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Abstract

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Experimental Investigations on the Behavior of GFRP bar Reinforced HSC and UHSC Beams under Static and Impact Loading

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ABSTRACT

This paper presents an experimental investigation into the behavior of Glass Fiber-Reinforced Polymer (GFRP) bar reinforced high strength concrete and ultra-high strength concrete beams. In total, twelve GFRP bar reinforced concrete beams (GFRP-RC beams) were constructed and tested. Six GFRP-RC beams were tested under static loading. Higher strength concrete was found to influence the overall behavior of GFRP-RC beams under static loading in terms of load carrying capacity, deflection, and post-cracking bending stiffness. Six GFRP-RC beams were tested under impact loading at various levels of impact energy. The GFRP-RC beams displayed a shift in the failure mode (from shear failure to flexure failure) as a result of the use of ultra-high strength concrete under impact loading.

Author Keywords: Reinforced Concrete; GFRP; Beams; Impact; High Strength Concrete; Ultra High Strength Concrete

1. Introduction

Durability, corrosion resistance, and blast and impact resilience are the current requirements for high-performance reinforced concrete (RC) structures. Understanding and modeling of concrete behavior under extreme environmental loading conditions are essential for making RC structures safer and more efficient. In particular, the corrosion of reinforcement can expedite the aging process and deterioration of the infrastructure. The aging and deterioration process of the infrastructure may cause aesthetic problems together with significant financial implications resulting from increased maintenance cost. Hence, it is an important challenge for structural engineers to design structures to resist extreme loads in harsh environmental conditions.

To overcome corrosion related damage and deterioration, Fiber-Reinforced Polymer (FRP) bars are considered to be an alternative option for reinforcing concrete structures as opposed to conventional steel reinforcement [1-3]. FRP bars possess non-corrosive behavior, which makes it viable to reinforced concrete structures in coastal environments. Furthermore, FRP bars have a high-strength to weight ratio, making it easy to transport on site. Commercially available FRP bars include glass (Glass Fiber Reinforced Polymer, GFRP), carbon (Carbon Fiber Reinforced Polymer, CFRP), aramid (Aramid Fiber Reinforced Polymer, AFRP) and basalt (Basalt Fiber Reinforced Polymer, BFRP). The FRP bars are well known to have linear stress-strain behavior up until failure under uniaxial tension, with no or limited

ductility, unlike the conventional steel reinforcement. Also, FRP bars have a low modulus of elasticity (e.g., 35 GPa -51 GPa, according to ACI [4]). FRP bars are relatively expensive compared to steel reinforcement. However, the service life and durability of concrete structures reinforced with FRP bars are higher, resulting in a decrease in the overall maintenance costs.

Previous experimental studies investigated the impact behaviour of RC beams reinforced with conventional steel reinforcement [5-21]. Three types of responses were observed: local response, global response, and a combination of local and global response. Local failure modes of steel RC beams under impact have been termed as being scabbing, which results in the spalling of the concrete cover, penetration, and diagonal shear cracking around the contact zone. Local response is typically referred as a shear “plug” type, even for flexural-critical steel RC beams [12] or a localized dynamic punching shear failure [8, 22-25], which occurs at higher velocity impacts. In the local response, the majority of energy from the impact in the steel RC beams is dissipated around the impact area. A global response of the steel RC beams represents bending and deformation responses of the beams. The behavior of steel RC beams under impact loading has been reported as the combination of local and global responses (bending and deformation) [7]. However, the global response has been reported as the main concern for the steel RC beams subjected to impact loading [7]. The influences of different parameters including impact velocity, impact energy, cracking response, and shear mechanisms were investigated and static and impact failure modes of steel RC beams were compared in the literature. Also, the previous studies were mostly limited to normal strength concrete (NSC) beams. Only a limited number of studies examined the impact response of high strength concrete (HSC) beams reinforced with conventional steel reinforcement [26-31]. A few studies reported that brittle shear failure occurs in HSC [32-36]. Although the behavior of steel RC beams under impact loading was studied extensively, limited attention has been devoted on the experimental investigation of the impact response of GFRP bar reinforcement concrete beams (GFRP-RC beams) [37]. Goldston et al. [37] reported that flexural-critical GFRP-RC beams displayed a shear “plug” type of failure under impact loading, which indicated the importance of shear mechanisms. It was also found that using high strength concrete and increasing the tensile reinforcement ratio, fewer inclined shear cracks occurred. However, from an extensive literature review, it was found that no study has so far addressed the impact behavior of ultra-high strength concrete (UHSC) (concrete compressive strength greater than 100 MPa) beams reinforced with GFRP bars [38]. It is noted that concrete compressive strength above 100 MPa has been considered as UHSC in Vincent and Ozbakkaloglu [39] and Ozbakkaloglu [40].

This paper investigates experimentally the static and impact responses of GFRP bar reinforced high strength concrete (HSC) and ultra-high strength concrete (UHSC) beams. Three different GFRP tensile longitudinal reinforcement ratios and two different grades of concrete were used. Under static loading, the influences of concrete strength and reinforcement ratio on load-carrying capacity, deflection, crack pattern and failure mode of the GFRP-RC beams were investigated. Under impact loading, the influences of impact energy on the dynamic midspan deflection, dynamic strain in the GFRP reinforcement bars, crack patterns, and failure modes of GFRP-REC beam were investigated.

2. Experimental Program

2.1 Material Properties

A local company supplied the ready-mix concrete used in this study. Concrete cylinders with 100 mm diameter and 200 mm height were cast to measure the concrete compressive strength according to Australian Standard AS 1012.9 [41]. Table 1 provides the details of concrete mix designs for concrete of nominal compressive strengths of 80 MPa and 120 MPa. The average compressive strengths of concrete at 28 days were 84.6 MPa (for the nominal concrete strength of 80 MPa) and 100.5 MPa (for the nominal concrete strength of 120 MPa), respectively. On the day of static testing (day 62), the average compressive strengths of concrete were 95 MPa (for the nominal concrete strength of 80 MPa) and 117 MPa (for the nominal concrete strength of 120 MPa). Three different diameters of sand-coated GFRP reinforcement bars were used. The #2S (Standard) bars had a nominal diameter of 6.35 mm, #3HM (High Modulus) had a nominal diameter of 9.53 mm, and #4HM had a nominal diameter of 12.7 mm. The tensile properties of the GFRP reinforcement bars were determined by testing three specimens from each type of GFRP reinforcement bar (#2S, #3HM and #4HM). Average tensile strength (f_u), modulus of elasticity (E_f) and rupture strain (ε_{fu}) using the Instron 8033 universal testing machine were obtained according to ASTM [42]. The GFRP reinforcement bars were loaded until failure at the rate of 1 mm/min. Strains in the bars were measured using a 100 mm extensometer attached to the GFRP bars within the free length. The stress-strain behavior of the GFRP bars was found to be linear. All tensile test specimens failed due to splitting and rupture of the GFRP fibers. For #2S GFRP reinforcement bars, $f_u = 732$ MPa, $\varepsilon_{fu} = 1.96\%$ and $E_f = 37.5$ GPa. For #3HM GFRP reinforcement bars, $f_u = 1764$ MPa, $\varepsilon_{fu} = 3.18\%$ and $E_f = 55.6$ GPa. For #4HM GFRP bars, $f_u = 1605$ MPa, $\varepsilon_{fu} = 3.30\%$ and $E_f = 48.6$ GPa.

