Failure and Impact Resistance Analysis of Plain and FRP-Confined Concrete Cylinders under Axial Impact Loads

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Abstract

This study conducts an experimental and numerical investigation on the failure and impact resistance of plain and FRP-confined concrete. The impact resistance of concrete cylinders wrapped with different types of fibers including carbon fiber and glass fiber is examined. Drop-weight tests are utilized to conduct the impact tests while the numerical simulation is conducted by using LS-DYNA. The experimental and numerical results have proved that fiber reinforced polymer (FRP) can be efficiently used to improve the impact resistance of concrete cylinders. In general, FRP ruptures at a lower strain than those in static tests and the rupture strain of glass fiber is much higher than that of carbon fiber. The findings in the experimental tests are confirmed by the numerical results. Glass fiber, therefore, exhibits a much better performance than carbon fiber. It is recommended to use glass fiber to enhance the impact resistance of concrete structures strengthened with FRP. In addition, the stress evolution of the specimens is analyzed to investigate the failure mechanism.

Keywords: Fiber Reinforced Polymer; Impact loading; Impact resistance; Strengthening; Retrofitting.

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19 **Introduction**

20 Concrete confined with fiber reinforced polymer (FRP) has shown excellent improvement as
21 compared to referenced plain concrete in terms of strength and energy absorption capacity
22 under static loads [1-3]. However, these parameters under dynamic loads have not been well
23 examined. There have been a few studies investigating the dynamic behavior of confined
24 concrete [4-6]. Shan et al. [4] used gas gun equipment to investigate the impact resistance of
25 confined strength of the specimens under impact loads. The impact velocities ranging from 52
26 m/s to 200 m/s were used in that study, which gave the strain rates in the range between 389 s⁻¹
27 and 1621 s⁻¹ and the specimen damage was observed initiating from the impact end. Strain
28 gauges were utilized to monitor the stress wave velocity. The authors reported a very high
29 range of the dynamic increase factor ranging from 2.62 to 44.71.

30 Uddin et al. [5] utilized an Instron drop-tower testing machine to perform impact tests on the
31 concrete cylinders confined with FRP. The drop height was kept constant at 30 cm with
32 multiple impacts. To eliminate the inertia effect, the authors attached a rubber pad to the
33 projectile. The experimental results of the impact force time histories showed the first peak of
34 these specimens were quite similar with a slight increase (~18%) for the confined specimens.
35 In addition, the impact resistance capacities also experienced a marginal variation (~10%)
36 between the tested specimens. Strain rate was not reported in the mentioned study.
37 Meanwhile, Xiao and Shen [6] conducted a study on impact behaviors of CFRP-confined
38 concrete by using a drop-weight testing apparatus, which has the maximum projectile weight
39 of 800 kg dropping from 16 m. Specimens failed by rupturing of CFRP jackets at about 2 ms
40 after the impact event. The progressive failure initiated from the specimen top and propagated
41 downward to the mid height. The authors observed that when the impact energy increased, the

maximum impact forces did not considerably change but the impact duration slightly extended.

There are very limited studies of impact behavior of unconfined and confined concrete. The above studies have presented some qualitative observations from testing. This study attempts to provide more information and analysis about the axial impact resistance of unconfined and confined concrete. Stress wave propagation and progressive failure are examined and explained. The impact resistance of plain concrete and FRP confined concrete is also investigated. In addition, numerical simulation is conducted to verify the experimental results and perform further analysis on the progressive failure and FRP strain.

Stress wave propagation in cylinders

Considering a short cylindrical block of elastic-linear strain-hardening material sitting on a frictionless flat rigid base is impacted by a rigid projectile with a speed V . When a projectile impacts a cylinder, a stress wave propagates from the impact end to the other end. There are two possible wave speeds including the elastic wave speed (c_0) and the plastic wave speed (c_1) [7]. When impact with plastic deformation takes place, these two waves simultaneously propagate from the impact end. They travel along the cylinder at speeds of c_0 and c_1 as shown in Fig. 1b. A stress wave will travel with a plastic wave speed when the stress (σ_0) is greater than the yield stress (Y). As shown in the book by Johnson [7], stress in the cylinder is changing along the height associated with time under uniform impact velocity (V) within a period of time $6L/c_1$. Since the stress in the cylinder changes in a sophisticated rule, it is hard to estimate where the failure initiates. It is noted that concrete does not have an idealized uniform characteristic as assumed in [7] and in a drop-weight impact test it is not possible to maintain an unchanged impact velocity in a period of time. It is, therefore, difficult to predict where failure initiates.

