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Residual resistance of impact-damaged reinforced concrete beams

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The behaviour of reinforced concrete (RC) beams under single or repetitive drop-weight impact loading has come under increasing attention within the engineering community in the past decades. However, until now, little effort has been sought towards examining the residual resistance of RC beams after impact damage. To contribute towards a better understanding in this area, both experimental and numerical investigations were carried out. The beams were first tested under drop-weight impact loading. Subsequently, quasi-static bending tests were conducted on the same specimens to obtain the residual behaviour. Thereafter, one beam from each series without any prior damage was tested under monotonic static loading to compare its behaviour with impact-damaged specimens. Furthermore, to investigate the structural response in detail, a numerical procedure was developed in an explicit finite-element program. Upon successful validation of the numerical results with the experimental outcomes, numerical case studies were carried out to quantify the variation of residual resistance index in terms of various parameters.

Notation

a/d	shear span to effective depth ratio
D	damage index
F_c	hardening cap
F_f	shear failure surface
f'_c	compressive strength of concrete
g	gravitational acceleration
h	drop height of impactor
J_1	first invariant of stress tensor
J'_2	second invariant of deviatoric stress tensor
J'_3	third invariant of deviatoric stress tensor
m_b	mass of beam
m_i	mass of impactor
P	pressure
$\mathfrak{R}$	Rubin three-invariant reduction factor
$\mathbf{S}_{ij}, \mathbf{S}_{jk}, \mathbf{S}_{ki}$	deviatoric stress tensor
V	velocity
α	mass ratio
κ	cap hardening parameter
ρ_L	longitudinal reinforcement ratio
ρ_T	transverse reinforcement ratio
$\sigma_1, \sigma_2, \sigma_3$	principal stresses

inevitable that they will be exposed to various types of impact loading during their service life. Examples include

- transportation structures (bridge piers, guard rails, traffic signal posts, electric poles and so on) subjected to vehicle crash impact
- falling rocks on rock sheds in mountainous regions
- falling heavy loads on industrial facilities due to accidents
- marine and offshore structures exposed to ship or ice impact or subjected to tornado- or tsunami-borne debris impact
- columns in multi-storey car parks or bridge pier strikes by moving vehicles
- protective structures subjected to projectile or aircraft impact
- structures sustaining shock and impact loads during explosions or earthquakes.

Progressive collapse could be triggered in these systems due to impact loading thrusts in the structural elements and therefore it is of utmost importance to evaluate the residual resistance of the structural components of an impact-damaged structure. Detailed understanding of the residual properties of structural components would be helpful in determining the integrity and stability of impact-damaged structures during search and rescue operations. Moreover, to ensure that damaged structures will not fail catastrophically during their service life and will maintain structural efficiency, it is essential to study the residual properties of the structures after impact. This knowledge can be further

Introduction

Reinforced concrete (RC) structures are the main structural choice in new infrastructure projects across the world. Due to the wide distribution and prevalence of such structures, it is almost

extended to an evaluation of proper strengthening or rehabilitation schemes.

Assessments of the behaviour of RC beams under drop-weight impacts (Abbas *et al.*, 2010; Bentur *et al.*, 1986; Bhatti *et al.*, 2009; Chen and May, 2009; Fujikake *et al.*, 2009; Kishi *et al.*, 2001, 2002) and high loading rates (Adhikary *et al.*, 2012, 2013, 2014; Kulkarni and Shah, 1998) are well documented in the literature and research on the residual strength of blast-damaged RC columns (Bao and Li, 2010; Park *et al.*, 2014; Wu *et al.*, 2011) is prolific. However, evaluations of the residual response of impact-damaged beams are lacking. The quasi-static response of undamaged and impact-damaged specimens was thus investigated in this work.

The beams were first subjected to free-falling impact loadings from various drop heights. Subsequently, the pre-damaged specimens were tested quasi-statically to determine their load–mid-span deflection responses. Finally, one beam from each group without any prior damage was tested under monotonic static loading to evaluate its original load–mid-span deflection behaviour. Comparative load–mid-span deflection profiles of undamaged and impact-damaged beams were plotted and values of the residual resistance index (RRI) and residual stiffness index (RSI) were computed. The RRI is defined as the ratio of the ultimate resistance of undamaged and impact-damaged specimens. Similarly, the RSI is the normalisation of the secant stiffness. In this work, the secant stiffness was computed by joining the origin to the peak load in the load–mid-span deformation curves of beams suffering brittle-type failure; the yield resistance was taken as the reference point to calculate the same for specimens having ductility.