Steel stirrups were used as shear reinforcement. Three 4 mm diameter steel reinforcement bar specimens were tested using the Instron 1343 universal testing machine, with a tensile capacity of 100 kN according to ASTM [43]. The tensile test specimens were loaded at 0.2 mm/min until necking. Mean yield strength, ultimate tensile strength and elastic modulus were measured as 583 MPa, 640 MPa and 200 GPa, respectively.

2.2 Details of GFRP-RC Beams

A total of twelve simply supported GFRP-RC beams were constructed and tested under static and impact loading. The experimental program consisted of two series of test specimens. The first series consisted of six GFRP-RC beams tested under static loading (S) (three-point bending) to investigate the influence of tensile GFRP reinforcement bars on the flexural behavior of beams. The test variables were the amount of tensile longitudinal reinforcement and the compressive strength of concrete. Three beams were constructed with concrete of 80 MPa nominal compressive strength, and three beams were constructed with concrete of 120 MPa nominal compressive strength. The parameters investigated were load-deflection behavior, failure mode, energy absorption and strain in the concrete and GFRP reinforcement bars. The second series consisted of six UHSC GFRP-RC beams tested under impact loading to investigate the dynamic response of UHSC GFRP-RC beams. The six GFRP-RC beams under impact loading (I) were constructed with the nominal concrete compressive strength of 120 MPa. Three beams had tensile longitudinal reinforcement ratios (ρ_f) of 1.0% and three other beams had ρ_f of 2.0%. The GFRP-RC beams were subjected to three different impact heights of the 580 kg drop hammer for specimens with $\rho_f = 1.0\%$ and $\rho_f = 2.0\%$. Based on the test results of the energy absorption capacity (50%, 75% and 100% energy absorption capacity) of the GFRP-RC beams under static loading, the height of the drop hammer was calculated. Three beams with $\rho_f = 1.0\%$ were subjected to drop hammer heights of 355 mm, 533 mm and 710 mm. The three beams with $\rho_f = 2.0\%$ were subjected to drop hammer heights of 550 mm, 825 mm and 1100 mm. Test parameters investigated included dynamic midspan deflection, dynamic bending resistance, dynamic strain in GFRP reinforcement bars, failure mode and crack patterns. The GFRP-RC beams were 2400 mm long, 100 mm wide and 150 mm deep. The GFRP-RC beams were reinforced with two GFRP bars in the tensile and two GFRP bars in the compressive region. The concrete clear cover was 15 mm (from the outer surface of the steel stirrup to the tensile face of the GFRP-RC beams). The effective depths (d) were calculated as 127.8 mm, 126.2 mm and 124.7 mm for beams with $\rho_f = 0.5\%$, 1.0% and 2.0%, respectively. The

4 mm diameter steel reinforcement used as shear reinforcement was spaced evenly at 50 mm centers. The reinforcing cages are shown in Fig. 1. A side view of the GFRP-RC beams is shown in Fig. 2.

The GFRP-RC beams were designed in accordance with ACI [4] to fail by both concrete crushing (over-reinforced), where the maximum usable compressive strain in the concrete (ϵ_{cu}) is assumed as 0.003 and GFRP reinforcement rupture (under-reinforced). Design nominal moment capacities (M_n) were calculated according to ACI [4] for the over and under-reinforced GFRP-RC beams. For GFRP-RC beams, the preferred design is over-reinforced, as the beam is assumed to be less brittle with an amount of pseudo-ductility. Under-reinforced GFRP-RC beams fail in a catastrophic way without warning. Two GFRP-RC beams were under-reinforced and ten GFRP-RC beams were over-reinforced according to ACI [4].

The GFRP-RC beams were labeled according to the series, nominal concrete strength, longitudinal reinforcement type, reinforcement ratio and type of loading. The arrangement is in the form of A–B–C–D_x, where A is the nominal concrete compressive strength (80 MPa or 120 MPa), B is the GFRP reinforcement bar type (#2S, #3HM or #4HM), C is the tensile GFRP longitudinal reinforcement ratio ($\rho_f = 0.5\%$, 1.0% and 2.0%) and D is for the type of loading, (S for static loading and I for impact loading). For GFRP-RC beams under impact loading, the subscript (x) represents the height of the drop hammer in meters. For example, GFRP-RC beam 80-#3HM-1.0-S was designed with the concrete compressive strength of 80 MPa, #3HM GFRP reinforcement bars, $\rho_f = 1.0\%$ and tested under static loading. For GFRP-RC beam 120-#4HM-2.0-I_{1.1}, the nominal concrete compressive strength was 120 MPa, #4HM GFRP reinforcement bars, $\rho_f = 2.0\%$ and was subjected to a 1.1 m drop hammer height under impact loading. Table 2 provides a summary of the properties of the GFRP-RC beams including design nominal moment capacity, M_n according to ACI [4], calculated using preliminary material properties obtained from experimental testing.

3. Experimental Setup

3.1 Static Testing

Test set-up for the GFRP-RC beams under static loading involved placing the beams between two steel I-beams with a clear span of 2000 mm. The beam had a 200 mm overhang on each side. The beams were simply supported: a pin support at one end and a roller support at the other end. A 600 kN hydraulic actuator anchored to a steel frame was used to apply monotonic increasing loads on a steel circular plate positioned at the midspan. The hydraulic actuator had a built-in transducer which captured the midspan deflection. The GFRP-RC beams were tested under the

displacement controlled loading at a rate of 1 mm/min until failure. At the top on each side of the GFRP-RC beams, directly underneath the position of the load cell, two strain gauges were attached to measure concrete strain. Also, one strain gauge was attached to each of the tensile GFRP reinforcement bars at the center to measure the average tensile strain. All data including load, midspan deflection and strain were recorded with a high-speed data acquisition system (NI PXIe-1078). Fig. 3 shows the test setup of the GFRP-RC beams under static loading.

3.2 Impact Testing

Six GFRP-RC beams were subjected to a 580 kg high capacity free falling drop hammer as shown in Fig. 4. The test setup involved fixing two steel blocks to the floor so that the GFRP-RC beams had a clear span of 2000 mm with a 200 mm overhang on each side. All impact GFRP-RC beams were simply supported and positioned on a steel pin and a steel roller. To prevent rebound during impact, steel frame rollers were connected to the steel blocks. The drop hammer was lifted mechanically to the required drop height using an automotive control system. The drop hammer was released using an electronic quick release system. The dynamic midspan deflections were determined by image processing technique using high-speed video camera recordings by positioning a leveler next to the midspan of the beams. Black and white dots were marked onto the beams in order to accurately analyze the deflections. The recording rate of the high-speed video camera was 1000 frames/sec. The dynamic concrete strain was not measured due to the extensive damage in the impact area caused by the drop hammer. However, the dynamic tensile strain was measured from the strain gauges located in the middle of the GFRP tensile reinforcement bars. The recording rate of the high-speed camera was 1000 frames per second. The high-speed data acquisition system, NI-PXI-1050, was used to record all the data, including impact force (load cell connected to the underside of the drop hammer) and dynamic tensile strain, with a frequency of 100,000 samples per second.