Figure 1. Stress evolution mechanism under impact

The elastic and plastic wave velocity can be calculated by

$$c_0 = \sqrt{\frac{E}{\rho_0}} \tag{1}$$

$$c_1 = \sqrt{\frac{P}{\rho_0}} \tag{2}$$

where E and P are respectively the Young's modulus and plastic modulus of material, and ρ_0 is the density of the material in its unstrained state. After the elastic stress wave reaches the stationary base, the material is already stressed to the compressive yield state. Accordingly, the reflected wave from the stationary base must be a plastic wave as shown in Fig. 1e. The incoming plastic wave meets the reflected plastic wave at A. After this instant, there are only two plastic waves acting in the material while no elastic wave is produced although the material is still loaded. Johnson [7] presented a solution to calculate stress in the specimen at any instant within $6L/c_1$, which is not linearly distributed along the longitudinal axis of the cylinder. If the impact velocity is sufficiently high, the stress in Zone 2 at the impact end,

which is proportional to the impact velocity, may be excessive than the failure stress of the material and thus damage occurs at the impact end. Otherwise, the specimen may damage at the base end (Zone 3) if the material at the impact end can resist the impact. It is noted that damage rarely occurs at Zone 1 since the stress in this zone is not proportion to the impact velocity but the material properties. In general, damage occurs at a zone when the stress at that zone reaches the failure stress of the material as presented in Fig. 1f. Therefore, predicting failure position of a specimen is really a challenging task for a non-homogeneous material, such as concrete.

Experimental program

Test matrix and materials' properties

Concrete cylinders were cast and tested until failure under drop-weight tests. The cylinders were 100 mm in diameter and 200 mm in height. The compressive strength of concrete was 46 MPa at 28 day age. These cylinders wrapped with Carbon FRP (CFRP) and Glass FRP (GFRP) of different schemes representing heavy confinement, sufficient confinement and insufficient confinement as shown in Fig. 2. Details of the specimens and testing results are presented in Table 1. For easy reference, names of the concrete cylinders include three parts: the first part is Letter C and G stating the type of fiber. The second part indicates the number of FRP layers. The third part refers to the drop height at which the projectile will be released. For instance, Cylinder C1-2 means this specimen is wrapped with one CFRP layer and is tested under 2 m drop height. The wrapping arrangement and specimens' names are illustrated in Fig. 2. If a specimen does not fail in the first drop, it will be repeatedly tested under the same drop height until failure. The name of these specimens will be added one more number in a bracket to indicate the number of drops until failure occurs as shown in Table 1. It is

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worth mentioning that the confined strength under static loads is estimated based on the model by Pham and Hadi [8] for FRP-confined concrete.

Figure 2. Wrapping schemes and specimen design (all dimensions in mm)

Table 1. Test matrix

Specimen	Number of FRP layers	Drop height (m)	Theoretical estimation – static (kN)	Experimental impact results (kN)	Impact duration (ms)	Impulse (kN.ms)
R-1	-	1	361	406	0.9	210
R-1.5		1.5		496	0.9	228
R-2		2		565	0.6	228
C1-1.5	1 CFRP layer	1.5	546	651	1.4	394
C1-2		2		#	#	
C1-2.5		2.5		#	#	
C1-2.5(2)		2.5		756	1.0	320
C2-2	2 CFRP layers	2	737	#	#	
C2-2.5		2.5		915	2.1	993
C2-2.5(2)		2.5		952	1.8	822
C2-3		3		#	#	#
G1-1.5	1 GFRP layer	1.5	434	#	#	#
G1-2		2		993	1.0	470
G1-2.5		2.5		#	#	#
G2-2	2 GFRP layers	2	513	#	#	#
G2-2(2)		2		#	#	#
G2-2(3)		2		772	1.2	477
G2-2.5		2.5		872	1.6	954
G2-3		3		1024	1.1	564