To gain knowledge beyond the range of parameters investigated experimentally, a methodology was developed to numerically simulate the drop-weight impact tests and post-impact quasi-static

residual resistance tests of damaged beams. To simulate the impact response of the beams, the explicit finite-element (FE) code LS-Dyna (LSTC, 2007) was used by considering the same boundary conditions and the exact shape of the impactor as used during the experiments. Then, a restart analysis was performed by replacing the impactor with a plate to perform quasi-static simulation. The resulting deformation and damage of the beam from the impact stages were utilised to carry out a residual resistance simulation. Results from the explicit FE simulations were compared with the impact and post-impact quasi-static residual resistance tests results. The simulation results seem to match the experimental results with reasonable accuracy, and hence this numerical procedure could be useful in providing some insight into the behaviour of impact-damaged specimens before performing actual laboratory tests. Moreover, newly designed structural elements could be analysed prior to their construction on site, and existing impact-damaged structural elements could be retrofitted effectively by calculating their residual resistance following the numerical scheme. Finally, it is essential to stress that impact performance assessment of RC beams is not the objective of this paper.

Experimental investigation

Test specimens

The reinforcement detailing and overall geometry of the beams is shown in Figure 1. In total, 30 specimens were tested under free-falling drop-weight impact loading and three-point quasi-static bending tests were conducted on the same specimens to obtain their residual resistance and stiffness. Thereafter, six beams without any prior damage were tested under static loading to compare their responses with the impact-damaged specimens.

Specimens denoted SR correspond to a singly reinforced beam; this notation is followed by the shear span to effective depth ratio

Figure 1. Detailing of RC beams (dimensions in mm)

($a/d = 3.8$ or 5.7) and then the longitudinal tensile reinforcement ratio ($\rho_L = 0.8$ or 1.6). Specimens denoted DR refer to a doubly reinforced beam, followed by the shear span to effective depth ratio ($a/d = 3.3, 3.8$ or 5.7), the longitudinal tensile and compressive reinforcement ratio ($\rho_L = 0.8, 1.6$ or 2.4) and then the transverse reinforcement ratio ($\rho_T = 0.11, 0.12, 0.15, 0.20$ or 0.56).

To take into account RC beams with a limited amount of transverse reinforcement, in some specimens the intervals of transverse reinforcements were kept large (i.e. greater than the effective depth of the beam). The influence of the shear span to effective depth ratio, longitudinal reinforcement ratio, shear reinforcement ratio and drop height on the residual properties was investigated. The target concrete compressive strength at 28 d was 40 MPa and the maximum aggregate size was 10 mm for all specimens. The yield strengths of T22, T13 and R6 reinforcing bars (T represents deformed bars and R denotes plain bars; the corresponding digit represents the diameter of bars in millimetres) were 520, 520 and 310 MPa, respectively. In this work, T22 and T13 were used as longitudinal reinforcement and R6 was used as the transverse reinforcement.

All specimens were cast at the same time using ready-mixed concrete from the same batch and hence all specimens were of identical strength. The 28 d compressive strengths of three concrete cylinders were 39.2, 39.6 and 41.0 MPa, respectively. The static design parameters for the beams are listed in Table 1; the static flexural and shear resistances (i.e. nominal values) were calculated according to ACI 318-08 (ACI, 2008). All specimens were well instrumented to capture loads, displacements and steel rebar strains during impact testing. A dynamic load cell of capacity 980 kN and measuring frequency of 5 kHz was attached to the impactor to measure the impact force when it struck the beam. Also, 2 mm strain gauges were installed in the mid-span of the longitudinal tensile reinforcements and in the mid-point of the two legs of the transverse reinforcements to measure the strain history. The mid-span deflection of the beams was measured with laser-type variable displacement transducers (LVDTs) having a measuring range of 80 mm. Data from the sensors were

collected by a digital data acquisition system which had a sampling rate of 100 kHz for the impact loading tests. Data for the post-impact quasi-static tests were captured with a sampling rate of 100 Hz. Cracking patterns and failure modes of the beams were captured using digital photography and a high-speed video camera.