4. Experimental Results and Discussions

4.1 Response under Static Loading

4.1.1 Failure Modes

The GFRP-RC beams were designed to have two distinct failure modes under static loading: GFRP reinforcement rupture (for beams with $\rho_f = 0.5\%$) and concrete crushing (for beams with $\rho_f = 1.0\%$ and 2.0%). During testing,

the beams designed as under-reinforced (GFRP-RC beams 80-#2S-0.5-S and 120-#2S-0.5-S) showed vertical flexural cracking, which initially formed around the midspan. Flexural cracks started to form at around 3 kN. New vertical cracks started to propagate closer to the supports at higher loading levels. Already formed cracks around the midspan continued to propagate vertically. The GFRP-RC beams failed because of the rupture of GFRP reinforcement bars (Fig. 5). This occurred unexpectedly with no sign of warning. Concrete strain at the time of failure was measured as 0.002 and 0.0017 for GFRP-RC beams 80-#2S-0.5-S and 120-#2S-0.5-S, respectively (Fig. 6). Rupture strain of GFRP reinforcement bars was not recorded, since the strain gauges failed prior to failure of the GFRP-RC beams. At the time of failure, the experimental load-carrying capacities were measured as 15 kN and 16.2 kN for beams 80-#2S-0.5-S and 120-#2S-0.5-S, respectively. Midspan deflections, Δ_{exp} , were recorded as 81.8 mm and 77.5 mm for beams 80-#2S-0.5-S and 120-#2S-0.5-S, respectively.

For the over-reinforced GFRP-RC beams (beams 80-#3HM-1.0-S, 80-#4HM-2.0-S, 120-#3HM-1.0-S, and 120-#4HM-2.0-S), two distinct failure modes were observed. Initially, the crushing of concrete cover occurred. This occurred at compressive strains between 0.003 and 0.004 (Fig. 6), which is considered “failure” from a design point of view. Thus, at these recorded concrete strains, experimental load carrying capacity (P_u) was determined. Also, at this point, strains in the GFRP reinforcement bars (ranging from 1.3% to 1.9%) were lower than the rupture strain, indicating a concrete crushing failure (Fig 6). However, the GFRP-RC beams showed signs of continually sustaining the load, which indicated signs of reserve capacity or an amount of pseudo “ductility”. At higher loading levels, concrete cover continued to crush before the total failure. At the total failure, the GFRP-RC beams failed by the rupture of the GFRP reinforcement bars and were unable to carry additional loads (Fig. 7).

4.1.2 Load-Midspan Deflection Response

The load-midspan deflection response of the GFRP-RC beams under static loading is shown in Fig. 8. All GFRP-RC beams displayed a bi-linear response. Initially, before cracking, the bending stiffness of the beams was high. The bending stiffness reduced once the cracking occurred, especially for the GFRP-RC beams with the lowest amount of reinforcement. This was attributed to the low elastic modulus of the #2S bars. From the preliminary test, the modulus of elasticity was calculated as 37.5 GPa for the #2S GFRP reinforcement bars. For higher amounts of reinforcement, the bending stiffness reduced, but not as drastic as for the GFRP-RC beams with $\rho_f = 0.5\%$. Energy absorption capacities (E_1 and E_2) were calculated as the area under the load-midspan deflection curves [44, 45]. For the over-

reinforced GFRP-RC beams, at the first major drop in load carrying capacity, which was considered “failure” (at E_1) and thus the reserve capacity or “ductility” was calculated after this loading point, that is E_2 . A similar approach was adopted in Goldston et al. [37] and Goldston et al. [38] to calculate the energy absorption capacity of the beam. The GFRP-RC beams 80-#2S-0.5-S and 120-#2S-0.5-S had no reserve capacity as they collapsed because of the rupture of GFRP reinforcement bars. Total energy absorption capacities for the GFRP-RC beams ranged from 714 J to 6377 J. Table 3 reports the results for the GFRP-RC beams tested under static loading.

4.1.3 Influence of Concrete Strength and Tensile Reinforcement

The influence of concrete compressive strength and amount of tensile reinforcement were systematically investigated to understand their influences on the behavior of GFRP-RC beams under static loading, in terms of load carrying capacity, midspan deflection, and post-cracking bending stiffness. For the GFRP-RC beams with $\rho_f = 0.5\%$, the effect of concrete compressive strength had minimal influence on the load carrying capacity. For the increase in the concrete compressive strength from 95 MPa to 117 MPa, the load increased by 8% (15 kN to 16.2 kN). This is because the GFRP-RC beams were designed as under-reinforced beams and hence their failure was governed by the strength of the GFRP reinforcement bars under tension. Midspan deflection was shown to decrease by 5% (81.8 mm to 77.5 mm) for $\rho_f = 0.5\%$ for an increase in the concrete compressive strength (95 MPa to 177 MPa). A 12% increase in post-cracking bending stiffness was observed for $\rho_f = 0.5\%$ for an increase in the concrete compressive strength from 95 MPa to 117 MPa.

Concrete compressive strength was more influential for the GFRP-RC beams with tensile longitudinal reinforcement ratios of $\rho_f = 1.0\%$ and $\rho_f = 2.0\%$ in increasing the load carrying capacity, as the failure was governed by the compressive strength of concrete (crushing of concrete cover). For $\rho_f = 1.0\%$ and $\rho_f = 2.0\%$, the load increased by 27% (33 kN to 41.8 kN) and 13% (46.1 kN to 52.2 kN), respectively for the increase in the concrete compressive strength from 95 MPa to 117 MPa. However, increasing concrete compressive strength increased the midspan deflection by 17% and 10% for $\rho_f = 1.0\%$ and $\rho_f = 2.0\%$, respectively. In terms of post-cracking bending stiffness, for $\rho_f = 1.0\%$, the stiffness increased by 10% for an increase in concrete compressive strength. However, for $\rho_f = 2.0\%$, a reduction of 0.07% in post-cracking bending stiffness was observed. At higher reinforcement ratios, higher concrete compressive strength (UHSC) did not improve the post-cracking bending stiffness. The effect of concrete

compressive strength on load carrying capacity, midspan deflection and post-cracking bending stiffness is shown in Fig. 9, Fig. 10 and Fig. 11, respectively.