G3-2.5	2.5	952	1.9	1017		
G3-2.5(2)	2.5	1035	1.9	1030		
G3-2.5(3)	2.5	1089	2.0	1072		
G3-2.5(4)	2.5	833	2.5	1051		
G3-3	3 GFRP layers	3	593	1202	2.4	1087
G3-3(2)	3	1010	2.4	1099		
G3-3(3)	3	878	1.5	719		
G3-3.5	3.5	1021	2.7	1156		
G3-3.5(2)	3.5	964	2.9	1036		

CFRP = carbon fiber reinforced polymer

GFRP = glass fiber reinforced polymer

() indicate the number of drops if the specimen did not fail at the first drop

- not applicable

data lost

FRP was bonded to the substrate of concrete by epoxy resin which has a tensile strength of 54 MPa, tensile modulus of 2.8 GPa, and 3.4% tensile elongation [9]. The adhesive used was a mixture of epoxy resin and hardener at 5:1 ratio. Before the first layer of FRP was attached, the adhesive was spread onto the specimen's surface and FRP was attached to the surface. After the first ring, the adhesive was spread onto the surface of the first FRP layer and the second layer was continuously bonded, ensuring that 100-mm overlap was maintained.

The FRPs are the same types from the same supplier used in a number of previous studies [10-13]. In these studies, at least five CFRP coupons were fabricated and tested according to ASTM D3039 [14]. The CFRP used was 75 mm in width with a unidirectional fiber density of 340 g/m². The nominal thickness of CFRP was 0.45 mm and the tensile strength was 1548 MPa. The average strain at the maximum tensile force and the average elastic modulus were 1.74% and 89 GPa, respectively. The GFRP used was 50 mm in width with a unidirectional fiber density of 440 g/m². The nominal thickness of GFRP was 0.35 mm and the tensile strength was 833 MPa. The average strain at the maximum tensile force and the average elastic modulus were 1.97% and 41 GPa, respectively.

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128 In order to measure the lateral strain of FRP wraps and the axial strain of the specimens, strain

129 gauges are attached to three different positions which are top, middle and bottom of the

130 specimens. Details of these strain gauges are presented in Fig. 2. Strain gauges are bonded out

131 of the overlap zone of FRP wraps.

132 ***Impact testing procedure***

133 Drop-weight impact tests were conducted by dropping a weight from a certain height onto the

134 top of the cylinders using the impact test apparatus, as shown in Fig. 3. The weight was made

135 of a solid steel cylinder, weighing 97.5 kg. It is worth mentioning that the shape of the

136 impactor plays an important role to the impact force and the impact contact thus it was

137 designed to have a smooth flat bottom with a radius $r = 50$ mm. A plastic guiding tube was

138 utilized to ensure the impactor falling vertically to the targets. A load cell was placed at the

139 bottom of the specimens to measure the impact force. A high-speed camera which was set to

140 capture 50400 frames per second was used to monitor the failure processes. This frame rate

141 was set after a few trials with lower frame rate which was not fast enough to capture the very

142 short impact events (about 1 millisecond, ms). The data acquisition system controlled by a

143 computer was used to record signals from the load cell and strain gauges. The data acquisition

144 system recorded data at a sampling rate of 1 MHz.

Figure 3. Drop-weight test apparatus

Experimental results of dynamic tests

Failure modes and crack patterns

Grease was applied to both ends of the tested specimens to eliminate the end friction effect. The progressive failure of the tested specimens was monitored by the high-speed camera. The failure modes are divided into two different types including splitting failure of unconfined concrete and FRP fracture of the confined specimens. The splitting failure mode of the unconfined concrete specimens indicates that friction at the specimens' ends was negligible as shown in Fig. 4. At a very early stage of the impact event (about 0.04 ms), small cracks were observed at the impact end after the projectile in contact with the Specimen R1.5. Afterward, the splitting crack initiated at about 0.12 ms. This major vertical crack propagated from the impact end to the base of the specimen within 0.42 ms (from 0.12 ms to 0.54 ms after impact). This splitting crack and crushing failure at the impact end dominated the failure mode of unconfined concrete. It is worth mentioning that the higher impact velocity was, the earlier the vertical cracks started propagation. However, major vertical cracks in the all three

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Figure 4. Progressive failure of unconfined concrete

