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Sensitivity of Lateral Impact Response of RC Columns Reinforced with GFRP Bars and Stirrups to Concrete Strength and Reinforcement Ratio

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Abstract

This is the first study in the literature which experimentally and numerically investigates the impact behavior of concrete columns reinforced with fiber-reinforced polymer (FRP) reinforcements. The effect of concrete strength (50 MPa vs 100 MPa) and longitudinal FRP ratio on the lateral impact response of the columns was investigated. The experimental results showed that the longitudinal reinforcement ratio strongly affected the failure modes and impact-resistant capacity while the use of high strength concrete (HSC) did not effectively improve the performance of the columns, which on the contrary, might have caused spalling failure due to its brittleness. The peak impact force and displacement of the columns increased linearly with the impact velocity up to their maximum capacity. The longitudinal reinforcement ratio slightly affected the peak impact forces while the concrete strength showed marginal variation. The use of HSC did not effectively reduce the maximum displacement of the columns. The energy absorption of the columns and the impact velocity exhibited an approximately linear relationship regardless of the reinforcement ratio and concrete strength. Different from the static case, the numerical results show three critical sections, i.e. at the impact location and column ends, which need to be carefully designed.

Keywords: Impact response; Impact loading; GFRP bars; High strength concrete.

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Introduction

Columns of car parks, bridge piers, and lower story columns of buildings are vulnerable to impact loads induced by vehicle collisions. To avoid catastrophic losses of human lives and economy, protection and impact damage mitigation are important when designing reinforced concrete (RC) against possible impact loads. The impact response of reinforced concrete (RC) columns under transverse impact loading has been investigated experimentally [1-6] and numerically [7-10]. Huynh et al. [1] experimentally tested axially loaded high strength concrete (HSC) columns under multiple impacts. A few observations were reported, such as the failure modes of the columns shifted from flexure under static tests to shear or flexural shear under impact loads and the addition of steel fibers significantly increased the shear resistance of the columns. Pham et al. [3] experimentally investigated the lateral impact response of rubberized concrete columns. The authors found that rubberized concrete columns had lower peak impact forces as compared to normal concrete columns subjected to the same impact condition. In addition, the rubberized concrete columns exhibited higher impact energy absorption as compared to the reference columns. In the meantime, previous numerical studies on the lateral impact behavior of RC columns also provide interesting observations.

Do et al. [7] reported a numerical investigation of bridge columns – vehicle collision and found that the peak impact force caused a considerable increase in the axial stress. The authors also observed that the bending moments and shear forces varied significantly during an impact event and there are four critical sections in the columns that need to be considered in design. It is worth mentioning that the boundary condition in the previous studies was dissimilar so that it affected the columns differently. The shear resistance of RC columns at the impact point was found critical for the columns to resist impact loads. The use of different concrete strengths, fiber volumes, and fiber types significantly affect the shear resistance of concrete columns and thus the overall impact response, for example, the different shear behavior of normal and high strength concrete may lead to dissimilar impact response but it was not reported in the literature. Particularly, the number of studies on the impact resistance of RC columns is very limited as mentioned in the previous work [2, 3, 5, 6].

Meanwhile, concrete structures reinforced with normal steel reinforcements have been facing the costly issue of corrosion. Steel corrosion remains a major engineering and economic problem [11] and it has been estimated that the average annual cost of maintaining and improving bridges in the United States of America could respectively reach \$5.8 billion and \$10.6 billion during the period of 1998 to 2017 [12]. In Australia, GHD [13] estimated that there is a shortfall of \$17.6 billion AUD for maintenance expenditure of infrastructure including corrosion related issues from 2010 to 2024. It is, thus, imperative to build structures that have long durability and less maintenance requirement. In this circumstance, replacing steel by fiber-reinforced polymer (FRP) reinforcements is a great solution due to its advantageous properties including excellent corrosion resistance, high tensile strength, light-weight (20-25% of the density of steel) and easy applications [14, 15].

Previous studies have shown that concrete beams or columns reinforced with FRP reinforcements exhibited good performance under static loads [16-20] and dynamic loads [21-23]. Under static loading, RC beams reinforced with glass FRP (GFRP) bars exhibited larger deflection and wider crack widths as compared to steel reinforced concrete beams with equivalent reinforcement ratios because GFRP has lower elastic modulus (35-50 GPa) [24]. Accordingly, RC beams reinforced with GFRP bars also showed lower post-cracking bending stiffness and more damages compared to steel RC beams due to its low elastic modulus [19]. For impact loading, Goldston et al. [21] tested twelve GFRP RC beams under impact loads and also found that the failure mode shifted from flexure under static loads to shear. The authors reported an average dynamic amplification factor of 1.15 for these beams. Sadraie et al. [22] investigated the impact behavior of GFRP/steel RC slabs under impact loads and observed that these slabs exhibited good impact resistance but GFRP RC slabs yielded slightly less resistance than steel RC slabs. As can be seen that most of previous studies of the impact behavior of concrete structures reinforced with FRP rebars concentrated on beams while there has been no such study of FRP reinforced concrete columns under lateral impact loads.