Test programme

A total of nine types of specimens were used, categorised in terms of their shear span to effective depth ratio (a/d), longitudinal reinforcement ratio and shear reinforcement ratio. Details of test programme are given in Table 2. A drop hammer of mass 300 kg was dropped freely from different heights onto the top surface of the RC beam at mid-span. The mass ratios (α) (i.e. ratio of beam mass m_b to impactor mass m_i) for specimens with $a/d = 3.3, 3.8$ and 5.7 were 0.42, 0.49 and 0.26, respectively. The striking head of the impactor had a hemispherical tip with a radius of 90 mm. Specially designed support devices were mounted over the clear span of the beam, allowing it to rotate freely while preventing uplift of the beam under impact loading. The impact force was measured by a load cell firmly attached to the impactor. After completion of the impact test, the damaged specimens were placed in a three-point bending test set-up to apply quasi-static loading in order to capture their residual resistance against mid-span deformation curves. The test set-up is shown in Figure 2.

Summary of experimental results

Load-mid-span displacement curves

Quasi-static load against mid-span characteristics of undamaged and impact-damaged specimens (i.e. damaged by impacts from different drop heights) are shown in Figure 3. Figure 3(a) shows the curves of RC beams with $a/d = 3.3$. It was observed that residual resistance of DR3-3-2.4 (without transverse reinforcement) almost reached the same resistance as undamaged specimens with the same stiffness when the impactor struck the specimen from a drop height of 0.15 m. However, notable degradations in both resistance and stiffness were observed for

Specimen	a/d	ρ_L : %	ρ_T : %	Bending resistance: kN	Shear resistance: kN	Shear to bending resistance ratio
DR3-3-2.4	3.3	2.4	—	203.70	66.20	0.33
DR3-3-2.4-0.12	3.3	2.4	0.12	203.70	87.10	0.43
DR3-3-2.4-0.56	3.3	2.4	0.56	203.70	170.40	0.84
SR3-8-0.8	3.8	0.8	—	70.45	67.83	0.96
DR3-8-0.8-0.11	3.8	0.8	0.11	67.83	93.32	1.38
DR3-8-0.8-0.15	3.8	0.8	0.15	67.83	102.30	1.51
SR5-7-1.6	5.7	1.6	—	42.40	35.32	0.83
DR5-7-1.6-0.15	5.7	1.6	0.15	42.40	50.52	1.19
DR5-7-1.6-0.20	5.7	1.6	0.20	42.40	56.40	1.33

Table 1. Details of specimens and static design parameters

Specimen	Static test of undamaged beams	Impact test drop height: m	Impact energy: kJ	Residual quasi-static tests
DR3-3-2-4	FEM determined	0.15, 0.30, 0.60, 1.20	0.44, 0.88, 1.77, 3.53	✓
DR3-3-2-4-0.12	FEM determined	0.30, 0.60, 0.90, 1.20	0.88, 1.77, 2.65, 3.53	✓
DR3-3-2-4-0.56	FEM determined	0.60, 0.90, 1.20, 1.6	1.77, 2.65, 3.53, 4.71	✓
SR3-8-0.8	Tested	0.30, 0.60, 0.90	0.88, 1.77, 2.65	✓
DR3-8-0.8-0.11	Tested	0.60, 0.90, 1.20	1.77, 2.65, 3.53	✓
DR3-8-0.8-0.15	Tested	0.60, 0.90, 1.20	1.77, 2.65, 3.53	✓
SR5-7-1.6	Tested	0.30, 0.45, 0.60	0.88, 1.32, 1.77	✓
DR5-7-1.6-0.15	Tested	0.30, 0.45, 0.60	0.88, 1.32, 1.77	✓
DR5-7-1.6-0.20	Tested	0.30, 0.45, 0.60	0.88, 1.32, 1.77	✓

Table 2. Test programme

Figure 2. Test set-up: (a) drop-weight impact test;
(b) post-impact residual resistance test

Figure 3. Quasi-static load against mid-span deflection of undamaged and impact-damaged beams for various drop heights: (a) $a/d = 3.3$; (b) $a/d = 3.8$; (c) $a/d = 5.7$