165 Meanwhile, rupture of the FRP jacket governed the failure of the FRP-confined specimens.
166 As previously mentioned, the position of failure is difficult to predict and three types of
167 failure were observed in the tests, which included failure from the top, failure from the base,
168 or failure at the middle. For the case of failure from the top, considering Specimen G2-3 as an
169 example, a visible crack initiated at the impact end at about 0.62 ms and then propagated
170 downward as shown in Fig. 5. It should be noted that failure of FRP is associated with a
171 change in the fiber color as seen in the high-speed images. The visible crack stopped
172 propagating to the bottom of the specimen but developed in the hoop direction at about the
173 quarter top of the specimen. The FRP jacket was then torn off leading to a complete collapse
174 of the specimen. In this study, all the fully confined specimens that failed at the first drop
175 exhibited a consistent progressive failure at which cracks propagated from the impact end.

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Figure 5. Progressive failure of confined concrete

Specimens G3-3.5(2) and G3-3(2) are examples of failure at the mid height or the base as shown in Fig. 6. Specimen G3-3.5(2) showed a horizontal crack at the mid height at 0.72 ms. This crack propagated horizontally until about 1.4 ms when fiber at this region ruptured. Meanwhile, Specimen G3-3(2) showed cracks propagating from the base upwards to the mid height. At 0.5 ms, an inclined crack appeared at the bottom one third of the specimen and this crack developed longer until 0.56 ms. A second inclined crack was observed at mid height of the specimen at 0.64 ms. It is noted that this specimen did not fail by this drop. Specimens which did not fail at the first drop might have failure occurring at the mid height or base region of the specimens. The lateral confinement effect may be attributed to this change of the failure mode. When the projectile impacts a specimen, it generates stress waves propagating axially from the top to the bottom of the specimen. If the stress waves are strong enough, they damage the specimen instantly. In such cases, the damage initiates at the top and propagates to the bottom. The compressive stresses in these specimens are not uniform before the damage of the specimen. However, if the stress waves are not strong enough to destroy the specimens immediately, they propagate forth and back in the specimen and make the compressive stresses approximately uniform after a few reflections [15]. The evolution of the stress, presented in the previous section, is very complicated and the maximum value is dependent on the elastic wave velocity, the plastic wave velocity, the impact velocity, and the yield strength of the material. Therefore, if the stress wave propagates a few times before the specimen damages, it is hard to predict the failure initiation. This observation can also be used to explain the different failure modes presented in previous studies [4-6]. In general, if stress waves generated from high impact energy are intensive enough to destroy the concrete matrix and the FRP jacket upon impact, the failure at the top of specimens is usually observed as reported in the studies [4, 6]. This failure mode usually occurs in split Hopkinson pressure bar and gas gun tests when a specimen fails without uniform stresses along its length [16].

Figure 6. Progressive failure from mid height and base of confined concrete specimens

Once the compressive stresses are uniform, the failure mode of the specimens is expected to be similar to that under static tests. In static tests, the friction force at the ends of concrete cylinders confines the specimen's ends. The cylinders usually fail at the weaker region which is at the mid height of the specimens. Therefore, when the compressive stresses in a concrete cylinder are approximately uniform, it likely fails at the mid height because of the minimum end friction confinement in this region. Otherwise, when the stress waves are not strong enough to damage a specimen at the first drop and multiple stress wave reflections make the stress state uniform in the specimen, the specimen is likely to fail at mid height of the specimen because of the minimum end friction confinement. Similar observations were also reported by Uddin et al. [5].

Different confining pressures may result in different failure surfaces of the tested specimens. This is presented and discussed here. Specimens with weak confining pressure exhibited an inclined failure surface as Specimen C1-2.5 shown in Fig. 7a. In cases of specimens being heavily confined by more ductile FRP (GFRP), the fiber did not completely rupture although the concrete core completely failed. Fig. 7b shows the complete failure of the concrete core of Specimen G3-3.5 after removing GFRP jacket. It is worth noting that although specimens C2 have higher confining pressure than specimens G3 in static analysis, an inclined failure surface was still observed on specimens C2 because of the FRP rupture (see Fig. 7c). This performance is because CFRP is more brittle than GFRP so that it could not effectively confine the concrete core under impact loads. These observations indicate the relatively ductile GFRP can better enhance the impact resistant capacity of confined concrete than the more brittle CFRP.