The above review has shown that there is only one study on the impact response of HSC columns while no such study on GFRP RC columns can be found in the literature. Therefore, this study carries out a systematic investigation of the lateral impact response of HSC columns reinforced with GFRP bars and stirrups. In addition, the effect of GFRP reinforcement ratio on the impact response of the columns is also studied. The HSC columns reinforced with GFRP bars are tested with pendulum impacts, and advanced numerical simulation is also carried out to investigate the impact response of the columns.

Experimental program

Mix design and material properties

Two concrete mixtures with different compressive strengths covering normal strength and high strength concrete were adopted. The mix design of concrete is presented in Table 1. The compressive strengths of normal and high-strength concrete respectively were 51-56 MPa and 92-101 MPa in accordance with AS 1012.9 [25]. The compressive concrete strength of the columns at the testing date is given in Table 2. Slump tests were carried out for these concrete mixes and the achieved slump for all the mixes falling between 205-220 mm for normal concrete and 220-240 mm for HSC. The higher slump measured for HSC resulted from a larger amount of superplasticizer used in the mix as listed in Table 1.

GFRP reinforcements were used for both longitudinal and transverse reinforcements in these columns. The GFRP reinforcements were supplied by Pultron Composites, New Zealand [26]. The cross-section of each rebar has two parts, including fibers and glass coating. The total cross-section area of fibers is equivalent to the nominal diameter while the glass coating forms the actual diameter of the rebars. The nominal diameters, cross-sectional area, ultimate tensile strength, and guaranteed tensile strength of GFRP bars are presented in Table 3. Galvanized steel rebars were utilized as anchors at the connections with the footing and top slabs. Galvanized carbon steel rebar anchors were designed under tension and splice in accordance with AS 3600 [27]. The authors had no access to GFRP anchors (bent bars) during the time of the project, therefore corrosion resistance galvanized bent bars were used instead. The anchors were carefully designed to avoid premature failure at the connections between the columns and the footing/slabs to ensure large deformation of the columns which was the main objective of this study.

Specimen design and test matrix

A total of eight columns were cast and classified into two groups, including four columns made of normal concrete and the other four columns made of HSC. For each group, four different sizes of glass-fiber reinforcement were used which is 6 mm, 8 mm, 10 mm, and 12 mm in diameter while all the columns were reinforced with 8 mm square spirals stirrups at a spacing of 45 mm. A completed GFRP reinforcement cage and columns are shown in Fig. 1. Each column specimen had 400mm x 400mm square footing and top slab. Thicknesses of the footing and top slab were 140 mm and 60 mm, respectively. Cross-section of the column was 120 mm x 120 mm and height was 800 mm as shown in Fig. 1. The cross-section of the column was 1/5 or 1/10 of the full-scaled bridge model considered in the previous studies [7, 28]. This scale was selected based on the capacity of the pendulum impact system. The maximum impact velocity can be achieved from the pendulum system used in the test is 3.58 m/s while the mass of the steel projectile attached to the pendulum arm is limited to 300 kg. From the previous experimental and numerical studies [3, 7], normal-strength reinforced concrete columns with the cross-section dimension ranging from 80-120 mm failed under the maximum impact testing conditions, i.e. 300 kg projectile and 3.58 m/s. Therefore, in order to investigate the dynamic responses and the failure mode of the high-strength concrete columns in comparison to the normal-strength concrete columns, the cross-section size of 120 x 120 mm was chosen. The added mass on the column top was designed as 12 times of the column's self-weight. The longitudinal reinforcement ratio considered in the analyses varied from 0.79% to 3.14%, i.e. diameter ranging from 6 mm to 12 mm, which are the common ratio of longitudinal reinforcements.

The load-carrying capacities of these columns were estimated in accordance with AS 3600 [27] and ACI 440.1R-15 [29]. ACI 440.1R-15 [29] does not recommend GFRP bars to resist compression loads. The estimation of the axial load-carrying capacity is carried out using $f_{frp,c}$ taken as half of their ultimate tensile strength, i.e. 465 MPa while $f_{frp,y}$ is taken as 930 MPa as recommended in the previous studies [30, 31]. The maximum estimated bending moments and axial forces of these columns are summarized in Table 2. The axial forces and bending moments of the HSC columns were significantly higher than those of the corresponding normal strength concrete columns. It is worth mentioning that the shear resistance estimated based on ACI 440.1R-15 [29] is very conservative since the design procedure does not consider the contribution of the dowel effect of longitudinal GFRP bars and the aggregate interlock of the cracked section. Accordingly, the shear resistance of the two groups of columns was similar, which may not distinguish their different shear behavior. Therefore, the shear resistance is also estimated with the contribution from concrete of the whole effective section as suggested by ACI 318-14 [32].

