel I-beam for the four-point bending load (refer to the next section). In order to capture the contact between the supports, steel I-beam, and concrete, this material model requires input of density, modulus of elasticity, and Poisson's ratio. The I-beam and the supports in this study were

constituted of steel. Therefore, the values of the density and Poisson's ratio of the I-beam and the supports were chosen as those of steel.

3 Overview of experimental investigations

This study includes the experimental investigations of nine GFRP-RC beams under static loads to determine the flexural capacity of these beams. In addition, four specimens were tested under static loads to determine the bond-slip properties of the GFRP bars in concrete. The full details of the experimental investigations of the nine GFRP-RC beams can be found elsewhere [18, 43]. However, for completeness, a brief description of the experimental results is presented.

For the validation of the numerical model, nine GFRP-RC beams were tested under static loads. All beams were 100 mm in width, 150 mm in height and 2400 mm in length. The clear concrete cover was 20 mm. The distance between the supports was 2000 mm. The beams were divided into three main groups according to the compressive strengths of concrete. The compressive strengths of concrete on the day of testing for Groups A, B, and C of GFRP-RC beams were 55.4, 70.8, and 90.1 MPa, respectively (Table 1). Furthermore, each group included three beams with different longitudinal reinforcement ratios. The numbers 1, 2, and 3 following the group name indicate the longitudinal reinforcement of the tested GFRP-RC beams. Number 1 indicates that the GFRP-RC beam was reinforced longitudinally with 2 #2 bars on the tension side and 2 #2 bars on the compression side. The number 2 indicates that the GFRP-RC beam was reinforced longitudinally with 2 #3 bars on the tension side and 2 #3 bars on the compression side. The number 3 indicates that the GFRP-RC beam was reinforced longitudinally with 2 #4 bars on the tension side and 2 #4 bars on the compression side. For example, Beam C2 indicates that the compressive strength of concrete for this beam is 90.1 MPa and that the reinforcement comprises 2 #3 bars in tension and 2 similar bars in compression. All the beams had transverse reinforcement of 4 mm steel bars spaced at 100 mm

centre-to-centre. All GFRP-RC beams were designed to fail in flexure with the ratio of shear resistance to bending resistance larger than one. Thus, all the beams were flexure-critical. Groups A and B beams were tested under four-point loads, whereas beams belonging to Group C beams were tested under three-point loads. Based on the classification of ACI [6], Beams B1 and C1 were under-reinforced, Beam A1 was balanced, and the remaining six beams (A2, A3, B2, B3, C2, and C3) were over-reinforced. The variables of the experiment were the longitudinal reinforcement ratio, compressive strength of concrete, and loading type (three-point and four-point loading). Table 1 presents the dimensions, average compressive strength of concrete on the day of testing, reinforcement ratio, and failure modes of the GFRP-RC beams tested. Moreover, results of the modulus of elasticity, ultimate strength, and rupture strain of the GFRP bars used are presented in Table 2. The beams rested on a pin support and a roller support with a clear span of 2000 mm. For the three-point bending test, the load was applied at the midspan of the beam, whereas for the four-point bending test, the load was applied at a distance of $\frac{l_e}{3} = 667$ mm from the supports.

In order to determine the bond-slip properties of the GFRP bars in concrete, four GFRP-RC specimens were tested in accordance with RILEM [44]. The first two specimens had #3 (9.53 mm) embedded GFRP bars. The two other specimens had #4 (12.7 mm) embedded GFRP bars. The average compressive strength of concrete was measured as 59 MPa. However, the compressive strength of concrete does not significantly influence the bond-strength of the GFRP bars in concrete especially for concrete of compressive strength over 30 MPa [45]. The specimens were made of two parts, as shown in Figure 3. Each part was 100 mm in width, 180 mm in depth, and 375 mm in length and was separated by a 50 mm hinge. The specimens were tested using the 5000 KN Instron machine at 1 mm/min until the failure of the specimens. The failure of the specimens was considered as the sudden drop in the load and a major slip in the

GFRP bar occurs. The slip of the GFRP bars was measured using LVDTs and the strains in the bars were measured via strain gauges attached at the middle of the GFRP bars.

For the #3 (9.53 mm) GFRP bars, the average maximum bond stress was 38.45 MPa, the average maximum slip was 0.14 mm, and the average exponential decay [37] was 0.11. Whereas, for the #4 (12.7 mm) GFRP bars, the average maximum bond stress was 31.3 MPa, the average maximum slip was 0.11 mm, and the average exponential decay was 0.15. The #2 (6.35 mm) GFRP bar was not tested due to unavailability of the GFRP bar during testing. It was reported in the literature [46, 47] that the bond strength increases with the decrease of the bar diameter. Therefore, the bond-slip properties of the #2 GFRP bars were assumed the same as the bond-slip properties of #3 GFRP bars for the numerical model.

3.1 Failure modes

In traditional Steel-RC beams, the preferred failure mode is yielding of steel reinforcement prior to concrete crushing. However, FRP bars do not yield. Rupture of bars in FRP-RC beams is sudden which leads to a brittle failure. Hence, the design philosophy of GFRP-RC beams differs significantly from that of Steel-RC beams. The failure mode of under-reinforced GFRP-RC beams occurs when GFRP bars rupture and the beam suddenly collapses. The failure mode of balanced GFRP-RC beams occurs when GFRP bars rupture simultaneously with concrete crushing at compression. Also, the failure mode of over-reinforced GFRP-RC beams occurs when concrete at compression fails. Under-reinforced and balanced failure modes of GFRP-RC beams provide no warning prior to failure. These failure modes are described as sudden and brittle. Design codes [6-8] recommend an over-reinforced design for GFRP-RC beams to provide more ductility with an adequate warning prior to failure. The failure modes of GFRP-RC beams can be identified by analysing the strains in the reinforcement and concrete at failure. For example, if the strain in the concrete reaches the failure strain, as specified by the design

code, while the strain in the GFRP bar reinforcement is below the failure strain, then the failure is governed by concrete crushing.