0.30 m and 0.60 m drop heights. Similarly, for DR3-3-2-4-0-12, drop heights of 0.60–1.20 m caused significant strength and stiffness reductions. One interesting phenomenon for this specimen was that the stiffness in the case of the 0.90 m drop height was smaller than for a 1.20 m drop height. This may be because more severe diagonal shear cracks (i.e. the number of diagonal cracks) were observed in post-impact residual test for the 0.90 m drop height. For the same series of specimens with a higher amount of transverse reinforcement (i.e. DR3-3-2-4-0-56), the resistance and stiffness degradation was less than that of the other two beams. It is to be noted that the static response of the undamaged beams in this series was not evaluated experimentally, but was determined by FE simulation. For the beams $a/d = 3.8$ and 5.7 (Figures 3(b) and 3(c)), specimens with greater transverse reinforcement experienced less strength degradation than the other specimens.

Residual resistance index (RRI)

The RRI, defined as the ratio of the peak resistance of impact-damaged to undamaged specimens, is shown in Figure 4 for beams under different impact energies. For DR3.3-2-4 (without transverse reinforcement), an increase in input impact energy from 0.44 kJ to 0.88 kJ and 1.77 kJ led to a reduction in ultimate residual resistance (URR) of 4% and 46%, respectively. A transverse reinforcement ratio of around 0.12% means that the specimen can absorb more impact energy and the reduction in URR was reduced compared with the non-reinforced specimens. With a further increase in transverse reinforcement ratio to 0.56%, the URR was reduced from 5 to 24% when the input impact energy was increased from 1.77 kJ to 4.71 kJ. Moreover, not only did the URR of the singly reinforced $a/d = 3.8$ series specimens reduce, but it was also greater than the ultimate resistance of the undamaged specimens. This is due to the change

Figure 4. Residual resistance index (RRI) of RC beams under various impact energies

in the nature of the failure mode (the undamaged specimens failed in shear whereas the damaged beams failed in flexure (i.e. the strain hardening phenomenon was significant in impact-damaged beams)) under the impact energies considered during testing (i.e. 0.88–2.65 kJ). Doubly reinforced beams of the same specimen series showed very similar RRIs at low impact energies

(1.77 kJ and 2.65 kJ); however, specimens with a greater amount of transverse reinforcement performed better for a higher input energy (i.e. 3.53 kJ).

For specimens with $a/d = 5.7$, a systematic trend of RRI degradation was observed; with increasing impact energies, better performance was observed in specimens with more transverse reinforcement. In general, a decreasing trend in RRI was observed with increasing impact energy and this trend was more pronounced in specimens with less transverse reinforcement. In a few cases, the URR of the damaged specimens was greater than that of the undamaged ones; this may be due to the hardening effect of the steel reinforcing bars. The RRI could be used to define the extent of damage in a RC beam after impact loading through a damage index D , defined as

$$1. \quad D = (1 - \text{RRI})$$

with the degrees of damage defined as low ($D = 0.0\text{--}0.2$), medium ($D = 0.2\text{--}0.5$), high ($D = 0.5\text{--}0.8$) and severe ($D = 0.8\text{--}1.0$).

Shi *et al.* (2008) proposed a similar type of damage index for RC columns damaged by blast loads and also mentioned that the definition was solely subjective. Thus, here, the damage term for $D = 0.8\text{--}1.0$ has been changed from ‘collapse’ to ‘severe damage’ as some researchers believe the shear failure mode to be the collapse stage. Hence, the degree of damage of a beam for a particular impact energy can be calculated from Equation 1. For $\text{RRI} > 1$, it could be concluded that the beam did not suffer any substantial damage or that the developed hairline cracks might close as soon as the impact load was removed.

Residual stiffness index (RSI)

The RSI has been defined as the ratio of the secant stiffness of impact-damaged to undamaged specimens. Here, the secant stiffness was computed by joining the origin to the peak load in the load–mid-span deformation curves of beams with brittle failure type (i.e. a sharp fall in load–deformation curves after peak load), whereas the yield resistance was taken as the reference point to calculate the same for specimens having ductility. The variation in RSI of beams under different impact energies is shown in Figure 5.