Impact test setup

The impact test setup consists of an A-frame connected to a pendulum as shown in Fig. 2. The pendulum has a long steel arm of 2.8 m and the total weight of the impactor is 300 kg made of solid steel. The pendulum was raised to the desired height and dropped to collide with the columns. The release angle was measured by using an inclinometer. There were four release angles, i.e. 3^o, 10^o, 20^o, 30^o, and 40^o which correspond to Impact 0, Impact 1, Impact 2, Impact 3, and Impact 4. The impact velocities correspond to these release angles are 0.27m/s, 0.91m/s, 1.82m/s, 2.71m/s and 3.58 m/s, respectively. The column footing was firmly fixed to the strong floor by using four M16 steel bolts while the column top was free. A load cell was incorporated into the pendulum to measure the impact force at midheight of the columns. Once the impactor hit the column and rebounded, it was restricted from making the second impact by manually holding a rope connected to the impactor. To represent superstructure of columns, an added mass of 320.5 kg was firmly fixed to the top slab using four strong bolts as shown in Fig. 2. A data acquisition system with a high sampling rate of 50 kHz was used to record all signals. A high-speed camera is set at a distance of about 5 meters from the test device with a sampling rate of 20000 fps (frame per second). A total of 6 tracking points were fixed on the load cell and the columns for tracking displacement and velocities of the columns.

Experimental results

Impact response and failure modes

The column response under lateral impact loads can be classified into two phases including the local and global responses. When the pendulum went into contact with the column, it responded locally that the boundary condition effect has a marginal influence on the responses as shown in Fig. 3. During this local response phase, only a portion of the column close to the impact point responded to the impact load while the footing and the free column top remained stationary. This observation agreed well with the previous studies by Do et al. [7] on bridge piers under vehicle impact and Pham and Hao [33] on RC beams subjected to impact loads. Afterward, the entire column deformed under the impact and the column top reached its maximum displacement when the impact force had ceased.

The columns were repeatedly impacted with an increased release angle. Under Impact 0 (3°), the impact energy was small so that the columns did not exhibit any damage but vibrated in an elastic range. This impact level was utilized to determine the dynamic characteristic of the columns in the elastic range. After Impact 1 (10°), all the columns showed minor flexural hairline cracks on the rear surface of the columns as shown in Fig. 4. The top slab of the column vibrated considerably and resulted in concrete damage at the column-slab connection with debris flying out. The columns were severely damaged by Impact 2 (20°) but the columns still stood. Existing flexural cracks in the rear face widened while new shear cracks at the impact point and flexural cracks in the front face appeared. These shear cracks occurred in all the columns except Columns C01 and C05 which had a low flexural strength. These two columns only showed flexural cracks through the column height including the impact point. The column-slab connection experienced severe damage after this impact. These columns still survived after this impact condition even though they showed significant damage. After Impact 3 (30°), Columns C01 and C05 failed with very large residual displacement due to severe damage at the impact point, slab-column connection, and footing-column connection while other columns were able to carry on a further test, i.e., Impact 4 (40°). All the remaining columns completely failed at Impact 4 with different responses which will be discussed subsequently.

The impact response of the columns was affected by both the impact velocity (impact energy) and the longitudinal reinforcement. Columns C01 and C05 were reinforced with four 6-mm

GFRP bars so that the flexural capacity was significantly less than the shear capacity. For instance, the flexural capacity of Column C01 (3.95 kN.m) was approximately a half of that of Column C02 (6.22 kN.m) while their shear capacities were comparable (52.88 kN vs 54.03 kN) as presented in Table 2. Therefore, Column C01 exhibited more flexural cracks while Columns C02-04 were governed by flexural and shear responses as shown in Fig. 5. Similarly, Column C05 also experienced flexural cracks without shear cracks. The flexural capacities of the columns increase significantly with the reinforcement ratio. As a result, the columns reinforced with larger-diameter bars failed with two major shear cracks at the impact point, for example, the shear failure at the impact point of Column 06 as shown in Fig. 5. This column even did not show considerable flexural cracks. In addition, severe damage at the impact point and spalling in the back of the columns were observed in the columns made of high strength concrete at a high impact velocity. The premature spalling of concrete cover of high-strength concrete was also reported due to its brittleness [34, 35]. A number of technologies could be used to mitigate this problem, e.g., wrapping the column with FRP sheets or using ultra-high performance concrete with fibres to diminish the premature spalling failure and provide sufficient ductility to the impact response of the column.

Impact force

The impact force time histories of all the columns are presented in Figs. 6-7. The impact forces of the column increased quickly to the peak within about 5 ms and diminished at approximately 20 – 30 ms after impact. It is noted that Impact 0 was conducted with the released angle of 3^{o} of the projectile to ensure there were no cracks and all the columns were still in the elastic range. The maximum impact forces increased with the impact velocity as shown in Figs. 6-7 except Impact 4. This phenomenon is understandable since a higher impact velocity generates larger impact energy and thus higher impact force. However, the impact force is not only governed by the impact energy but also the contact stiffness as reported in the previous study [36]. After impact 3, all the columns showed substantial damage at the impact point and also other critical sections. It means that the contact stiffness significantly reduced and this phenomenon, in turn, led to a lower peak impact force even though the columns were subjected to higher impact energy. For example, the maximum peak impact forces of Column 03 under Impact 3 (30^o) and Impact 4 (40^o) were 149 kN and 126 kN, respectively. When the impact velocity was still small, i.e. Impacts 0-2, the maximum peak impact forces of Columns 1-4 were quite similar. For instance, the maximum peak impact forces of these columns were approximately 20 kN, 50 kN, and 100-110 kN for impact 0, impact 1, and impact 2, respectively.