Under-reinforced failure

Under-reinforced GFRP-RC Beams B1 and C1 failed due to GFRP bar rupture. Beam B1 was tested under four-point load and C1 was tested under three-point load. Prior to cracking of concrete, both beams exhibited an elastic response and the load was resisted by the gross cross-section. After cracking of concrete, GFRP bars started resisting the applied load. The strain in concrete and GFRP bars kept gradually increasing until the strain in GFRP bars reached the failure strain. The GFRP bars ruptured at the midspan without adequate warning. As a result of this rupture, the GFRP-RC beams were split into two parts and this was identified as the failure mode of under-reinforced beams. Beams B1 and C1 showed no reserve capacity and the failed in a brittle manner. The GFRP ruptured bars could be identified clearly. Figure 4a presents the image of the failed Beam B1.

Balanced failure

The balanced Beam A1 failed due to simultaneous rupture of GFRP bar and concrete crushing. Similar to the Beams B1 and C1, the GFRP bars started resisting the load after cracking of the concrete. The four-point bending load gradually increased causing cracks to widen in concrete. The strain in both the GFRP bars and concrete at compression kept increasing until failure of the beam. The measured strain in concrete was $\varepsilon_{cu} = 0.0029$. Beam A1 showed no reserve capacity after rupture of the GFRP bars and the ruptured bars were visible. The simultaneous failure of concrete and GFRP bars indicated a balanced failure, where this was considered the failure mode of balanced beams. The drop in the load-carrying capacity of the beam was evident at the rupture of the GFRP bars in Beam A1.

Over-reinforced failure

The six remaining GFRP-RC Beams A2, A3, B2, B3, C2, and C3 were over-reinforced beams, where concrete crushing governed the failure of these beams. The over-reinforced failure mode is the recommended mode by the design codes [6, 7] due to the higher ductility provided by this failure mode in comparison with the under-reinforced and balanced failure modes. At the beginning of the loading, the beams exhibited an elastic response. After cracking, the GFRP bars started resisting the applied load. As the applied load increased, the existing cracks widened and propagated upwards. The testing continued until failure of the beams where a drop in the load-carrying capacities of the beams was observed. Figure 4b shows the failure mode of Beam B2 (Only Beam B2 was presented in Figure 4b since Beams B2 and B3 have the same failure mode). The strain measured in concrete at compression was in the vicinity of $\varepsilon_{cu} = 0.003$, whereas the strain in the GFRP bars was lower than the rupture strain. The beams showed reserve capacity and were able to resist more loads due to the confinement effect provided by the stirrups. Moreover, since the GFRP bars did not rupture, the beams were able to carry additional tensile loads.

3.2 Load-midspan deflection

The load-midspan deflection graph of all GFRP-RC beams can be described as bilinear (Figures 5-7). The first part of the curve represents the pre-cracking stage that exhibited a high bending-stiffness. In this first part of the curve, the uncracked section started resisting the gradually increasing load. As the load increased, cracks started appearing on the tension side of the GFRP-RC beams. The load at which cracks appeared was referred to as P_{cr} . The corresponding midspan deflection at this load was referred to as the cracking deflection, Δ_{cr} . After the cracking load, a significant drop in the bending stiffness of the beam took place. This can be attributed to the low modulus of elasticity of GFRP bars. A transformation in the second

moment of area from the gross second moment of area, I_g , to the cracking second moment of area, I_{cr} , took place. Hence, the bending stiffness of the GFRP-RC beams decreased. The bending stiffness for the uncracked section, EI_g , was higher than the bending stiffness of the cracked section, EI_e . The second part of the curve showed the behaviour of the cracked section of the GFRP-RC beam. The load applied on the beam kept increasing with the increase in the midspan deflection until failure of the beam where the drop in the load at the first peak was evident. The load-midspan deflection graph at the second part can be described as linear until failure. The load measured at failure (as per the three specified failure modes in 3.1) of the beam was referred to as ultimate load, P_u . For example, for an under-reinforced and a balanced beam, the ultimate load is the load at rupture of the GFRP bar. The measured values of cracking and ultimate loads and midspan deflections were presented in Table 3. However, for comparison with the numerical investigation, the failure of the over-reinforced beams in this study was considered up to the first peak in the load-midspan deflection graph.

It was observed that all load-midspan deflection curves, regardless of the reinforcement ratio and compressive strength of concrete, exhibited a bilinear behaviour as described above. Furthermore, it was observed that the ultimate load resisted by the GFRP-RC beams increased with an increase in the reinforcement ratio. The reinforcement ratio had a significant influence on the ultimate load resisted by the GFRP-RC beams. An increase in reinforcement ratio from 1% to 2%, (Beams A2 and A3) led to a 27% increase in the ultimate load. Moreover, an increase in the reinforcement ratio from 0.5% to 1%, (Beams A1 and A2) led to an increase of almost twice in the ultimate load. The significant increase in the ultimate load can be attributed to the increase in the reinforcement ratio that caused a shift in the failure mode of the GFRP-RC beam from balanced failure to concrete crushing. Therefore, the reinforcement ratio had a significant influence on the ultimate load resisted by a GFRP-RC beam.

It was observed that the compressive strength of concrete had less influence on the load-midspan deflection behaviour of GFRP-RC beams. The increase in the compressive strength of concrete of GFRP-RC beams from 50.4 MPa to 70.8 MPa (Beams A3 and B3) for beams having the same reinforcement ratio of 2% led to an increase in the ultimate load of 15%.