In terms of reinforcement ratio, the increase in the amount of tensile longitudinal GFRP reinforcement increased the load-carrying capacity, reduced deflection and increased post-cracking bending stiffness, regardless of concrete compressive strength as shown in Fig. 9, Fig. 10 and Fig. 11. For the GFRP-RC beams with concrete compressive strength of 95 MPa, the load carrying capacity increased by 120%, with a decrease in the midspan deflection of 23% and an increase in post-cracking bending stiffness by 231% for the increase in the ρ_f from 0.5% to 1.0%. This significantly large change in the post-cracking bending stiffness is due to the change in failure mode (from the rupture of the GFRP reinforcement to the crushing of concrete). However, for the increase in the ρ_f from 1.0% to 2.0%, the load carrying capacity increased by 40%, with 7% reduction in the deflection and 61% increase in the post-cracking bending stiffness.

Similar results were observed for the UHSC (117 MPa) GFRP-RC beams. For the increase in the reinforcement ratio ρ_f from 0.5% to 1.0% and from 1.0% to 2.0%, the load-carrying capacity increased by 158% and 25%, respectively. A decrease of 5% in the midspan deflection was observed for a change in the ρ_f from 0.5% to 1.0%, compared to 12% for the increase in the ρ_f from 1.0% to 2.0%. In terms of post-cracking bending stiffness, an increase of 224%, and 47% was observed for an increase in the ρ_f from 0.5% to 1.0% and from 1.0% to 2.0, respectively.

4.1.4 Experimental versus FRP Code Recommendations

The FRP design recommendation [4] for the calculation of nominal load carrying, P_n , was compared using experimental results for load carrying capacity (P_u) for the GFRP-RC beams under static loading. Based on the preliminary material testing results, nominal bending moment and load carrying capacities were calculated. In general, the ACI [4] provided relatively conservative results compared to the experimental results, with a mean $P_n/P_u = 0.73$ (Table 3). That is, the ACI [4] under-predicted load by an average of 37%. Regardless of the failure mode (concrete crushing or GFRP reinforcement rupture), the experimental load carrying capacity was found to be higher to that of the calculated nominal load carrying capacity for all GFRP-RC beams.

The most conservative results were achieved at the highest tensile longitudinal GFRP reinforcement ratio ($\rho_f = 2.0\%$). For $\rho_f = 2.0\%$, an average of $P_n/P_u = 0.70$ (under-prediction by 43%) was calculated for ACI [4]. For $\rho_f = 0.5\%$, ACI [4] under-predicted deflection by an average of 37%. The least conservative results were observed for the

GFRP-RC beams with $\rho_f = 1.0\%$. For the GFRP-RC beams with $\rho_f = 1.0\%$, the midspan deflection was under-predicted by 28%. In terms of concrete strength, for the higher the concrete strength (117 MPa), more conservative the nominal load carrying capacity was calculated by ACI [4] compared to experimental load carrying capacity. According to ACI [4], for the concrete compressive strength of 117 MPa, a mean $P_n/P_u = 0.71$ was calculated (under-prediction of 41%), compared to $P_n/P_u = 0.76$ for concrete compressive strength of 95 MPa (under-prediction of 32%). Table 3 reports the comparison between experimental load carrying capacity and nominal load capacity according to ACI [4].

4.2 Response under Impact Loading

4.2.1 Failure Modes

Three GFRP-RC beams with the tensile longitudinal GFRP reinforcement ratios of $\rho_f = 1.0\%$ and $\rho_f = 2.0\%$ were subjected to various drop hammer heights. The quasi-static energy absorption capacity of the beam was used as the input impact energy in Hughes and Al-Dafiry [46]. In this study, the quasi-static energy absorption capacity of the beams was used to select the initial drop height. The quasi-static energy absorption capacities of the GFRP-RC beam 120-#3HM-1.0-S, at 50%, 75% and, 100% were 2029 J, 3043 J and 4057 J, respectively. Hence, the three drop heights were calculated as 355 mm, 533 mm and 710 mm, respectively. For GFRP-RC beam 120-#5HM-2.0-S, at 50%, 75%, and 100% energy absorption capacity, the calculated static energy absorption capacities were 3189 J, 4783 J, and 6377 J, respectively. Hence, the drop heights were calculated as 550 mm, 825 mm, and 1100 mm, respectively. Overall, the experimental failure mode and general behavior including crack patterns were relatively similar for all six GFRP-RC beams subjected to various drop heights. The experimental failure mode was found to shift under impact loading as a result of the use of UHSC. This resulted in localized concrete crushing on the top surface with flexural cracks observed around the impact region, with flexural-shear cracks occurring closer towards the support regions. This can be seen in Fig. 12 which shows the point of impact between the drop hammer and the GFRP-RC beam, displaying flexural-shear cracks. This was expected as the impact area is subjected to high shear forces and large bending moments.

The GFRP-RC beam 120-#3HM-1.0-I_{0.355} experienced minor crushing of the concrete cover on the top surface at the impact point as shown in Fig. 13(a). During impact, cracks were predominately observed as a combination of flexure, flexure-shear and minor shear cracks propagating from the tensile region throughout the height of the GFRP-RC beam. The majority of these cracks were observed to be localized around the impact zone. A few flexure-shear cracks were

observed closer towards the supports. When the beam was subjected to impact energy of 2029 J, there was no permanent deflection (residual deflection). The GFRP-RC beam 120-#3HM-1.0-I_{0.533} showed signs of additional crushing of concrete cover, with the exposure of the compressive GFRP reinforcement bars (Fig. 13(b)). The crushing of concrete cover was not symmetric under the impact area, with the localized crushing of concrete cover to one side of the impact point. Under a drop hammer height of 533 mm, a small amount of rupture of the tensile concrete cover occurred and the GFRP tensile reinforcement bars were exposed around the impact zone, which significantly widened a few cracks around the midspan. The cracks were mostly the flexure cracks throughout the span of the GFRP-RC beam. A few flexure-shear cracks and a few minor inclined shear cracks were also observed. The GFRP-RC beam 120-#3HM-1.0-I_{0.710} showed extremely localized concrete cover crushing and rupture of the tensile concrete cover, causing the concrete to spall off as shown in Fig. 13 (c). The spalling off of the concrete was symmetrical under the impact point, which exposed the compressive and tensile GFRP reinforcement bars. Also, a predominant flexural crack pattern around the impact zone was observed. Only a few signs of flexure-shear cracks and minor inclined shear cracking were observed. This GFRP-RC beam showed the least number of cracks during impact. By close inspection, some signs of the splitting of fibers from GFRP tensile reinforcement bars were observed.

The GFRP-RC beam 120-#4HM-2.0-I_{0.550} experienced minor concrete crushing in the impact zone and cracks along the span as shown in Fig. 13(d). These cracks were predominately vertical with the presence of a few flexure-shear cracks and a minor inclined shear cracks. After the impact, the GFRP-RC beam remained elastic after the removal of the drop hammer mass. GFRP-RC beam 120-#4HM-2.0-I_{0.825} was subjected to impact energy of 4783 J, from a drop height of 825 mm. A large amount of tensile concrete cover spalled off during impact, causing cracks to widen around the midspan as shown in Fig. 13(e). A few more signs of inclined shear cracking were present, especially closer to the support regions. But the majority of cracks were predominately flexure-shear with the presence of flexural cracks. Finally, the impact energy caused permanent deformation (residual deflection) of GFRP-RC beam 120-#4HM-2.0-I_{0.825} with crushing of concrete on the top surface.