It is also known that the dynamic responses include two phases including both local and global responses [33, 37, 38]. It means that if the impact duration is longer and closer to the duration of the natural vibration period of a structure, the global response is more prominent [39]. Meanwhile, the impact duration of these columns increased with the impact velocity. For instance, the impact force duration of Column 01 was 10 ms, 15 ms, 25 ms, and 35 ms when the impact velocity increased from impact 0 (3^o), impact 1 (10^o), impact 2 (20^o), to impact 3 (30^o) due to damage in each applied impact, respectively. It is worth mentioning that the impact duration is resulted from the sophisticated interaction between the impactor and specimen, which is governed by the contact stiffness, structural stiffness, impact energy and impact velocity. As a result, an impact with a higher velocity associated with higher kinetic energy requires a longer impact duration to transfer energy to the specimens if other parameters remain the same, i.e. contact stiffness and structural stiffness. The natural vibration period of these columns can be estimated as about 162 ms for normal strength concrete (equivalent Young's modulus of concrete was approximately 33 GPa), similar to the previous study [7] (Eq. 1). As a result, the influence of the global response (structural stiffness and boundary condition) on the impact responses of these columns becomes more prominent under impacts 3-4. Therefore, the columns with greater rigidity respond to the same impact velocity with a higher impact force. For instance, under impact 3, the peak impact forces of the columns were approximately 86 kN, 121 kN, 149 kN, and 138 kN for Columns 01-04 which were reinforced with 4D6, 4D8, 4D10, and 4D12, respectively (Fig. 6). It is noted that the maximum impact force of Column 04 was slightly smaller than that of Column 03. This variation did not follow the general trend of other columns and it may be attributed to the error in measuring the impact force, which significantly fluctuated.

$$T = 2\pi\sqrt{\frac{m}{k}} = 2\pi\sqrt{\frac{m}{\frac{3EI}{L^3}}} \tag{1}$$

where *m* is the total of the effective mass of the column and the added mass, *k* is the global stiffness of column under lateral static load, *E* is the equivalent Young's modulus of material, *I* is the moment of inertia, and *L* is the effective length of the column considering the dimension of the added mass [7].

The peak impact forces of the columns increased linearly with the impact velocity until impact 2 (20°, 1.82 m/s) for Columns 1-2 and 5-6 and impact 3 (30°, 2.71 m/s) for Columns 3-4 and 7-8 as shown in Fig. 8. It means that if the impact force was still smaller than the impact resistance the columns, the peak impact force proportionally increased with the impact velocity. When the impact force causes considerable damage and is higher than the impact resistance of the columns, the peak impact forces declined. The longitudinal reinforcement ratio slightly affected the peak impact forces while the concrete strength exhibited a corresponding marginal variation (Fig. 8). For instance, the variations of the peak impact force of Columns 1 and 4, which had the same concrete strength but different FRP ratio, were 0% at impact 1 and 18% at impact 2. Meanwhile, when comparing the peak impact forces of two columns with the same reinforcement but different concrete strength, the variation was marginal, i.e. under impact 2, C05 vs C01 (0% variation), C06 vs C02 (8% variation), and C07 vs C03 (-3% variation). Therefore, to improve the impact resistance of a column, it is recommended that increasing the longitudinal reinforcement ratio is more effective than using a higher concrete strength.

Displacement and energy absorption

The midheight displacement time histories of all the columns are shown in Figs. 9 and 10. The maximum displacement of the columns under Impact 0 was very small and the columns returned to their original position quickly. The columns' response was still in the elastic range and no crack was observed. Under higher impact angles associated with higher impact velocities, the impact forces were higher and the maximum displacement was also larger. The columns with a higher longitudinal steel ratio experienced smaller maximum displacement as expected. For example, under Impact 3, Column C01 exhibited the maximum displacement of 57 mm while the corresponding displacement of Column C04 was just 11 mm. The similar observation was also recorded for the high-strength concrete columns in which greater longitudinal reinforcement ratio significantly reduced the maximum lateral displacement. It is worth mentioning that the residual displacement of the columns reinforced with GFRP bars seemed a lot smaller than those of traditional RC columns as shown in Fig. 10 of the previous study by Pham et al. [3]. In most cases, the residual displacement was less than approximately 50% of the corresponding maximum displacement. On the other hand, the residual displacement of the RC columns in the previous study [3] was approximately 80% of the maximum displacement. This observation indicates that with their elastic behaviour, GFRP bars provided better centering capability, in which the columns can restore back to their original position.