3.3 Cracks profile

The GFRP-RC beams tested were designed as flexure-critical beams with the ratio of the shear-resistance to bending-resistance larger than one. It was observed that all GFRP-RC beams, regardless of their longitudinal reinforcement ratio and compressive strengths, developed cracks propagating vertically in the beams. The shear cracks that were observed were minor and the major contribution of cracks was influenced by flexural cracks. Hence, this confirms the predictions that GFRP-RC beams that were designed as flexure-critical beams failed in flexure.

4 Comparison between numerical and experimental results

Table 3 presents the numerical and experimental cracking and ultimate loads and corresponding midspan deflections, Energy Absorption Capacity (EAC), and bending stiffness of the GFRP-RC beams.

4.1 Failure modes

The numerical models using LS-DYNA show the ability to capture the failure modes of GFRP-RC beams. As described before, the failure modes of GFRP-RC beams were identified by analysing the strains in the reinforcement and concrete at failure. The under-reinforced beams failed due to GFRP bar rupture prior to concrete reaching its ultimate strain ($\varepsilon_{frp} = \varepsilon_{frpu}$ while $\varepsilon_c < \varepsilon_{cu}$). The balanced beam failed due to simultaneous GFRP bar rupture and concrete at compression reaching its ultimate strain ($\varepsilon_{frp} = \varepsilon_{frpu}$ and $\varepsilon_c = \varepsilon_{cu}$). The over-

reinforced beams failed due to concrete crushing with the strain in concrete reaching the ultimate strain and the strain in GFRP bar lower than the rupture strain ($\varepsilon_c = \varepsilon_{cu}$ while $\varepsilon_{frp} < \varepsilon_{frpu}$). The FEM accurately captures the failure modes of all GFRP-RC beams. The ultimate load for under-reinforced and balanced beams is the load at which the GFRP bars rupture. Whereas, the ultimate load for an over-reinforced beam is the load when the strain of concrete at compression reaches $\varepsilon_{cu} = 0.0035$ as specified by CSA [7].

The models analysed in LS-DYNA show that the under-reinforced Beams B1 and C1 failed without prior warning due to rupture of GFRP bars (Figure 4a). As the load gradually increased, the strains in the GFRP bars and the concrete increased as well. When the strain in the GFRP bars reached the failure strain, the bars ruptured and the beams failed. This was represented numerically by deletion of the beam element at the midspan representing the GFRP reinforcement. This led to a sudden drop in the load-carrying capacities of these beams showing no or very limited ductility. The GFRP bars ruptured and the beam was split into two parts with spalling of concrete (erosion of solid elements) at the midspan.

Moreover, the balanced Beam A1 failed due to simultaneous rupture of GFRP bars and crushing of concrete in compression. As the applied load increased, the strain in the reinforcement and concrete increased as well. The GFRP bars ruptured when reaching the failure strain and a sudden drop in the load-carrying capacity of the beam was observed. Moreover, since the strain in concrete at compression reached the failure strain, the failure mode can be described as balanced.

As for the remaining over-reinforced beams, all the models showed that these beams failed due to concrete crushing (Figure 4b). The strains in the GFRP bars and the concrete increased with the applied load. The FEM matches the experimental observations where the failure occurred at a strain of $\varepsilon_{cu} = 0.0035$. Moreover, this strain matches the failure strain of concrete

specified by CSA [7]. In addition to that, the strain measured in the GFRP bars was lower than the failure strain.

4.2 Load-midspan deflection behaviour

A comparison between the numerical and experimental load-midspan deflection curves of the GFRP-RC beams is presented in Figures 5-7. It can be observed from Figures 5-7 that the FEM is in very good agreement with the experimental results. The FEM shows the ability to capture the bilinear response of the GFRP-RC beams. Furthermore, the FEM captures the cracking loads and midspan deflections and also, the high bending stiffness of the uncracked section. Moreover, the FEM also captures accurately the ultimate loads and midspan deflections of GFRP-RC beams, and similarly, the bending stiffness of the cracked section.

The under-reinforced Beams B1 and C1 in addition to the balanced Beam A1 all experienced similar load-midspan deflection behaviours. When the strain in the GFRP bars reaches the failure strain, the bars rupture and a sudden drop in the load-carrying capacity of the beam was observed. The load-midspan deflection graphs of the beams shown in Figure 5a, Figure 6a, and Figure 7a all show the drop in the load-carrying capacity of the beams upon rupture of the GFRP bars. The FEM shows the ability to replicate the experimental conditions and model the rupture of the bars and the sudden drop in the load-carrying capacity of the beams. As for the over-reinforced beams, it can be observed from the load-midspan deflection graphs that there are some small discrepancies between the numerical and experimental trends of the midspan deflection corresponding to the ultimate load. The experimental values of the midspan deflections decrease as the compressive strength of concrete increases, whereas the numerical analysis showed an increase in the midspan deflection corresponding to the ultimate load with an increase in the compressive strength. This is due to the differences between the experimental and numerical failure strain of concrete. While the numerical failure strain was fixed at $\epsilon_{cu} = 0.0035$, the experimental failure strain varied causing the drop in the load-carrying capacity to

occur at a smaller deflection. In addition, the over-reinforced GFRP-RC beams show reserve capacity even after reaching the failure strain in concrete. It should be noted, however, that in this study the response of the GFRP-RC beams under static loads was modelled up to the ultimate strain in concrete ($\varepsilon_{cu} = 0.0035$) as specified by CSA [7].