The GFRP-RC beam 120-#4HM-2.0-I_{1.1} was subjected to an impact height of 1.1 m. The general behavior of GFRP-RC beam 120-#4HM-2.0-I_{1.1} was similar to the behavior of GFRP-RC beam 120-#4HM-2.0-I_{0.825}, but additional post impact permanent deformation was noticed as shown in Fig. 13(f). In terms of impact zone damage, concrete crushing on the top surface was localized on one side of the impact point. Rupture of the tensile concrete also occurred only on the same side of the impact point where concrete crushing occurred. Also, by close inspection, the impact caused the

de-bonding of the sand-coat of the GFRP tensile reinforcement bars around the midspan. In terms of cracking, very few cracks were formed compared to GFRP-RC beams 120-#4HM-2.0-I_{0.550} and 120-#4HM-2.0-I_{0.825}. A combination of flexure and flexure-shear cracks were observed and spaced evenly along the span of GFRP-RC beam 120-#4HM-2.0-I_{1.1}. Moreover, no sign of rupture or splitting of fibers was detected.

4.2.2 Dynamic Midspan Deflection Response

Dynamic midspan deflection time history responses for the GFRP-RC beams under impact loading are shown in Fig. 14. These graphs were drawn by image processing from the high-speed camera. Fig. 14 was modified to initiate the first contact point between the drop hammer and the GFRP-RC beams (i.e., at the coordinates of 0, 0). For GFRP-RC beam 120-#3HM-1.0-I_{0.710}, a black dot was used to track midspan deflections frame by frame. However, during impact, the crushing of concrete cover caused the black dot to disappear after a period of time. Thus, maximum dynamic midspan deflection (Δ_{exp}) was difficult to be captured and was illustrated by the irregular dynamic midspan deflection time history response. 170 mm for maximum dynamic midspan deflection was assumed. The remaining GFRP-RC beams had a white dot painted on, which increased the visibility and therefore increased the accuracy for determining the maximum dynamic midspan deflection. A parabolic curve was attained for dynamic midspan deflection versus time, with the first portion of the graph (positive dynamic midspan deflection rate) representing the contact between the drop hammer and the GFRP-RC beam up until maximum dynamic-midspan deflection. At post dynamic midspan deflection, the GFRP-RC beams began to rebound and move in the opposite direction (negative dynamic midspan deflection rate) since the impact energy wasn't sufficient to cause total failure. It is noted that GFRP-RC beam 120-#4HM-2.0-I_{1.1} did not rebound as the impact energy caused total collapse. For the other two GFRP-RC beams with $\rho_f = 1.0\%$, maximum dynamic midspan deflections were calculated as 93.4 mm and 75 mm for GFRP-RC beams 120-#3HM-1.0-I_{0.533} and 120-#3HM-1.0-I_{0.355}, respectively. For the GFRP-RC beams with $\rho_f = 2.0\%$, dynamic midspan deflection was calculated as 70.5 mm, 129.5 mm and 249.5 mm for GFRP-RC beams 120-#4HM-2.0-I_{0.550}, 120-#4HM-2.0-I_{0.825} and 120-#4HM-2.0-I_{1.1}, respectively.

4.2.3 Dynamic Load-Time History Response

The dynamic load-time history response of the GFRP-RC beams under impact loading in a 120-millisecond window is shown in Fig. 15. Initially, a short high magnitude duration pulse (between 217 kN to 591 kN for all six GFRP-RC

beams) occurred at the first contact between the GFRP-RC beams and the drop hammer. This is indicative that the dynamic force was initially resisted by the inertia forces at the first contact point. After this short time duration, the dynamic force was then resisted by the GFRP-RC beams flexural resistance for four of the six GFRP-RC beams. Thus, dynamic bending resistance was extracted from the dynamic load-time history response. For GFRP-RC beams 120-#3HM-1.0-I_{0.355}, 120-#3HM-1.0-I_{0.533}, 120-#4HM-2.0-I_{0.550} and 120-#4HM-2.0-I_{0.825}, dynamic bending resistance was 49.7 kN, 54.4 kN, 66.5 kN and 78.6 kN, respectively. These four GFRP-RC beams displayed well-defined dynamic-load time history responses unlike GFRP-RC beams 120-#3HM-1.0-I_{0.710} and 120-#4HM-2.0-I_{1.1}, where the dynamic bending resistance could not be established. The reason for the differences was because for these two GFRP-RC beams, impact energy from the drop hammer caused total collapse, without rebounding, and thus the data was distorted. Average dynamic strain ($\epsilon_{frp.avg}$) is shown in Fig. 16. Initially, prior to the formation of cracking, dynamic strain rate was relatively high. At the start of cracking, a small drop in dynamic strain was observed. Post-cracking, dynamic strain rate reduced as a result of the formation of cracks and low elastic modulus of the GFRP reinforcement bars. The post-cracking strain increased fairly linearly up until average maximum dynamic strain (at approximately $t = 0.05$ s and $t = 0.04$ s, for GFRP-RC beams with $\rho_f = 1.0\%$ and $\rho_f = 2.0\%$, respectively). For GFRP-RC beam 120-#4HM-2.0-I_{1.1}, the strain gauges failed prior to the recording of maximum dynamic strain. Thus, linear regression analysis was carried out by increasing the post-cracking dynamic strain up to approximately $t = 0.04$ s, as at this time maximum dynamic strain occurred for the other two GFRP-RC beams with $\rho_f = 2.0\%$. The regression analysis gave approximately 3.0% dynamic strain. For the remaining five GFRP-RC beams, average dynamic strain decreased after maximum dynamic strain decreased due to the rebound effect.

4.2.4 Influence of Impact Energy

The effect of increasing impact energy on dynamic midspan deflection of the GFRP-RC beams under impact loading is shown in Figure 17. For $\rho_f = 1.0\%$, impact energies of 2029 J, 3043 J and 4057 J were applied to the three GFRP-RC beams. For the GFRP-RC beams with $\rho_f = 2.0\%$, impact energies of 3189 J, 4783 J and 6377 J were applied to the three GFRP-RC beams. For both $\rho_f = 1.0\%$ and $\rho_f = 2.0\%$, increasing impact energy increased dynamic midspan deflection. For $\rho_f = 1.0\%$, increasing impact energy by 50% (2029 J to 3043 J), 100% (2029 J to 4057 J) and 33% (3043 J to 4057 J), dynamic midspan deflection increased by 25%, 126% and 82%, respectively. A significant increase in the dynamic midspan deflection was observed for higher levels of impact energy. This was also evident

for the GFRP-RC beams with $\rho_f = 2.0\%$. For an increase in impact energy of 50% (3189 J to 4783 J), 100% (3189 J to 6377 J) and 33% (4783 J to 6377 J), the dynamic midspan deflection increased by 84%, 254% and 93%, respectively.