The maximum displacements at the midheight (impact point) and column top are summarized in Table 4. It can be seen that using a greater reinforcement ratio can significantly reduce the maximum displacement of the columns when comparing the displacement of Columns 1-4 or Columns 5-8. Meanwhile, the concrete strength exhibited minor influence on the maximum displacement. For example, under impact 2, the maximum displacement at the impact point of Columns 2 and 6 was 13 mm and 12 mm, respectively. Under impact 3, the corresponding value of these two columns was 27 mm and 24 mm, respectively. To further investigate the effect of the concrete strength and longitudinal reinforcement ratio on the column displacement, the midheight displacements of Columns 1, 4, 5, and 8 were plotted in Fig. 11. Columns 1 and 4 used had similar concrete strength but different longitudinal reinforcement ratios. The midheight displacement of Column 1 was significantly larger than that of Column 4, with an increase by 113%, 143%, and 425% for impacts 1, 2, and 3, respectively. Meanwhile, the midheight displacement of Columns 1 and 5, which had the same longitudinal reinforcement ratio but different concrete strengths (56 MPa vs 101 MPa), showed smaller variations. The increase of the midheight displacement of Column 1 with respect to Column 5 was -12%, 31%, and 40% for impacts 1, 2, and 3, respectively. Interestingly, the use of high strength concrete even resulted in a negative effect in some particular cases. For instance, the midheight displacement of Column 4 was smaller than that of Column 8 although the two columns had similar reinforcements and the later one used higher concrete strength (56 MPa vs 92 MPa). Since the displacement of the columns is sensitive to the longitudinal reinforcement ratio and the elastic modulus of concrete, high strength concrete possesses higher elastic modulus, it affects the displacement response, but its influence is marginal. Meanwhile, high strength concrete is brittle and thus it might exhibit more severe damage under similar impact condition, i.e. spalling damage in this study. Previous studies on high-strength concrete structures have suggested that the brittleness of high-strength concrete may cause an adverse effect on the structural performance [34, 40]. If the increased performance of high strength concrete could not counteract the adverse effect of the brittleness, high strength concrete might give negative effects on the structural performance as reported in previous studies. Therefore, it can be concluded that column displacement can be significantly reduced by using greater longitudinal reinforcement ratio while using higher concrete strength does not effectively reduce displacement or even increases displacement of a column.

The energy absorption is an essential index for the impact-resistant capacity of these columns. The energy absorption is defined as the enclosed area within the impact force vs midheight displacement curves. Figs. 12-13 show graphs of the impact force versus the midheight displacement of all the columns. The energy absorption was summarized in Table 5. Under higher impact velocity, the columns experienced higher impact force and displacement and thus energy absorption. As presented previously, the maximum impact force increased with the impact velocity from impact 0 to impact 3. As a result, these columns also absorbed a lot of energy when the impact velocity increased. Except Columns 01 and 05 failed at impact 3, the other columns showed higher energy absorption in impact 4 as compared to that in impact 3 although the maximum impact forces under impact 3 were greater than impact 4. Under impact 4, the maximum impact forces were slightly smaller than those under impact 3 as explained previously but the maximum displacement substantially increased. To examine the relationship between the energy absorption of these columns and the impact energy, Fig. 14 shows the energy absorption vs impact velocity graphs. As can be seen that the energy absorption exhibited an approximately linear relationship with the impact velocity (impact energy) until failure, regardless of the concrete strength and longitudinal reinforcement ratio.

Numerical simulation

Model development

Numerical models of these columns were developed by using the commercial software, namely ANSYS-ADPL/LS-DYNA. Details of the entire column and various components are presented in Fig.15. In the simulation, the impactor, steel plates, and all the concrete elements, i.e. footing, column, slab, and added weight were modelled by solid elements (hexahedral elements with one integration point) while the GFRP reinforcements and steel anchors were simulated by beam elements (Hughes-Liu with cross-sectional integration). It is noted that the above-mentioned element types are normally used for RC structures in LS-DYNA. Their accuracy in simulating the dynamic responses of RC structures under impact load has also been reported previously [7, 8, 36, 38]. The K&C concrete material model, i.e. *Mat_072R3 in LS-DYNA, was used to simulate the dynamic behaviors of normal and high strength concrete. The accuracy of this material model in predicting the dynamic response and damage of normal and high strength concrete has been widely proved in previous studies [7, 9, 33, 41, 42]. It is noted that there have been a few models for HSC and they were employed in the trial simulations but the results do not match well with the experimental tests while numerical simulations with K&C concrete material model (*Mat_072R3) generate reasonable prediction, i.e. elastic–plastic hydrodynamics model (Mat_010) [43] and continuous surface cap model (CSCM_Mat159) [44]. These models for HSC usually incorporate steel fibers so that it is different from this study. Therefore, material model *Mat_072R3 is adopted to simulate both normal and high strength concrete in this study. In the simulation, the unconfined compressive strengths of NSC and HSC are 50 MPa and 100 MPa, respectively. Furthermore, the *Mat_Piecewise_Linear_Plasticity (MAT_24) material model was employed in the numerical model to simulate the steel anchorages while the behavior of GFRP reinforcements and steel impactor were modelled by *Mat_Elastic. The properties of these materials are given in Tables 2 and 3. Moreover, the LS-DYNA function named *Mat_Erosion was also adopted in the simulation to eliminate the concrete and GFRP elements which are damaged under impact load and no longer contribute to resisting the load. In the simulation, the maximum principal strain was used as an erosion criterion, as suggested in previous studies [9, 37]. The erosion criterion of concrete was chosen at 0.5 after trials while the corresponding value of GFRP reinforcements was set up at 0.12 based on its material properties. Furthermore, the increase of the compressive and tensile strengths of materials under dynamic loads has also been taken into account in the numerical model through the dynamic increase factor (DIF). It is worth mentioning that there has been no DIF equation available in the literature so that the strain rate effect of GFRP bars has not been considered. For concrete, the DIF model proposed by Hao and Hao [45] was used in the simulation since this model has eliminated the effect of the end friction confinement and lateral inertia confinement contribution from the material dynamic tests. This model also shows advantages in modelling RC structures under both low and high loading rates as reported in the previous study. The corresponding model of steel proposed by Malvar [46] was chosen. More information about the simulation technique can be found in the authors' previous studies [28, 33, 37].