The EAC of GFRP-RC beams is calculated as the area under the load-midspan deflection curve. The EAC of the under-reinforced and balanced beams was significantly lower than that of the over-reinforced beams. The shift in the failure mode from GFRP bar rupture to concrete crushing increases the ultimate load resisted by a GFRP-RC beam. Hence, the EAC of over-reinforced beams is higher than that of under-reinforced. Table 3 presents the EAC, calculated both experimentally and numerically, of all GFRP-RC beams. It is observed from the numerical analysis that the EAC increases with an increase in the reinforcement ratio and the compressive strength of concrete. However, the opposite was observed in the experiment. This is due to the experimental strain in concrete at failure being higher than 0.35%, whereas the numerical strain at failure was 0.35%.

The bending stiffness was calculated by using Eq. (2) for three-point bending and Eq. (3) for four-point bending:

$$EI = \frac{Pl_e^3}{48\Delta} \quad (2)$$

$$EI = \frac{P(l_e) \left(3l_e^2 - 4 \left(\frac{l_e}{3} \right)^2 \right)}{48\Delta} \quad (3)$$

Where P is the applied load, l_e is the span of the beam, and Δ is the midspan deflection of the GFRP-RC beam.

The values of the bending stiffness for the cracked (EI_{post}) and uncracked (EI_{pre}) sections are presented in Table 3. The experimental and numerical values of the bending stiffness seem to be in good agreement. As the reinforcement ratio of the GFRP-RC beam increases, the post-bending stiffness increases as well. This is due to the larger load and smaller deflection of a GFRP-RC beam as the reinforcement ratio increases.

4.3 Crack profile

One of the strengths of Winfrith Concrete Model is the ability to simulate cracks in concrete. The crack profiles of balanced Beam A1 and over-reinforced Beam A2 are presented in Figure 8. It can be noted that all beams failed in flexure, as predicted, with flexural cracks propagating vertically across the beams. The FEM was able to capture the flexural cracks initiated at the bottom portion of the beam and propagating vertically upwards.

4.4 Summary of comparison

Table 4 presents the ratios of the numerical to experimental results. It is observed from Table 4 that FEM matched very well with the experimental results for all GFRP-RC beams in terms of the cracking, ultimate loads, and corresponding midspan deflections, bending stiffness, and EAC. In particular, FEM matches best with the experimental results for the over-reinforced beams. The numerical to experimental ratios were in the vicinity of one for all GFRP-RC beams.

It is noted that the ratio of numerical to experimental EAC of the balanced Beam A1 is 1.66. This can be attributed to the premature failure of the beam in the experimental investigations. Another interpretation may be the differences in the GFRP bar properties (modulus of elasticity, ultimate strength, and failure strain). A small variation in the GFRP bar properties can affect the failure load of under-reinforced and balanced GFRP-RC beam, which may lead to higher or lower ultimate loads. On the other hand, in terms of the pre-cracking bending

stiffness, the FEM values of Beams A2 and C1 significantly vary from the experimental values. This is attributed to the difficulty in accurately identifying the cracking load and deflection numerically since the load-midspan deflection curves are not perfectly bilinear.

5 Parametric study

5.1 Description of the parametric study

After the validation of the numerical modelling and proving its ability to capture the behaviour of GFRP-RC beams under static loads, four comprehensive parametric studies are carried out to investigate the effect of longitudinal reinforcement, compressive strength of concrete, shear span to effective depth ratio, and modulus of elasticity of the GFRP bars on the response of GFRP-RC beams under static loads. The purpose of these investigations is to explore the effect of the longitudinal reinforcement ratio, compressive strength of concrete, shear span to effective depth ratio, and modulus of elasticity of GFRP bars on the load-midspan deflection behaviour, EAC, and failure modes of GFRP-RC beams under static loads. Moreover, taking advantage of the symmetry, quarter models were employed with appropriate symmetry boundary conditions to model the beams in the parametric study. It is noted that, unless otherwise specified, the properties of the GFRP bars were taken as the guaranteed properties provided by V-Rod and Pultrall [48] with a modulus of elasticity of 59 GPa , ultimate strength of 900 MPa, S_{max} of 0.15 mm, and G_s of 111 MPa/mm. The value of the exponential decay of bond slip response was taken as per the recommendation of Shi, et al. [37].

5.2 Influence of longitudinal reinforcement ratio

To investigate the effect of longitudinal reinforcement ratio, GFRP-RC beams with different reinforcement ratios were modelled using LS-DYNA. The beams were 200 mm in width, 300 mm in height, and 2400 mm in length. The reinforcement used comprised two bars in the tension side and two similar bars in the compression side. The reinforcement ratios of the beams

were 0.5%, 1%, 1.5%, 2%, 2.5%, and 3%. To cover a range of values for the compressive strength of concrete f'_c was varied between 30 MPa and 60 MPa. The group name for these beams was R followed by a number indicating the reinforcement ratio and then another number indicating the compressive strength of concrete. For example, Beam R-1.5-30 indicates that the beam is reinforced with 1.5% longitudinal reinforcement and has a compressive strength of concrete of 30 MPa. All the beams were modelled for the behaviour under four-point static loads. The shear reinforcement of these beams comprised 10 mm steel stirrups spaced at 100 mm centre-to-centre which ensures that these beams were flexural-critical beams.