Figure 7. Failure surface

Impact resistance

The impact resistance is quantified by the impulse of the impact force. The impact force in this study was measured by a load cell placed at the base of the specimens. When a projectile impacts a specimen, it accelerates the specimen and thus generates inertia force. The contact force at the impact end is equal to the contact force at the base plus sum of the inertia forces. Rieder and Mindess [17] showed the difference between the contact forces at the impact end

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3 235 and base. It is, therefore, recommended that the impact force should be measured at the base
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5 236 of specimens and this suggestion has been adopted by previous studies [18, 19].
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7 237 The time histories of the impact force of the unconfined and confined concrete specimens are
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9 238 presented in Figs. 8 and 9, respectively. The peak impact force of unconfined concrete
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11 239 increased significantly as compared to that under static load as shown in Fig. 8. The impact
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17 242 higher impact energy associated with higher impact force and longer duration when they were
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19 243 tested under the same impact energy. For example, Specimen C2-2.5(2) resisted the
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21 244 maximum impact force of 952 kN with duration of 1.8 ms while the maximum impact force
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23 245 and impact duration of Specimen C1-2.5(2) were 746 kN and 1.36 ms. This observation
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25 246 shows that an increase in FRP layers may significantly increase the impact resistance and the
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27 247 peak impact force of confined concrete. In addition, the maximum impact forces of
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29 248 Specimens G2-2, G2-2.5, and G2-3 were 772 kN, 782 kN, and 1024 kN, respectively. These
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31 249 results again confirm that higher impact energy yields higher impact force. On the other hand,
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33 250 Fig. 9 demonstrates higher efficiency of GFRP than CFRP under impact loads. The estimation
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35 251 of the maximum static capacities of Specimens C1 and G2 is 553 kN and 521 kN,
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37 252 respectively. Interestingly, the maximum impact forces of Specimens C1-2.5(2) and G2-3
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39 253 were 746 kN and 1024 kN, respectively. As presented in the study by Pham and Hao [20],
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41 254 GFRP showed more ductile behavior than CFRP since the first one ruptured at a higher strain
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43 255 than that of the later one. The summary of FRP strain and FRP efficiency factor is presented
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45 256 in Table 2. It is noted that the strain rate is the maximum values of the specimens and the FRP
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47 257 efficiency factor is the ratio of the actual rupture strain and the ultimate strain of FRP from
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Figure 8. Impact force time histories of unconfined concrete

Figure 9. Impact force time histories of confined concrete

Table 2. FRP efficiency factor and strain rate

Specimen	Max FRP strain (%)	FRP efficiency factor	Strain rate (1/s)		
			SG 1	SG 2	SG 3
C1_2.5(2)	0.37	0.21	15.42	#	#
C1_1.5	-	*	#	7.03	10.71
C2_2.5(2)	-	*	#	3.98	#
C2_2.5	0.24	0.14	2.76	6.49	1.38
G1_2	1.30	0.66	19.12	25.00	8.00
G2_2(3)	0.70	*	9.72	#	#
G2_2.5	0.70	*	13.21	7.33	#
G2_3	-	*	#	13.68	43.33
G3_2.5	0.98	0.50	5.24	5.31	#
G3_2.5(2)	0.27	*	3.60	8.35	#
G3_2.5(3)	0.26	*	2.32	5.28	#
G3_2.5(4)	0.38	*	3.73	5.17	#
G3_3	1.00	*	16.13	8.21	#
G3_3(2)	-	*	#	4.90	#
G3_3(3)	-	*	#	4.65	#
G3_3.5	1.03	0.52	9.90	6.98	#
G3_3.5(2)	0.25	*	4.03	4.24	#

strain gauges failed

* FRP efficiency factor is only calculated from strain gauges capturing rupture

not applicable

Figs. 8-9 show that the duration of the impact force was about 1 ms. The compressive strength

of concrete was 46 MPa which had Young's modulus of 25.4 GPa. From Equation 1, the