In the simulation, the contact between the impactor and the column and the added weight and the top slab was modelled by the LS-DYNA contact keyword, namely *Contact_Automatic_Surface_to_Surface. The static coefficient of friction was used at 0.6 for these contacts [7, 28, 32]. To accurately simulate the impact force time histories of the column-impactor contact, the scale factor of slave and master penalty stiffness was 0.035 and 0.035, respectively. It is noted that these penalty stiffness are one of the most important parameters which need to be defined in the simulation by using trial-error approach in order to achieve a reasonable comparison between simulation and testing results as suggested by the previous study [36]. Furthermore, the contact between the GFRP reinforcements, steel anchorages and their surrounding concrete was assumed as perfect bonded because no slippage between the GFRP bars and concrete was recorded in the tests. Since no displacement or rotation of the column footing was recorded, the base of the footing was fixed in all directions in the simulation, as shown in Fig. 15.

Model verification

The model verification covers columns with different concrete strengths, i.e. 50 MPa and 100 MPa. The verification is also considered in columns which have sufficient impact force and not severe damage to ensure the high accuracy. It is noted that severe damage under Impacts 3 and 4 was cumulated through previous impacts and thus errors are also accumulated. Therefore, the verification is conducted under Impacts 1 or 2 only. The comparisons between the numerical simulation and experimental test in terms of impact force time histories and column damage are presented in Fig. 16. It can be seen in Fig. 16a that the impact force time histories of Column C02 under impact 1 and Column C08 under impact 2 were reasonably simulated by the numerical simulation. For instance, the peak impact forces of the Column C02 under impact 01 in the experiment and the simulation were 57 kN and 61 kN, respectively. For the Column C08 under Impact 02, the peak impact force in the simulation was 117 kN compared to the corresponding result of 115 kN in the experimental test. Furthermore, the damage contour in the column under impact loads is also presented in Fig. 16b to compare with the column damage as observed in the experiment. The figure shows that the flexural cracks on the front face, impact side, and rear face of the two columns observed in the experimental tests were reasonably simulated by the numerical model. Interestingly, as illustrated in Fig. 16b, on the rear surface the flexural cracks occurred in the top part of the two columns (from the impact location to the column top) while on the impact side, these cracks happened in the lower part (from the impact location to the footing). It is noted that the cracks in these surfaces of the columns were observed for all the eight tested columns. This observation in the experimental test is also simulated in the numerical model (see Fig. 16b). This is because of the variation of the bending moment along the column during the impact which will be discussed in the subsequent section.

Meanwhile, the experimental and numerical impact force time histories are slightly different due to two reasons. Firstly, as observed in the previous studies that the impact force time histories between the projectile and column structures are significantly affected by the contact stiffness between the two components [36]. A slight variation of the contact stiffness may cause a considerable difference in the waveforms of the impact force. Pham et al. [36] reported that the peak impact force of two similarly reinforced concrete beams was three times different owing to a slight change in the contact stiffness even though they were subjected to the same impact velocity and drop-weight. Due to the complexity of the experimental impact test, the contact stiffness is normally difficult to be simulated accurately in the numerical model. Accordingly, the peak impact force and impulse are two main parameters that are usually used for model validation as they are the governing parameters to structural responses. These parameters are reasonably validated in this study. Moreover, as discussed previously, this is the first study investigating the impact response of the high-strength concrete columns reinforced by GFRP bars. Current material models available in LS-DYNA are not necessarily reliable for high strength concrete. Therefore, slight differences in the column response are probably unavoidable.

In general, the comparisons between the numerical and experimental results showed that the presented simulation technique has the ability to yield a reasonable prediction of the normal strength and high strength concrete columns under impact loads. The numerical simulation was then used to investigate the distribution and variation of the bending moments and shear forces along the column during the impact load to further understand the impact response of the two columns that could not be recorded in the experiment.