The effect of increasing impact energy on the maximum dynamic strain of the GFRP-RC beams under impact loading is shown in Fig. 18. For $\rho_f = 1.0\%$, increasing impact energy showed to have a linearly increasing effect on the maximum dynamic strain. Increasing the impact energy by 50%, 100% and 33%, the maximum dynamic strain increased by 15%, 30% and 13%, respectively. At 100% impact energy, the maximum dynamic strain recorded was 2.6%, which was 22% lower than the mean rupture strain (3.18%) obtained from the preliminary testing. However, as noted previously, the rupture of the GFRP reinforcement bars was not evident after impact, only small signs of the splitting of fibers was evident. This illustrates that the GFRP-RC beam could sustain higher levels of impact before total rupture of the GFRP reinforcement bars. For $\rho_f = 2.0\%$, increasing impact energy by 50%, 100% and 33% increased the maximum dynamic strain considerably more, as compared to $\rho_f = 1.0\%$, by 18%, 76% and 50%, respectively. At 100% impact energy, the maximum dynamic strain was approximated to be 3% through regression analysis, which was 10% lower than from preliminary material testing (3.30%). This is illustrated by no signs of splitting or rupture of GFRP reinforcement fibers, since the GFRP reinforcement bars did not reach rupture strain.

4.2.5 Comparative Analysis of Failure Modes under Static and Impact Loading

Experimental investigations have shown that failure modes under static and impact loading are quite distinct. The failure mode of the GFRP-RC beams matches with the observation provided by Saatci and Vecchio [12]. Saatci and Vecchio [12] showed that a flexure-critical RC beam subjected to impact loading would experience shear cracking forming a shear “plug” around the impact zone. Comparing the differences in deflections under static and impact loading would not provide any reasonable outcomes due to the significant differences in the overall behavior and failure mode. Thus, failure modes and behavior including crack patterns were compared in terms of midspan deflection under static and impact loading.

For GFRP-RC beams 120-#3HM-1.0-S and 120-#4HM-2.0-S, at an energy absorption capacity of 50%, midspan deflections were measured as 82 mm and 89 mm, respectively. At this midspan deflection, the overall behavior of the GFRP-RC beams displayed signs of crushing of concrete cover with predominately flexural cracks and a few flexural shear-cracks propagating from the tensile region throughout the span of the beam. This type of behavior was also

observed for the identical GFRP-RC beams under impact loading subjected to drop hammer heights of 355 mm (GFRP-RC beam 120-#3HM-1.0-I_{0.355}) and 550 mm (GFRP-RC beam 120-#4HM-2.0-I_{0.550}). The minor crushing of concrete cover with flexural and flexural-shear cracks forming from the tensile area was observed. At these drop hammer heights, midspan deflections were measured as 75 mm and 73 mm, for GFRP-RC beams 120-#3HM-1.0-I_{0.355} and 120-#4HM-2.0-I_{0.550}, respectively. At 50% energy absorption capacities, the failure modes of the GFRP-RC beams were similar regardless of static or impact loading, resulting in relatively similar midspan deflections.

At higher energy absorption capacities, it was observed that failure modes and crack propagation had similar and distinctive differences under static and impact loadings. For GFRP-RC beam 120-#3HM-1.0-S, at 75% energy absorption capacity, the overall failure was predominately flexural critical with flexural-shear cracks and crushing of concrete cover on both sides of the loading cell. Also, similar behavior was observed for GFRP-RC beam 120-#3HM-1.0-I_{0.533}, with the main differences being more localized concrete crushing around the impact zone and rupture of the tensile concrete cover which resulted in the exposure of the tensile GFRP reinforcement bars. Overall, behavior was noticed to be alike and due to the similarities in failure modes, measured deflections were similar. For GFRP-RC beam 120-#3HM-1.0-S, at 75% energy absorption capacity, midspan deflection was measured as 106 mm, compared to 93 mm under impact loading for GFRP-RC beam 120-#3HM-1.0-I_{0.533}. However, failure modes for GFRP-RC beam 120-#4HM-2.0-S, at 75% energy absorption capacity and GFRP-RC beam 120-#4HM-2.0-I_{0.825} were very different. GFRP-RC beam 120-#4HM-2.0-S at 75% energy absorption capacity exhibited a flexural failure with concrete crushing on the top surface. The GFRP-RC beam 120-#4HM-2.0-I_{0.825} experienced localized crushing of the concrete cover, exposing the compressive GFRP reinforcement bars. Furthermore, rupture of tensile concrete cover occurred, causing cracks to widen, with the addition of minor inclined shear cracking around the impact zone. None of this behavior was observed for GFRP-RC beam 120-#4HM-2.0-S, at 75% energy absorption capacity, except for the concrete crushing of the cover. As a result of the differences in failure modes, the failure mode developed by GFRP-RC beam 120-#4HM-2.0-I_{0.825} displayed a higher midspan deflection, 139 mm, compared to 116 mm for GFRP-RC beam 120-#4HM-2.0-S at 75% energy absorption capacity.

At 100% impact energy (GFRP-RC beam 120-#3HM-1.0-I_{0.710} and GFRP-RC beam 120-#4HM-2.0-I_{1.1}), failure was described as a flexural failure. Localized damage around the impact zone, with severe rupture of the tensile concrete cover and crushing of compressive concrete cover, was observed. Very few cracks developed along the span of the beam, with these cracks predominately inclined shear cracks around the impact zone. Permanent deformation was also

evident after the removal of the drop hammer. This caused the GFRP-RC beams to have dynamic midspan deflections of 175 mm for GFRP-RC beam 120-#3HM-1.0-I_{0.710} and 250 mm for GFRP-RC beam 120-#4HM-2.0-I_{1.1}. Under static loading, at 100% energy absorption capacity, flexural cracks and flexural-shear cracks were evident along the span of the GFRP-RC beams, with the crushing of compressive concrete cover and rupture of tensile concrete cover. This type of failure mode resulted in midspan deflections of 128 mm and 140 mm for GFRP-RC beams 120-#3HM-1.0-S and 120-#4HM-2.0-S, respectively. Fig. 19 compares the crack pattern for the GFRP-RC beam with $\rho_f = 1.0\%$ at different energy absorption capacities under static loading and impact loading. Fig. 20 compares the crack pattern for the GFRP-RC beam with $\rho_f = 2.0\%$ at different energy absorption capacities under static and impact loading.