For DR3-3-2-4 (without transverse reinforcement), when the input impact energy increased from 0.44 kJ to 1.77 kJ, the reduction in stiffness was 0 and 81%, respectively. However, around 70–80% reduction in stiffness was observed for higher impact energies (around 2.65 kJ and 3.53 kJ) for specimens with 0.12% transverse reinforcement. The rate of stiffness reduction for DR3-3-2-4-0-56 was much less drastic with increasing impact energies (i.e. 1.77 kJ to 4.71 kJ) compared with other specimens of the same series. It is thus evident that the transverse reinforcement inhibited a significant degradation in stiffness. Around 20% stiffness reduction was found for SR3-8-0-8 for all input energies

Figure 5. Residual stiffness index of RC beams under various impact energies

considered during the experiment (0.88–2.65 kJ). However, for an impact energy of 2.65 kJ, the stiffness reduction was insignificant for doubly reinforced sections of the same series. However, for $a/d = 5.7$ series specimens, a consistent stiffness degradation was seen for increasing drop heights. For the same impact energy and

for $a/d = 5.7$, the degradation in stiffness was less in the DR specimens than the SR specimens.

Failure patterns

Assessments of crack patterns or failure modes are of paramount importance in both impact and post-impact quasi-static loading stages because they assist in determining the integrity and stability of impact-damaged structures. Figure 6 shows the crack profiles of the beams under impact and post-impact residual resistance test stages. For DR3-3-2-4, impact energies were varied from 0.59 kJ to 4.70 kJ. No visible cracks were observed for an impact energy of 0.59 kJ, but diagonal shear cracks were noted in the residual resistance test. With further increments in impact energy, diagonal cracks were observed and these cracks widened successively in the residual test with much damage in the impact regions. At a drop height of 1.20 m (i.e. impact energy of 4.70 kJ), the beam was so severely damaged that the post-impact residual test could not be performed.

For DR3-3-2-4-0.12, hairline diagonal cracks were seen under low-velocity impacts, whereas much wider cracks were produced by high-velocity impacts. The residual tests revealed further development of diagonal cracks and widening of the existing diagonal cracks with concrete spalling. Impact energies were varied from 2.35 kJ to 6.28 kJ for the drop-weight impact tests on specimen DR3-3-2-4-0.56. This beam showcases the significance of a higher amount of transverse reinforcement by not only absorbing higher impact energies (around 34% higher than the specimen with 0.12% shear reinforcement) but also performing better (i.e. resisting catastrophic failure mode) in the residual resistance test. However, some spalling in the side cover concrete near the impact region was observed under an impact energy of 6.28 kJ.

Flexural failure was observed in all beams of the $a/d = 3.8$ series for all impact energies (1.18–4.70 kJ) considered during the impact tests. A ductile nature (i.e. no change in failure mode) persisted during the residual tests of the damaged beams, but some additional flexural cracks developed or existing flexural cracks became wider. Thus, beams with $a/d = 3.8$ and a lower amount of longitudinal reinforcement (although less shear reinforcement) could withstand the impact and post-impact quasi-static load quite satisfactorily. Similarly, all beams of the $a/d = 5.7$ series failed in flexure under impact (impact energies of 1.18–2.35 kJ) and residual tests.

Finite-element framework

Eight-node solid hexahedron elements with a single integration point and viscous hourglass control were used to represent the concrete, while beam elements (two-node Hughes–Liu beam element formulation with 2×2 Gauss quadrature integration) were used to model the steel reinforcing bars. A mesh size of approximately 15 mm was used to create the solid element of the beam. A numerical convergence study showed that a further decrease in mesh size had very little effect on the accuracy of the

Figure 6. Failure patterns of RC beams under impact and post-impact residual tests: (a) $a/d = 3.3$; (b) $a/d = 3.8$; (c) $a/d = 5.7$

results, while considerably increasing the computational time. The mesh discretisation was established so that the reinforcement nodes coincided with the concrete nodes. The steel reinforcing bars were modelled explicitly using beam elements connected to the concrete mesh nodes. Due to this assumption of complete compatibility of strains between the concrete and steel nodes, they formed a perfect bond. Ultimate dynamic bond at failure is 70–100% higher than that of quasi-static loading conditions for the case of deformed reinforcing bars (Weathersby, 2003). Steel deformation under impact load is limited to a region beneath the point of impact. Moreover, there is not enough time to develop extensive bond slip along the length of the bar during an impact as the peak impact load is being reached within a few milliseconds. Thus, the assumption of perfect bond in between the reinforcement and concrete is quite reasonable. The FE model of the impactor was divided into three sections to reproduce the same used in the experiments. The first section consists of the hemispherical striking head with a 90 mm radius. A solid cylinder and rectangular block were used to model the remaining

parts. The impactor was meshed with a four-node tetrahedron and the mesh size was chosen around 20 mm.