Bending moment and shear force diagrams

The bending moment and shear force distributed along the Column C08 under impact 02 are presented in Fig. 17. The bending moment and shear force diagrams are built by connecting the bending moment and shear forces at 8 sections along the columns. These 8 sections are uniformly distributed with the spacing of 100 mm from the top of the footing to the top slab. As can be seen in the figure that due to the contribution of inertia force as well as the added weight at the column top, the bending moment and shear force in the column vary significantly under the impact load with several critical sections, i.e. impact locations and column ends, which need to be carefully considered in design analysis. Under impact loads, the bending moment and shear force in the column include three main phases, i.e. at the peak impact force, 5-12 ms post the peak impact force, and free vibration phase. At the peak impact force, the column responds to the impact force as a fixed-fixed connection with the maximum positive bending moment at the impact location while that at the column base and column top reaches the maximum value in the negative side, see Fig. 17a – From 0 ms to 2.5 ms. This mechanism is explained and discussed above and also described in Fig. 3. It is noted the bending moment at the column top occurs in the negative side because of the inertia resistance from the added weight which was also observed in the previous studies [7, 47].

In the meantime, the maximum shear force is also observed at the column ends. The column experiences the maximum shear force at the peak impact force so the shear force in this phase should be used for design for shear resistance. In addition, the shear force in the portion from the impact point to the footing is greater than that in the portion above the impact point to the column top. The bending moment at the column top and along the top part of the column then shift to the positive side while the bending moment at the lower part of the column remains in the negative side, see Fig. 17a. These variations of the bending moment thus cause the flexural cracks in both sides of the columns (see Fig.16b) and damage at the column top (Fig. 4). In this phase, the shear force at the lower part of the column also happens in the negative side while the shear force at the top part of the column is insignificant. In the free vibration phase, the bending moment and shear force fluctuate around zero levels. The bending moment time histories at the column ends and at the impact location are also presented in Fig. 18. Owing to the variation of bending moment and shear force, the axial stress in the GFRP reinforcements and steel anchorages also varies, as presented in Fig. 19.

From the numerical and experimental results, it can be observed that in the design analysis of the concrete column under impact loads, not only the bending moment capacity at the impact location and the column base but also at the column top need to be carefully designed due to the occurrence of the maximum bending moment at these locations. Also, the bending moment at the column top might occur in both sides of the column due to the variation of the inertia force and the influence of the added weight. Moreover, the results also indicate the significant contribution of the steel anchorages as provided in this study when it could prevent the flexural failure at the column base observed in the previous study without anchorages [3]. In this study, no flexural failure at the column base was observed in all the tested columns.

Contribution of GFRP reinforcements to the column capacities

Owing to variation of the bending moments and shear forces, the axial stress in the GFRP reinforcements also varies. To investigate the contribution of GFRP reinforcements in the columns, the column was subjected to a higher impact velocity of 3.58 m/s, corresponding to Impact 4 in the experiment as presented in Fig. 19. As can be seen that the axial tensile stress in the longitudinal GFRP bars on the positive side at the column mid-height increases to about 600 MPa at the peak impact force (see Fig. 19a) while the axial compressive stress of GFRP bars on the negative side reaches -125 MPa. When the bending moment at the column base reaches its highest value, the axial tensile stress of GFRP reinforcement on the negative side also increases to nearly its ultimate tensile strength at about 910 MPa while that on the positive side is about -125 MPa. Meanwhile, the axial tensile stress in the transverse GFRP stirrups at the column base also increases to its highest value of about 300 MPa when the shear force reaches the maximum value, as presented in Fig. 19b. The variation of the axial stress in the GFRP bars ranging from -125 MPa to 910 MPa shows the significant contribution of GFRP bars in resisting the resulting bending moments and shear forces in the column during the impact loads. These results indicate that GFRP bars can potentially replace steel reinforcements in concrete structures to resist impact loading. Furthermore, to further investigate the difference between steel and GFRP reinforcements in contributing to the impact-resistant capacity of the column, a numerical simulation of the column with steel reinforcements is then simulated. In the numerical model, the GFRP bars are replaced by steel bars with the yield strength of 500 MPa while all the other parameters of the column are kept unchanged. It is noted that material model *Mat_Piecewise_Linear_Plasticity is adopted for steel reinforcements, more information can be found in the previous studies [7, 28]. The comparison of the axial stress in the reinforcements is presented in Fig. 20. The figure shows that the columns with GFRP bars and steel bars behave similarly in which the distribution of the stress in the reinforcement cage during the impact force phase is comparable. However, because of the higher ultimate tensile strength, the maximum axial stress in the GFRP bars (about 910 MPa) is higher than that of steel reinforcements (670 MPa). It is worth mentioning that steel has yielded and its tensile stress is greater than its static tensile strength of 500 MPa which corresponds to a dynamic increase factor of 1.34. In contract, due to the higher Young's modulus (200 GPa for steel and 76 GPa for GFRP reinforcements), the compressive stress in the steel reinforcements (-350 MPa) is higher than that of GFRP bars (-125 MPa). Meanwhile, the maximum displacement at the top of the column with GFRP reinforcements (35.5 mm) is higher than that of the column with normal steel bars (27.6 mm) due to the lower stiffness of GFRP compared to steel bars, as shown in Fig. 21. Therefore, the large displacement response of concrete columns reinforced with GFRP reinforcements should be considered when choosing an appropriate material.