4.2.6 Dynamic Amplification Factor (DAF)

A Dynamic Amplification Factor (DAF) was obtained for the GFRP-RC beams. The DAF is defined as the ratio of the experimental dynamic moment capacity (M_d) to the experimental static moment capacity (M_{static}). Dynamic moment capacity was calculated using Equation (1). The $R_1(t)$ was assumed as half the impact force, that is $R_1(t) = I(t)/2$. Thus simplifying Equation (1), $M_d = I(t)/2$, where $L = 2000\text{mm}$. Static moment capacities (M_{static}) were calculated based on energy absorption capacity. That is for GFRP-RC beam 120-#3HM-1.0-S, at 50%, 75%, and 100% energy absorption capacity, static moment capacities were calculated as 21 kN.m, 23 kN.m and 23 kN.m, respectively. For GFRP-RC beam 120-#4HM-2.0-S, at 50%, 75%, and 100% energy absorption capacity, static moment capacities were measured as 30 kN.m, 32 kN.m and 35 kN.m, respectively. Table 4 reports the dynamic and static moment capacities of the GFRP-RC beams. Overall, an average DAF was calculated as 1.17. An average of 17% higher capacities under dynamic loading was obtained, indicating higher reserve capacity for the GFRP-RC beams under impact loading as compared to static testing. However, a DAF could not be obtained for GFRP-RC beams 120-#3HM-1.0-I_{0.710} and 120-#4HM-2.0-I_{1.1} since they totally collapsed, and thus dynamic moment capacity was not calculated. A time history of dynamic moment capacity for GFRP-RC beam 120-#4HM-2.0-I_{0.825} is shown in Fig. 21.

$$M_d(t) = \frac{2L}{6}R_1(t) + \frac{I(t)}{2} \times \frac{L}{6} \quad (1)$$

4.2.7 Verification of the failure modes

The design code ACI [4] provides equations to calculate the nominal shear strength of a GFRP-RC beams. The nominal shear strength of a GFRP-RC beams is the sum of the contribution of concrete and the contribution of the steel stirrups to the shear strength. The nominal shear strength values of the beams tested under impact loading were presented in Table 5. To calculate the maximum experimental shear forces, the equilibrium of dynamic forces was used [12]. According to Saatci and Vecchio [12], during the first few milliseconds of impact, the inertia load and impact load of the RC beam coincide. Since the accelerations were not measured in the experiments, the inertia load was assumed to be equal to the impact load during the first 3 milliseconds of impact, when the maximum impact load takes place. The maximum experimental shear force was then calculated and presented in Table 5. It can be observed that when the nominal shear strength of the beam was larger than the experimental maximum shear force, the failure of the beam was flexural failure. However, when the experimental shear force was very close to the shear capacity of the beam, the failure was flexural-shear failure. Therefore, the high shear capacity of the GFRP-RC beams with UHSC prevented the dominant shear failure in the beams.

5. Conclusions

An experimental program consisting of twelve simply supported GFRP bar reinforced concrete beams (GFRP-RC beams) subjected to static and impact loadings has been carried out. The behavior of GFRP-RC beams with varying reinforcement ratio and concrete strengths (HSC and UHSC) have been investigated. The following conclusions have been drawn based on the observations from the experimental results.

The failure mode of GFRP-RC beams under static loading (three-point bending) can be determined using sectional analysis used for beams reinforced with steel reinforcement bar. For the GFRP-RC beams with more than balanced reinforcement (over-reinforced), failure was caused by the crushing of concrete cover. For the GFRP-RC beams with less than balanced reinforcement ratio (under-reinforced), failure was observed to be caused by GFRP reinforcement rupture.

Load-midspan deflection behavior of GFRP-RC beams under static loading (three-point bending) showed a bi-linear response. The first part of the bi-linear response represented an uncracked section and the second part represented a crack section with a reduction in the bending stiffness. The over-reinforced GFRP-RC beams displayed signs of pseudo “ductility”, where the beams were able to resist load before total collapse. The under-reinforced GFRP-RC beams which failed suddenly by rupture of GFRP reinforcement, resulting in no reserve capacity. The design

recommendation for concrete beams reinforced with FRP bars [24] was found to be very conservative, under-predicting the load carrying capacity by an average of 37% for the GFRP-RC beams under static loading.

The effect of HSC and UHSC was found to influence the overall behavior of GFRP-RC beams under static loading (three-point bending) in terms of load carrying capacity, deflection, and post-cracking bending stiffness. For the GFRP-RC beams with the tensile longitudinal reinforcement ratio of 0.5% ($\rho_f = 0.5\%$), the increase in the concrete strength from 95 MPa to 117 MPa, the load carrying capacity increased by 8% (15 kN to 16.2 kN). The small increase in load carrying capacity is because these GFRP-RC beams are designed as under-reinforced and hence the failure is governed by the tensile strength of the GFRP bars. For GFRP-RC beams with $\rho_f = 1.0\%$ and $\rho_f = 2.0\%$, load carrying capacity increased by 27% (33 kN to 41.8 kN) and 13% (46.1 kN to 52.2 kN), respectively, for the increase in the concrete compressive strength from 95 MPa to 117 MPa. However, increasing concrete strength increased midspan deflection for the GFRP-RC beams with $\rho_f = 1.0\%$ and $\rho_f = 2.0\%$ by 17% and 10%, respectively. In terms of post-cracking bending stiffness, for the GFRP-RC beams with $\rho_f = 1.0\%$, stiffness increased 10% for a change in concrete strength from HSC (95 MPa) to UHSC (117 MPa). At higher reinforcement ratios ($\rho_f = 2.0\%$), concrete strength (HSC and UHSC) did not improve the post-cracking bending stiffness.

Under impact loading, the UHSC GFRP-RC beams displayed a change in failure, from shear to a flexural failure. This was a result of using Ultra High Strength Concrete (UHSC) as opposed to Normal Strength Concrete (NSC) or HSC. Flexural cracking around the impact region with the crushing of concrete cover was observed. Flexural-shear cracks were observed closer to the supports. However, the GFRP-RC beams under static loading failed in a flexural response. Thus, the shear behavior of flexure-critical GFRP-RC beams must be considered when designing structures subjected to impact loads.

The increase in impact energy increased the dynamic midspan deflection of the GFRP-RC beams. At lower levels of impact energy, for the same amount of reinforcement dynamic deflections were found to be similar. However, at very large levels of impact energy, a significant increase in the dynamic deflection was observed. Also, by increasing impact energy by 50%, 33% and 100%, the dynamic strain in the GFRP reinforcement bars increased approximately linearly, especially for a reinforcement ratio of $\rho_f = 1.0\%$. The average dynamic amplification factor was 1.17.

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Tables

Table 1. Concrete Mix Designs of NSC and UHSC

Material	Nominal Concrete Strength	
	80 MPa	120 MPa
Bastion General Purpose Cement	540 kg/m ³	600 kg/m ³
Fine Grade Fly Ash	40 kg/m ³	N/A
Micro Silica Densified Silica Fume	40 kg/m ³	40 kg/m ³
10 mm Aggregate	1040 kg/m ³	1020 kg/m ³
Coarse Sand	420 kg/m ³	450 kg/m ³
Fine Sand	100 kg/m ³	150 kg/m ³
Sika Viscocrete PC HRF2 (Superplasticiser)	4 L/m ³	5 L/m ³
Water	160 L/m ³	155 L/m ³