In terms of the failure modes of the GFRP-RC beams of Group R, the beams were modelled as under-reinforced, balanced, and over-reinforced. Beams R-0.5-50 and R-0.5-60 were under-reinforced and failed with the rupture of the GFRP bars. A sudden drop in the load-carrying capacity of the beam occurred and failed in a sudden manner. Furthermore, Beams R-0.5-30 and R-0.5-40 were balanced and the failure mode was simultaneous rupture of GFRP bars and crushing of concrete. All the remaining beams were over-reinforced. The failure mode of these beams was governed by concrete crushing. It was clear that at failure, the strain in the concrete reached the failure strain while the strain in the GFRP bars was lower than the failure strain of the GFRP bar. For example, Beam R-2-40 failed when the strain in concrete reached the failure strain, $\epsilon_{cu} = 0.0035$, whereas the strain in the reinforcement was lower than the failure strain, ϵ_{frpu} . The maximum measured strain at the midspan of the GFRP bars was $\epsilon_{frp} = 0.68\%$, which was less than the failure strain, $\epsilon_{frpu} = 1.5\%$. Beams with reinforcement ratio of 0.5% were under-reinforced and balanced. Increasing the reinforcement ratio from 0.5% to 1% changed the beams to over-reinforced beams. Increasing the reinforcement ratio from 0.5% to 1% influenced the failure mode of the beams where a change in the failure mode from GFRP bar rupture to concrete crushing took place with the increase in the reinforcement ratio.

In terms of the load-midspan deflection behaviour, it can be observed from Figure 9a that the longitudinal reinforcement ratio had a prominent influence on increasing the ultimate load of GFRP-RC beams. It can be observed from Figure 9a that beams R-0.5-40, R-0.5-50, and R-0.5-60 have the same ultimate load of 155 KN. This was due to the fact that increasing the compressive strength of concrete from 30 MPa changed the beam from balanced to under-reinforced. Once the load applied reached 155 KN, the GFRP bars ruptured. Furthermore, increasing the reinforcement ratio from 0.5% to 1% led to a significant increase in the ultimate load (35% to 67%). This significant increase can be attributed to the change of the failure modes from under-reinforced and balanced to over-reinforced failure modes. The GFRP-RC beams were able to sustain larger loads when the failure was governed by concrete crushing. However, for over-reinforced beams, as the reinforcement ratio increased, the percentage of increase in the maximum load decreased. An increase in the reinforcement ratio from 2.5% to 3% led to a 4% increase in the maximum load. Similar observations can be drawn regarding the midspan deflection corresponding to the ultimate load of the GFRP-RC beams. Figure 9b presents the midspan deflections corresponding to the ultimate loads of the GFRP-RC beams of Group R. The midspan deflections corresponding to the ultimate loads of the Beams R-0.5-40, R-0.5-50, and R-0.5-60 is 28.5 mm. This was due to the GFRP bars rupturing upon reaching this deflection. Moreover, it can be observed that increasing the reinforcement ratio from 0.5% to 1% led to a 25% decrease in the midspan deflections corresponding to the ultimate loads. This can be attributed to the change of failure mode from GFRP bar rupture to concrete crushing. Furthermore, it can be observed that as the percentage of reinforcement increased for over-reinforced beams, the percentage of decrease in the deflection corresponding to the ultimate load decreased. Increasing the reinforcement ratio from 2.5% to 3% led to a 10% decrease in the deflections corresponding to the ultimate loads of these beams. Figure 9c presents the EAC of GFRP-RC beams of Group R. The EAC was calculated as the area under

the load-midspan deflection curve of the beam. The EAC was affected by the ultimate load and the corresponding midspan deflection. For the GFRP-RC beams that were over-reinforced, increasing the reinforcement ratio led to a decrease in the EAC of the beam. For example, Beam R-1-30 had an EAC of 1890 J, whereas Beam R-1.5-30 had an EAC of 1790 J. However, the increase in the EAC in Figure 9c, between the beams of reinforcement ratio of 0.5% and 1%, occurred because of the change in the failure modes (from GFRP bar rupture to concrete crushing). It was also observed that the highest EAC calculated was for beams with a reinforcement ratio between 1%-1.5%.

In conclusion, an increase in the longitudinal reinforcement ratio leads to an increase in the ultimate load, a decrease in the midspan deflection corresponding to the ultimate load, and EAC of the GFRP-RC beam.

5.3 Influence of compressive strength of concrete

To investigate the effect of the compressive strength of concrete, GFRP-RC beams with different compressive strengths of concrete were modelled using LS-DYNA. The beams were classified into four main groups according to their compressive strengths. The compressive strengths of concrete of these beams were 30, 40, 50, and 60 MPa. The beams were 200 mm in width, 300 mm in height, and 2400 mm in length. To cover a range of reinforcement ratios, each main group included four reinforcement ratios of 0.5%, 1%, 2%, and 3%. The group name for these beams was M followed by a number indicating the compressive strength of concrete and then another number indicating the reinforcement ratio. For example, Beam M-40-3 indicates that this beam has a compressive strength of 40 MPa and is reinforced with 3% as a longitudinal reinforcement. All the beams were tested under four-point static loads. The shear reinforcement of these beams comprised 10 mm steel stirrups spaced at 100 mm centre-to-centre which ensures that these beams were flexural-critical beams.

In terms of the failure modes, Group M included under-reinforced, balanced, and over-reinforced beams. Beams M-50-0.5 and M-60-0.5 were under-reinforced, whereas Beams M-30-0.5, M-40-0.5 were balanced. All the remaining beams were over-reinforced. The under-reinforced and balanced GFRP-RC beams failed due to GFRP bar rupture which was evident through the sudden drop in the load-carrying capacity. For the over-reinforced beams, it was clear that at failure, the strain in the concrete reached the failure strain $\varepsilon_{cu} = 0.0035$ while the strain in the GFRP bars was below the failure strain. For Beam M-40-2, the maximum measured strain was $\varepsilon_{frp} = 0.78\%$, which was less than the failure strain $\varepsilon_{frpu} = 1.5\%$. Moreover, the maximum measured strain in the GFRP bars decreased as the compressive strength of concrete increased.