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3 265 stress wave velocity can be estimated as 3563 m/s so that it takes about 0.06 ms to travel 200
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5 266 mm. During the period of 1 ms, elastic stress wave can travel about 9 times forward and back
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7 267 along the specimen. If the secant modulus of concrete at high stress can be assumed 10% of
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9 268 its Young's modulus, the plastic wave velocity is 1126 m/s. The plastic stress wave takes 0.18
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11 269 ms to travel 200 mm so that it can propagate almost 6 times forward and back within 1 ms.
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13 270 When the impact forces reach the peaks (at about 0.5 ms), the elastic stress wave can travel
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15 271 about 4 times while the plastic stress wave does 3 times. This number is at the border line of
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17 272 achieving a uniform stress condition so that some specimens may have an equilibrium
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19 273 condition and failed at the mid height while other specimens could not achieve this condition
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21 274 and damaged at the impact end or the base.
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25 275 The impact resistance of these specimens is presented in Fig. 10 and Table 1. The
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27 276 experimental results have shown that the impulse of the impact force had a slight change if
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29 277 higher impact energy was applied to the specimens. Wrapping unconfined concrete with FRP
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31 278 can significantly improve the impulse of the impact force. Especially in the case of GFRP
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33 279 confined concrete, the impulse increased from about 200 kN.ms up to about 1100 kN.ms. The
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35 280 results have proven that GFRP confined concrete can resist much higher impact energy as
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37 281 compared to that of the CFRP confined concrete. Moreover, Fig. 11 shows the impulse of the
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39 282 impact force of Group G3 under multiple impacts. The impulses of these specimens were
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41 283 almost unchanged under multiple impacts if the same impact velocities are maintained.
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43 284 However, the impulse of the last impact at the same velocity can be either the same or
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45 285 significantly lower than previous impacts owing to severe specimen damage. For examples,
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47 286 Specimens G3-2.5 failed at the fourth impact with the velocity of 7 m/s and the impulse of
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49 287 these impacts was about 1050 kN.ms. It means that Specimen did not have severe damage
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51 288 after the third impact. On the other hand, the impulse of the last impact of Specimens G3-3
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53 289 and G3-3.5 was significantly lower than those of the previous impacts.
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Figure 10. Impulse of the impact forces

Figure 11. Impulse of the specimens in Group G3

Numerical simulation and results

Modelling

To further examine the impact resistance of the above specimens, a numerical model is developed by using LS-DYNA, which has been proven yielding reliable numerical prediction of structural responses to impact loads [21-27]. 8-nodes constant stress solid element with 1-point quadrature integration is employed to model the concrete (138124 solid elements in total). The mesh convergence test is conducted by halving the mesh size. The results show that the numerical simulation converges when the mesh size is 4 mm. Further reducing the element size only has insignificant influence on the numerical results but increases the computing time and the risk of the computer memory overflow. Thus, 4 mm mesh is used in this numerical model. The penalty contact algorithm *Contact_Automatic_Surface_to_Surface is used to simulate the contact between concrete, the load cell, and the projectile.

LS_Dyna provides a number of constitutive models (e.g. MAT 16, 25, 72, 84/85, and 159) to represent the concrete behavior under impact and blast loads. Some of these available models have been adopted to simulate concrete against impact loads and their verification has been confirmed by previous studies [21, 28-30]. In this study, the material model

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3 307 *Mat_Concrete_Damage_Rel3 (MAT_72_REL3) is used to simulate the concrete behavior.
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5 308 This material model takes into account the strain-rate effect, plasticity, and damage softening
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7 309 after failure. Previous studies have shown that this material model yielded reliable numerical
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9 310 predictions of concrete structures against impact and blast loads [30-33]. The material model
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11 311 is based on the concrete compressive strength of 46 MPa. In addition, the card
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13 312 *Mat_Add_Erosion is used to eliminate concrete elements which are no longer contributing to
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15 313 resisting the impact load. This study uses the maximum principal strain at failure as a criterion
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17 314 to delete the failed concrete elements while no erosion criteria is applied for other materials.
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19 315 The value of 0.5 is used after trials to yield fairly good agreement with the experimental
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21 316 results. The detailed 3D modelling is presented in Fig. 12 and material models are presented
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319 Figure 12. Numerical modelling of the impact tests