Table 2. Details of GFRP-RC Beams

GFRP-RC Beam	d (mm)	ρ_f (%)	f'_c (MPa)	ρ_s (%)	Reinforcement Condition	M_n (kN.m)
						(ACI [4])
80-#2S-0.5-S	127.8	0.5	80	0.50	Under	5.7
80-#3HM-1.0-S	126.2	1.0	80	0.50	Over	13.6
80-#4HM-2.0-S	124.7	2.0	80	0.50	Over	16.0
120-#2S-0.5-S	127.8	0.5	120	0.50	Under	5.7
120-#3HM-1.0-S	126.2	1.0	120	0.50	Over	15.2
120-#4HM-2.0-S	124.7	2.0	120	0.50	Over	18.0
120-#3HM-1.0-I _{0.71}	126.2	1.0	120	0.50	Over	15.3
120-#3HM-1.0-I _{0.533}	126.2	1.0	120	0.50	Over	15.3
120-#3HM-1.0-I _{0.355}	126.2	1.0	120	0.50	Over	15.3
120-#4HM-2.0-I _{1.1}	124.7	2.0	120	0.50	Over	18.1
120-#4HM-2.0-I _{0.825}	124.7	2.0	120	0.50	Over	18.1
120-#4HM-2.0-I _{0.550}	124.7	2.0	120	0.50	Over	18.1

Note: d is effective depth, ρ_f is tensile reinforcement ratio, f'_c is nominal concrete compressive strength, ρ_s is shear reinforcement ratio and M_n is nominal moment capacity calculated according to ACI (2015).

Table 3. GFRP-RC Beams under Static Loading

GFRP-RC Beam (Failure Mode)	Experiment al Load, P_u (kN)	Midspan Deflection Δ_{exp} (mm)	Reserve Capacity E_2 (J)	Total Energy Absorption ($E_1 + E_2$) (J)	Average Strain		Nominal Load, P_n (kN) ACI [4]	P_n/P_u
					$\varepsilon_{frp.avg}$ (%)	$\varepsilon_{c.avg}$		
80-#2S-0.5-S (GFRP Rupture)	15.0	81.8	0	742	2.8*	0.002	11.4	0.76
80-#3HM-1.0-S (Concrete Crushing)	33.0	62.6	2689	3909	1.6*	0.003*	27.2	0.82
80-#4HM-2.0-S (Concrete Crushing)	46.1	58.3	4540	6050	1.3*	0.0035*	32.0	0.70
120-#2S-0.5-S (GFRP Rupture)	16.2	77.5	0	714	3.5*	0.0017	11.4	0.70
120-#3HM-1.0-S (Concrete Crushing)	41.8	73.3	2335	4057	1.9*	0.004*	30.4	0.73
120-#4HM-2.0-S (Concrete Crushing)	52.2	64.3	4494	6377	1.6*	0.004*	36	0.69

Note:* Data was extrapolated using linear regression analysis to calculate average strain at Peak 1, $\varepsilon_{frp.avg}$ is the average strain in the GFRP tensile reinforcement bars and $\varepsilon_{c.avg}$ is the average strain in the concrete.

Table 4. Dynamic Amplification Factor (DAF)

GFRP-RC Beam	M_{static} (kNm)	M_d (kNm)	DAF
120-#3HM-1.0-I _{0.355}	21	25	1.19
120-#3HM-1.0-I _{0.533}	23	27	1.17
120-#3HM-1.0-I _{0.710}	23	*	*
120-#4HM-2.0-I _{0.550}	30	33	1.10
120-#4HM-2.0-I _{0.825}	32	39	1.22
120-#4HM-2.0-I _{1.1}	35	*	*
Mean			1.17

Note: * DAF could not be calculated since GFRP-RC beams totally collapsed under impact loading and dynamic load-time history response was inconclusive.

Table 5. Verification of failure modes of the GFRP-RC beams under impact loading

GFRP-RC Beam	Nominal shear strength as per ACI [4] (V_n) (kN)	Maximum experimental shear force (kN)	Failure mode
120-#3HM-1.0-I _{0.355}	51.9	27.4	Flexure
120-#3HM-1.0-I _{0.533}		37.6	Flexure
120-#3HM-1.0-I _{0.710}		49.2	Flexure-shear
120-#4HM-2.0-I _{0.550}	55.2	35.8	Flexure
120-#4HM-2.0-I _{0.825}		51.2	Flexure-shear
120-#4HM-2.0-I _{1.1}		60.4	Flexure-shear

Figures

a) $\rho_f = 0.5\%$

b) $\rho_f = 1.0\%$

c) $\rho_f = 2.0\%$

Figure 1. Reinforcement Cages

Figure 2. Schematic of a GFRP-RC Beam Specimen

Figure 3. Experimental Set-up for GFRP-RC Beams under Static Loading

Figure 4. Experimental Set-up for GFRP-RC Beams under Impact Loading

Figure 5. Rupture of GFRP Bar in GFRP-RC Beam 80-#2S-0.5-S

Figure 6. Load-Strain Response of Concrete and GFRP Bar in GFRP-RC Beams under Static Loading

Figure 7. Rupture of GFRP Bars in GFRP-RC Beam 80-#4HM-2.0-S

Figure 8. Load-Midspan Deflection Response of GFRP-RC Beams under Static Loading

Figure 9. Influence of Concrete Compressive Strength and GFRP Reinforcement Ratio on the Load Carrying Capacity

Figure 10. Influence of Concrete Compressive Strength and GFRP Reinforcement Ratio on Midspan Deflection

Figure 11. Influence of Concrete Strength and GFRP Reinforcement Ratio on Post-Cracking Bending Stiffness

Figure 12. High-Speed Camera showing Formation of Cracks at the point of Contact between the GFRP RC beam and Drop Hammer.

(a)

(b)

(c)

(d)

(e)

(f)

Figure 13. Failure Modes and Crack Propagation in UHSC GFRP-RC Beams: (a) 120-#3HM-1.0-I_{0.355}, (b) 120-#3HM-1.0-I_{0.533}, (c) 120-#3HM-1.0-I_{0.710}, (d) 120-#4HM-2.0-I_{0.550}, (e) 120-#4HM-2.0-I_{0.825}, 120-#4HM-2.0-I_{1.1}

Figure 14. Dynamic Midspan Deflection-Time Histories

Figure 15. Dynamic Force History Response of GFRP-RC Beams

Figure 16. Maximum Dynamic Strain-Time History Response of GFRP-RC Beams

Figure 17. Effect of Impact Energy on Dynamic Midspan Deflection

Figure 18. Effect of Impact Energy on Maximum Dynamic Strain

Static Loading

(a) At 50% Energy Absorption

(b) At 75% Energy Absorption

(c) At 100% Energy Absorption

Impact Loading

Figure 19. Failure Modes of GFRP-RC Beams ($\rho_f = 1.0\%$ and $f'_c = 120$ MPa) under Static and Impact Loading

Static Loading

(a) At 50% Energy Absorption

(b) At 75% Energy Absorption

(c) At 100% Energy Absorption

Impact Loading

Figure 20. Failure Modes of GFRP-RC Beams ($\rho_f = 2.0\%$ and $f'_c = 120$ MPa) under Static and Impact Loading

Figure 21. Dynamic Moment -Time History Response of GFRP-RC Beam 120-#4HM-2.0- $I_{0.825}$ under Impact Loading