In terms of the load-midspan deflection behaviour, the results are discussed in terms of cracking load, ultimate load, corresponding midspan deflection, and EAC. The cracking load, P_{cr} , is directly proportional to the square root of the compressive strength of concrete. In the case of four-point load, P_{cr} can be calculated by using Eq. (4):

$$P_{cr} = \frac{6M_{cr}}{l} = \frac{6}{l} \left(\frac{0.62\sqrt{f'_c} I_g}{y_t} \right) \quad (4)$$

Increasing the compressive strength of concrete, led to an increase in the cracking load of a GFRP-RC beam. This was observed from the numerical modelling as well where the value of the cracking load increased with the increase in the compressive strength of concrete. Moreover, the ultimate load that a GFRP-RC beam increased with an increase in the compressive strength of concrete. Figure 10a presents the ultimate loads for GFRP-RC beams as a function of compressive strengths of concrete between 30 MPa and 60 MPa for four different reinforcement ratios. Beams with 0.5% reinforcement ratio showed an increase in the ultimate load of 6.7% when the compressive strength of concrete increased from 30 MPa to 40

MPa. However, further increase in the compressive strength of the concrete did not lead to any increase in the ultimate load. This can be attributed to the failure modes of the beams where the beams with the same ultimate load failed due to GFRP bar rupture. Beams that failed due to concrete crushing sustained higher loads upon increasing the compressive strength of concrete. This leads to the conclusion that as the compressive strength of concrete increases, for over-reinforced beams, the ultimate load carried by a beam increases as well. A similar observation can be drawn for the midspan deflections corresponding to the ultimate loads. Figure 10b presents the deflections corresponding to the ultimate loads for GFRP-RC beams as a function of compressive strengths of concrete between 30 MPa and 60 MPa for four different reinforcement ratios. For the beams with a reinforcement ratio of 0.5%, increasing the compressive strength of concrete from 40 MPa to 60 MPa did not cause any significant change in the value of the deflection corresponding to the ultimate load. Beams failing due to GFRP bar rupture failed at the same deflection corresponding to the ultimate load, of 28.7 mm, regardless of the compressive strength of concrete. In terms of the EAC, Figure 10c presents the EAC for GFRP-RC beams as a function of compressive strengths of concrete between 30 MPa and 60 MPa for four different reinforcement ratios. The EAC of the GFRP-RC beams was calculated by integrating the area under the load-midspan deflection curve. Since the EAC is related to the ultimate load and corresponding midspan deflection, then beams with reinforcement ratio of 0.5% have the same values for EAC of 3080 J. Moreover, as the compressive strength of concrete increased, the EAC of the beams increased as well in an approximately linear manner. For the beams with reinforcement ratio of 1%, the increase from 30 MPa to 40 MPa was 18%, and the increase from 45 MPa to 60 MPa was 22%. Furthermore, it was observed that beams with a reinforcement ratio between 1% and 2% sustained the highest EAC.

5.4 Influence of modulus of elasticity of GFRP bars

To investigate the effect of the modulus of elasticity, E_f , of GFRP bars, GFRP-RC beams with different E_f were modelled using LS-DYNA. The typical values of E_f of the GFRP bars as specified by ACI [6] are between 35 GPa and 51 GPa. Moreover, Hasan, et al. [49] tested GFRP bars with E_f up to 76.8 GPa. Therefore, the E_f of the GFRP bars chosen for this study was 30, 40, 50, 60, 70, and 80 GPa. The ultimate strength of the GFRP bars was fixed at 900 MPa. The compressive strength of these beams was 30 MPa. The reinforcement ratio of these beams was either 2% or 3 %. The group name for these beams was E, followed by a number indicating the modulus of elasticity of the GFRP bars and then another number indicating the reinforcement ratio. For example, Beam E-40-2 indicates that this beam has GFRP bars of modulus of elasticity of 40 GPa and a reinforcement ratio of 2%. All the beams were over-reinforced and flexure-critical. The purpose of this study was to investigate the effect of E_f on the ultimate load, corresponding midspan deflection, and EAC of the GFRP-RC beams.

In terms of the failure modes, all the beams failed due to concrete crushing. It was observed that the beams with higher E_f had lower values of strain in the GFRP bars when concrete failed. For example, Beam E-30-2 had a strain $\varepsilon_{frp} = 0.87\%$ when the strain in concrete reached $\varepsilon_{cu} = 0.0035$. Whereas Beam E-80-2 had a strain in the GFRP bars of $\varepsilon_{frp} = 0.36\%$ when the strain in concrete reached $\varepsilon_{cu} = 0.0035$.

Figure 11a presents the ultimate loads of GFRP-RC beams for variable values of E_f . It was observed that as E_f increased, the ultimate load of a GFRP-RC beam increased as well. For example, at $\varepsilon_{cu} = 0.0035$, Beam E-30-3 experienced an ultimate load of 248 KN, whereas Beam E-80-3 experienced an ultimate load of 284 KN. This increase of 15% was due to the increase in the modulus of elasticity of the GFRP bars. It was also observed that for high values of E_f (70 and 80 GPa), the increase in the ultimate load was very low. The increase in E_f from

70 to 80 GPa led to a 2% increase in the ultimate load only. The opposite observation can be drawn for the values of the midspan deflections corresponding to the ultimate loads. It was observed that these deflections decreased as E_f increased, Figure 11b. Beam E-30-3 experienced a midspan deflection of 14.74 mm, whereas Beam E-80-3 experienced a midspan deflection of 7.6 mm. This significant decrease of 48% between Beam E-30-3 and Beam E-80-3 was due to the higher modulus of elasticity of the GFRP bars. The GFRP bars with lower values of E_f allow the beams to undergo larger deflections. Moreover, the increase in E_f from 70 to 80 GPa led to an 8% decrease in the deflection corresponding to ultimate load. Furthermore, the EAC of these beams was calculated and a comparison was presented in Figure 11c. It can be observed from these figures that the EAC for all GFRP-RC beams decreased with an increase in the E_f of the GFRP bars. Beam E-30-3 experienced an EAC of 2360 J, whereas Beam E-80-3, with lower E_f , experienced an EAC of 1205 J.