320 Table 3. Parameters for material models

Material	Model in LS-DYNA	Input parameter	Value
Concrete	Mat_	Density	2160 kg/m ³
	Concrete_Damage_Rel	Compressive strength	46 MPa
	3 (MAT_72R3)	Poisson's ratio	0.2
FRP jacket	Mat_Enhanced_Comp[	Weight per sqm	340 g/m ²

	osite_Damge (MAT_054/055)	Modulus of elasticity	89 GPa
		Poisson's ratio	0.01
		Shear modulus	22 GPa
		Longitudinal tensile strength	1548 MPa
		Tensile failure strain	0.017
Steel plates, rollers, load cell and projectile	Mat_Elastic (MAT_1)	Density	7800 kg/m ³
		Modulus of elasticity	200 GPa
		Poisson's ratio	0.3

The strength increment is defined by the dynamic increase factor (DIF) at a given strain rate. Hao et al. [34] conducted a review of the concrete dynamic material properties. It was concluded that DIF was a material property although those obtained directly from high-speed impact tests contain contributions from other sources such as inertia and friction confinements. This statement agrees with results from previous studies [35-37]. Many empirical models have been proposed in the literature to quantify the strain rate effect of concrete and steel. The use of DIF to model the strain rate effect of concrete and steel materials associated with high strain rate was proven to yield reliable predictions [32, 33, 38]. Hao and Hao [38] have proposed a new DIF relation for concrete material, in which the contribution to strength increment from lateral inertia confinement associated with the high-speed impact tests has been removed. The accuracy of the proposed model had been experimentally verified by the split Hopkinson pressure bar tests [37]. The DIF relations for the concrete material used in this study are presented in the following expressions:

$$CDIF = \frac{f_{cd}}{f_{cs}} = 0.0419(\log \dot{\varepsilon}_d) + 1.2165 \quad \text{for } \dot{\varepsilon}_d \leq 30/s \quad (3)$$

$$CDIF = \frac{f_{cd}}{f_{cs}} = 0.8988(\log \dot{\varepsilon}_d)^2 - 2.8255(\log \dot{\varepsilon}_d) + 3.4907 \quad \text{for } \dot{\varepsilon}_d > 30/s \quad (4)$$

$$TDIF = \frac{f_{td}}{f_{ts}} = 0.26(\log \dot{\varepsilon}_d) + 2.06 \quad \text{for } \dot{\varepsilon}_d \leq 1/s \quad (5)$$

337
$$TDIF = \frac{f_{td}}{f_{ts}} = 2(\log \dot{\epsilon}_d) + 2.06 \text{ for } 1/s < \dot{\epsilon}_d \leq 2/s \tag{6}$$

338
$$TDIF = \frac{f_{td}}{f_{ts}} = 1.4431(\log \dot{\epsilon}_d) + 2.2276 \text{ for } 2/s < \dot{\epsilon}_d \leq 150/s \tag{7}$$

339 where CDIF and TDIF are the DIF for concrete in compression and tension, respectively; f_{cd}
340 and f_{cs} are the compressive strength of concrete at high strain rate and quasi-static loads,
341 respectively; f_{td} and f_{ts} are the tensile strength of concrete at high strain rate and quasi-static
342 loads, respectively; and $\dot{\epsilon}_d$ is the strain rate. The relationship between the tensile and
343 compressive DIF of steel and the strain rate is defined by the following equations [39]:

344
$$DIF = \left(\frac{\dot{\epsilon}}{10^{-4}} \right)^\alpha \tag{8}$$

345
$$\alpha = 0.074 - \frac{0.04 f_y}{414} \tag{9}$$

346 where f_y is the yield strength of steel in MPa and $\dot{\epsilon}$ is the strain rate of steel.

347 **Model verification of unconfined concrete**

348 The numerical simulation results are verified against the experimental results of specimen R-
349 1.5 as shown in Fig. 13. The time history of the impact force from the experimental test agrees
350 with the simulation results. The peak impact forces of the experiment and simulation are 496
351 kN and 526 kN, respectively. The corresponding duration of the impact forces is 0.81 ms and
352 0.67 ms, respectively. In addition to examining the stress wave velocity in the specimens, Fig.
353 14 shows the impact forces at the impact end and the base end. Based on the theoretical
354 consideration as presented in Fig. 1, the time lag of 0.06 ms between A and A' in Fig. 14 is
355 consistent with the stress wave velocity for the elastic wave to travel from the impact end and
356 the base before the arrival of plastic waves. For the specimen height of 200 mm, the estimated

elastic stress wave velocity from the time lag is 3333 m/s, which agrees very well with the theoretical value of 3563 m/s. In addition, the time lag of 0.18 ms between B and B' is consistent with the stress wave velocity of the plastic wave. As a result, the estimated plastic stress wave velocity is 1111 m/s as compared to the theoretical value of 1126 m/s. In general, the numerical simulation results agree well with the experimental and theoretical values.