Conclusions

This study experimentally investigates the lateral impact responses of concrete columns reinforced with both longitudinal and transverse GFRP bars and stirrups which significantly contribute to the capacity of the columns. The experimental observations were then verified with the numerical results. The findings from this study with both the experimental and numerical results are as follows:

1. The longitudinal reinforcement ratio strongly affected the failure modes and impact-resistant capacity. The longitudinal GFRP bars contributed significantly to the capacity of the columns in which the maximum compressive and tensile stresses in the longitudinal GFRP bars were approximately 910 MPa (98% rupture strength) and 125 MPa (14% rupture strength), respectively. The maximum tensile stress in GFRP stirrup was 300 MPa.

2. Using HSC did not effectively improve the impact-resistant capacity of the columns, but may even have caused spalling failure in the backside at a high impact velocity due to its brittleness.

3. The peak impact force and displacement of the columns increased linearly with the impact velocity until the columns reached their capacity. Accordingly, the energy absorption of the columns linearly increased with the impact velocity regardless of the reinforcement ratio and concrete strength.

4. The longitudinal reinforcement ratio and concrete strength showed a marginal effect on the peak impact forces.

5. The use of the steel anchorages in this study significantly improved the bending moment capacity of the column thus it prevents the flexural failure at the column base.

In general, to improve the impact resistance of concrete columns, they should be reinforced with a higher longitudinal ratio while increasing the concrete strength is relatively ineffective in the ranges of configurations and loadings considered. Moreover, due to the variations of the bending moment and shear force caused by the inertia force and the added mass, three critical sections, i.e. at the impact location and the column ends, need to be carefully designed.

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Tables

Table 1. Concrete mixtures of normal and high strength concrete

Mix design per 1 m^3 of normal strength concrete		Mix design per 1 m^3 of high strength concrete	
Water (kg)	205	Water (kg)	212
Cement (kg)	426	Cement (kg)	667
10 mm aggregate (kg)	444	Fly ash (kg)	174
7 mm aggregate (kg)	306	Silica Fume (kg)	82
<4 mm aggregate (kg)	130	7 mm aggregate (kg)	605
Fine aggregate (kg)	843	Fine aggregate (kg)	636
Sika Viscocrete 10 (ml)	3750	Sika Viscocrete 20HE (ml)	7083
-	-	Sika Viscocrete PC HRF-2 (ml)	1417

Table 2. Test matrix and design of all the columns

Group	Specimen	Longitudinal reinforcements	Reinforcement ratio (ρ)	f_c' (MPa)	Axial force (kN)	Bending moment (kN.m)	*Shear resistance (kN) [29]
Normal concrete	C01	6 mm	0.64	56	724.87	3.95	52.88
	C02	8 mm	1.23	51	699.94	6.22	54.03
	C03	10 mm	2.02	55	796.89	7.81	55.33
	C04	12 mm	2.89	56	862.14	9.07	56.39
HSC	C05	6 mm	0.64	101	1,105.62	3.95	53.46
	C06	8 mm	1.23	97	1,110.42	7.58	54.92
	C07	10 mm	2.02	98	1,166.36	10.85	56.39
	C08	12 mm	2.89	92	1,170.83	12.18	57.49

Note: * The shear resistance is estimated in accordance with ACI 440.1R-15 [29]

Table 3. Material properties of GFRP rebars (from the manufacturer [26])

Rebar diameter (mm)	6	8	10	12
Nominal diameter (mm)	5.2	7.2	9.2	11.0
Cross-sectional area (mm^2)	21.2	40.7	66.5	95.0
Ultimate tensile strength (MPa)	930	930	930	930
Guaranteed tensile strength (MPa)	911	910	907	904

Table 4. Maximum displacement of the columns

	Maximum displacement at mid-height (column top), mm			
Column	Impact 1 (10^{o}) 0.91 (m/s)	Impact 2 (20^{o}) 1.82 (m/s)	Impact 3 (30^{o}) 2.71 (m/s)	Impact 4 (40^{o}) 3.58 (m/s)
C01	6 (10)	21 (40)	57 (94)	-
C02	5 (12)	13 (26)	27 (55)	86 (157)
C03	4 (7)	11 (19)	23 (44)	41 (75)
C04	3 (4)	9 (17)	11 (39)	38 (74)
C05	7 (8)	16 (31)	41 (80)	-
C06	5 (6)	12 (23)	24 (50)	46 (50)
C07	5 (8)	12 (23)	25 (50)	46 (87)
C08	4 (6)	10 (19)	20 (42)	37 (71)

Note: - Not applicable

Table 5. Energy absorption of the columns

	Energy absorption (N.m)			
Column	Impact 1 (10^{o}) 0.91 (m/s)	Impact 2 (20^{o}) 1.82 (m/s)	Impact 3 (30^{o}) 2.71 (m/s)	Impact 4 (40^{o}) 3.58 (m/s)
C01	166	730	1454	-
C02	182	821	1647	2922
C03	128	621	1706	2166
C04	112	617	1046	2411
C05	151	745	1682	-
C06	186	764	1619	2842
C07	181	717	1494	2474
C08	155	737	1759	2764

Note: - Not applicable