In conclusion, the modulus of elasticity of the GFRP bars influences the midspan deflection corresponding to the ultimate load of the beams the most (48% decrease from E-30-3 to E-80-3), whereas it has a minor influence on the ultimate load of the beam (12.8% increase from E-30-3 to E-80-3). The EAC is significantly influenced by E_f as well (43.6% decrease from E-30-3 to E-80-3).

5.5 Influence of shear span to effective depth ratio

To investigate the effect of shear span to effective depth ratio, GFRP-RC beams with different shear span to effective depth ratio were modelled using LS-DYNA. The variable in this study was the span length of the beam, while all the other parameters were kept constant. The “ a/d ” ratio was altered between 4 and 8 by changing the length of the beam and keeping the effective depth fixed. All five beams were tested under four-point static loads. The beams were 200 mm in width and 300 mm in height. The compressive strength of concrete was fixed to 40 MPa.

Moreover, the longitudinal reinforcement ratio of these beams was 2%. The transverse reinforcement of these beams comprised 10 mm steel bars spaced at 100 mm centre-to-centre. Hence, the beams were designed to fail in flexure. All the beams were designed as over-reinforced beams with concrete crushing governing the failure mode of these GFRP-RC beams.

It was noted that decreasing the a/d ratio increases the ultimate load carried by the GFRP-RC beams. It can be noted from Figure 12 that an increase in the a/d ratio from 4 to 5 led to a 12% decrease in the ultimate load carried by the beam. Furthermore, increasing the a/d ratio increased the midspan deflection corresponding to the ultimate load of the beam linearly. Figure 12 shows that an increase in a/d ratio from 5 to 6 led to a 30% increase in the midspan deflection corresponding to the ultimate load of the GFRP-RC beam. This decrease in maximum load and increase in maximum deflection was attributed to the geometry of the beam where the maximum moment at the midspan of the beam increases with an increase in the span of the beam. It is recommended to use low a/d ratios (in the vicinity of 4) in the design of the GFRP-RC beams, if possible.

6 Conclusions

Nine GFRP-RC beams have been numerically modelled and analysed to validate the ability of the numerical model in capturing the experimental behaviour of GFRP-RC beams under static loads. The numerical model developed has the ability to accurately capture all three different failure modes of the GFRP-RC beams (under-reinforced, balanced, and over-reinforced). The numerical model developed has the ability to predict accurately the load-midspan deflection behaviour of GFRP-RC beams including the ultimate load, corresponding midspan deflection, and EAC. Therefore, it is recommended to use numerical investigations prior to experimental investigations for guidelines. After the validation, a comprehensive parametric study has been carried out to study the influence of the longitudinal reinforcement ratio, the compressive

strength of concrete, the modulus of elasticity of the GFRP bars, and the a/d ratio of the GFRP-RC beams on the ultimate loads, corresponding midspan deflections, and Energy Absorption Capacity (EAC) of GFRP-RC beams. The following conclusions are noted:

1. The GFRP-RC beams with reinforcement ratio of 0.5% experienced brittle failure modes due to GFRP bar rupture. Increasing the reinforcement ratio from 0.5% to 1% leads to significant increase in the ultimate load (67%), EAC (48%), and also leads to a (27%) decrease in the midspan deflection corresponding to the ultimate load of GFRP-RC beams due to the change in failure modes from GFRP bar rupture to concrete crushing. Moreover, the highest EAC were observed for reinforcement ratios between 1% and 1.5%.
2. The GFRP-RC beams with low reinforcement ratios (0.5% - 1%) and high compressive strength of concrete (60 MPa) fail in a brittle manner due to GFRP bar rupture. Increasing the compressive strength of concrete (from 30 MPa to 60 MPa) of the over-reinforced GFRP-RC beam leads to an (40%) increase in the ultimate load, corresponding midspan deflection, and (64% increase) EAC of the GFRP-RC beam. Moreover, increasing the compressive strength of concrete was more effective with reinforcement ratios between 1% and 2%. Also, for the under-reinforced GFRP-RC beams failing due to rupture of GFRP bar, the ultimate load and deflection do not change with the increase of the compressive strength of concrete.
3. Increasing the modulus of elasticity of the GFRP bars (from 30 GPa to 80 GPa) leads to a significant decrease in the midspan deflection corresponding to the ultimate load and a decrease in the EAC of GFRP-RC beams (48.4% and 43.6% respectively). Increasing the modulus of elasticity also leads to an increase in the ultimate load (12.8%). Also, increasing the modulus of elasticity beyond 70 GPa, for over-reinforced beams GFRP-RC beams, has a minor influence on the ultimate load and EAC.

4. Increasing the a/d ratios of the GFRP-RC beams (between 4 and 8) decreases the ultimate load and increases the corresponding midspan deflection linearly. Beams with a/d ratios in the vicinity of four performed the best in terms of ultimate loads and deflections.