Figure 13. Time histories of the impact force (experiment vs simulation)

Figure 14. Impact force time histories on top and bottom face of the specimen

Numerical simulation of FRP-confined concrete

Another numerical model is built to simulate the impact behavior of FRP-confined concrete and the numerical results are verified against the experimental results of Specimen C1-1.5. All the material models, element types, contact algorithms, and boundary conditions of this specimen are similar to those presented for the plain concrete material except the FRP jacket. The FRP jacket is modeled by using shell element. The material model *Mat_Enhanced_Composite_Damage (MAT_054/055) is used to model the FRP jacket. The bonding between FRP jacket and concrete is modeled by using the contact algorithm *Contact_Tiebreak_Surface_To_Surface, which depends on tensile failure stress (NFLS) and shear failure stress (SFLS) of the adhesive. The bonding failure at the contact occurs if the following condition is reached

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$$\left(\frac{|\sigma_n|}{NFLS}\right)^2 + \left(\frac{|\sigma_s|}{SFLS}\right)^2 \geq 1 \tag{10}$$

375 where σ_n and σ_s are the tensile stress and shear stress, respectively. The bonding strength is
376 based on the epoxy resin properties as presented in Table 4 [9]. The impact time histories of
377 FRP-confined concrete (Specimen C1-1.5) are shown in Fig. 15, indicating the numerical
378 results agree well with the testing data. The strain of FRP from the simulation also agrees
379 reasonably well with the experimental results as presented in Fig. 16. The progressive failure
380 of the specimen is investigated and presented in Fig. 17. In this figure, the contour of the
381 plastic strain is presented, indicating the FRP jacket initiates the failure at 0.6 ms at the impact
382 end as observed in the experiment. The FRP jacket is able to confine and hold the concrete
383 core inside so that the concrete core only fails if the FRP jacket has failed. The concrete
384 crushing at the impact end and the FRP rupture at the impact end are observed in both the
385 experimental testing and the numerical model. By comparing the numerical results and testing
386 results, it is concluded that the numerical model yields reasonably accurate predictions of
387 FRP-confined concrete subjected to axial impact. The validated numerical model of FRP-
388 confined concrete can be utilized to conduct intensive numerical simulations to predict the
389 response of confined concrete with different dimensions and various FRP strengthening
390 schemes subjected to different levels of axial impact loads, which is not presented herein.

391 Table 4. Mechanical properties of epoxy resin

Properties	Value
Tensile strength	54.1 MPa
Tensile elongation	3.4%
Tensile modulus	2813 MPa
Flexural strength	97.3 MPa
Flexural modulus	3178 MPa

Figure 15. Impact force time histories of FRP-confined concrete (experiment vs simulation)

Figure 16. FRP strain (experiment vs simulation)

Figure 17. Progressive failure from numerical simulation

Conclusions

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This study examines the failure and impact resistance of plain and confined concrete under impact loads. The experimental results have shown that FRP-confined concrete provides outstanding impact resistance ability as compared to that of plain concrete. The findings in this study can be summarized as follows:

1. FRP can be used to strengthen concrete to achieve significant improvement of impact resistance and ductility. The impact resistance of the confined specimens increased up to 5 times of that of the plain concrete.
2. GFRP provided much higher impact resistance and ductility as compared to CFRP. GFRP is thus highly recommended to be used under impact loads.
3. The progressive failure of specimens under impact loads is complicated and the initiation of the failure was explained.
4. Elastic and plastic stress wave propagations in concrete have been numerically examined. Plastic stress wave velocity should be considered in checking the stress equilibrium condition in the specimens.

Finally, axial impact resistance of plain concrete and confined concrete is a complicated problem. Further studies are deemed necessary to better understand this sophisticated mechanism.

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