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Figure 1, Three-dimensional model of a GFRP-RC beam

Figure 2, Numerical stress-strain curve for GFRP bar

Figure 3, Bond-slip beam test

(a)

(b)

Figure 4, Damage profile: (a) under-reinforced beam (Beam B1) and (b) over-reinforced beam (Beam

B2)

Figure 5, Load-midspan deflection behaviour of Group A beams

Figure 6, Load-midspan deflection behaviour of Group B beams

Figure 7, Load-midspan deflection behaviour of Group C beams

Number of elements cracked=10106

(a)

Time = 0.4435
Number of elements cracked=5136

(b)

Figure 8, Crack profile: (a) balanced beam (Beam A1) and (b) over-reinforced beam (Beam A2)

(a)

(b)

(c)

Figure 9, Effect of reinforcement ratios in GFRP-RC beams on: (a) Load, (b) Midspan deflection, and (c) Energy Absorption Capacity

(a)

(b)

(c)

Figure 10, Effect of compressive strength of concrete in GFRP-RC beams on: (a) Load, (b) Midspan deflection, and (c) Energy Absorption Capacity

(a)

(b)

(c)

Figure 11, Effect of E_f of GFRP bars on: (a) Load, (b) Midspan deflection, and (c) Energy Absorption Capacity

Figure 12, Effect of a/d ratios of GFRP-RC beams ($f'_c = 40$ MPa and $\rho = 2\%$) on load and midspan deflection

Table 1 Properties of the GFRP-RC beams tested and modelled

Beam name	Dimensions of the beam (mm)	Compressive strength of concrete f'_c (MPa)	Longitudinal reinforcement details (Similar for tension and compression)	Transverse reinforcement details	Test condition	Design failure mode
A1	100x150x2400	55.4	2 bars #2	4 mm steel bars @ 100 mm	Four-point loads	Balanced (GFRP rupture & concrete crushing)
A2			2 bars #3			Over-reinforced (Concrete crushing)
A3			2 bars #4			Over-reinforced (Concrete crushing)
B1		70.8	2 bars #2			Under-reinforced (GFRP rupture)
B2			2 bars #3			Over-reinforced (Concrete crushing)
B3			2 bars #4			Over-reinforced (Concrete crushing)
C1		90.1	2 bars #2		Three-point loads	Under-reinforced (GFRP rupture)
C2			2 bars #3			Over-reinforced (Concrete crushing)
C3			2 bars #4			Over-reinforced (Concrete crushing)

Table 2 Properties of the GFRP and steel reinforcement used in the experiment

Type of reinforcement bar	Diameter of the reinforcement bar (mm)	Modulus of elasticity E_f (GPa)	Tensile strength f_u (MPa)	Rupture strain ϵ_{frpu} (%)	Density (T/mm ³)	Poisson's ratio
GFRP	6.35 (#2)	37.5	732	1.96	2.2e-9	0.26
GFRP	9.53 (#3)	55.6	1764	3.18	2.2e-9	0.26
GFRP	12.7 (#4)	48.6	1605	3.30	2.2e-9	0.26
Steel	4	200	500	N/A	7.85e-9	0.30

Table 3 Numerical versus experimental results of GFRP-RC beams

Beam name	Numerical							Experimental						
	P_{cr} (KN)	Δ_{cr} (mm)	P_u (KN)	Δ_u (mm)	E_a (J)	$(EI)_{pre}$ (KN. m ²)	$(EI)_{post}$ (KN. m ²)	P_{cr} (KN)	Δ_{cr} (mm)	P_u (KN)	Δ_u (mm)	EAC (J)	$(EI)_{pre}$ (KN. m ²)	$(EI)_{post}$ (KN. m ²)
A1	4.3	1.6	18.1	70.2	900	381.7	36.6	4.2	1.8	13.8	52.2	540	331	37.5
A2	4.74	0.82	41.33	59.4	1387	820.9	95.5	3.1	0.8	39.2	60.4	1350	550	92.1
A3	2.5	0.5	49.31	48.54	1335	710	132.3	2.3	0.44	49.7	59.9	1730	742	117.8
B1	4.09	1.1	17.9	65.7	725	528	38.7	3.5	0.8	15.5	54.5	530	621	40.4
B2	4.68	0.91	48.35	68.6	1859	720	102.3	4.0	0.9	42.6	56.3	1350	631	107.4
B3	4.5	0.85	57.82	56.07	1787	751.6	148.5	3.8	0.8	49.5	47.3	1270	674.4	148.6
C1	3.87	2.35	15.2	66.1	635	233.9	32.65	3.1	0.9	15.0	81.8	735	489	26
C2	4.2	1.56	37.34	60.96	1246	382.3	87.5	3.8	1.7	33.0	62.7	1220	317.4	105
C3	4.5	1.36	44.95	49.55	1196	469.9	123	4.0	1.6	46.1	58.3	1510	355	112.3

Note: P_{cr} : Cracking load; P_u : Ultimate load; Δ_{cr} : Midspan deflection corresponding to cracking load; Δ_u : Midspan deflection corresponding to ultimate load

Table 4 Efficiency of numerical modelling

Beam name	Numerical/Experimental						
	P_{cr}	Δ_{cr}	P_u	Δ_u	E_a	$(EI)_{pre}$	$(EI)_{post}$
A1	1.02	0.9	1.3	1.3	1.66	1.15	0.98
A2	1.5	1.02	1.05	0.98	1.02	1.49	1.03
A3	1.09	1.02	0.99	0.81	0.77	0.96	1.1
B1	1.17	1.37	1.15	1.2	1.36	0.85	0.96
B2	1.17	1	1.13	1.21	1.37	1.15	0.95
B3	1.18	1.06	1.16	1.18	1.41	1.1	0.99
C1	1.24	2.6	1.01	0.81	0.86	0.48	1.2
C2	1.1	0.92	1.13	0.97	1.02	1.2	0.81
C3	1.13	0.85	0.97	0.85	0.79	1.3	1.1
Average	1.18	1.2	1.1	1.04	1.14	1.08	1.02