To simulate the actual experimental conditions, the beams were supported on two rigid cylinders made of solid elements. Furthermore, to prevent uplifting of the beam after impact loading, two inverted triangular plates were modelled. Constraints were defined to the support cylinder and triangular plates so that it could rotate about its own longitudinal axis but would not be able to translate. After placing the impactor 1 mm from the top surface of the beam at mid-span, the initial velocity was defined to it in order to save computational time. The initial velocity was calculated from the free-fall formula $V = (2gh)^{1/2}$, in which V is the velocity before impact, h is the drop height of the impactor and g is gravitational acceleration. Moreover, the gravitational acceleration of the impactor was taken into account in this numerical simulation. Global damping was not considered in the impact simulation here as the impact responses lasted only 30 ms. The algorithm Contact-Automatic-Surface-to-Surface in LS-Dyna

(LSTC, 2007) was used to model the contact between the support cylinder, impactor and beam. The contact non-linearity was stabilised by assigning a value of 30 for the viscous damping coefficient. This algorithm automatically generates slave and master surfaces and uses a penalty method where nominal interface springs are used to model interpenetration between elements and surfaces. The corresponding impact force due to the prescribed initial velocity to the impactor was then determined by monitoring the contact forces at the concrete nodes in contact with the impactor. After finishing the impact simulation, the impactor and two triangular plates were removed from the model. Then, a rigid loading plate was placed at the mid-span of the beam to initiate residual resistance simulation through restart analysis. The rigid loading plate was allowed to move only in the vertical direction and displacement-controlled loading was applied to it. Here, in the restart analysis environment, the deformation and damage to the beam resulting from the impact simulation were utilised to carry out the residual resistance simulation. Similarly to the determination of impact force history, residual resistance due to the prescribed displacement was then achieved by monitoring the contact forces at the concrete nodes in contact with the support solid cylinders. Figure 7 shows the three-dimensional (3D) FE model of the beam in impact and post-impact quasi-static loading stages.

A number of material models from the LS-Dyna (LSTC, 2007) material library can be used to represent concrete (e.g. Mat-Soil-

and-Foam, Mat-Pseudo-Tensor, Mat-Geologic-Cap-Model, Mat-Concrete-Damage-Rel3, Mat-CSCM-Concrete, Mat-Winfrith-Concrete, Mat-Brittle-Damage). However, it is well known that the material properties control the accuracy of numerical simulation results. Thus, to predict the residual resistance of a beam after impact damage quite accurately, it is important to choose the material model judiciously. In this research, Mat-CSCM-Concrete (Mat-159), commonly known as the continuous surface cap model, was used to model concrete. This model was primarily developed to simulate the deformation and damage of roadside safety concrete structures (i.e. bridge rails, safety barriers and guard rails) impacted by moving vehicles. This is a cap model with a smooth intersection between the shear yield surface and hardening cap, as shown in Figure 8. The initial damage surface coincides with the yield surface and the rate effects are modelled with viscoplasticity. The yield surface is formulated in terms of three stress invariants – J_1 (the first invariant of the stress tensor), J'_2 (the second invariant of the deviatoric stress tensor) and J'_3 (the third invariant of the deviatoric stress tensor). The invariants are defined in terms of the deviatoric stress tensor $\mathbf{S}_{ij}$, the pressure P and principal stresses σ_1 , σ_2 and σ_3 as follows

$$2. \quad J_1 = \sigma_1 + \sigma_2 + \sigma_3 = 3P$$

$$3. \quad J'_2 = \frac{1}{2} \mathbf{S}_{ij} \mathbf{S}_{ij}$$

$$4. \quad J'_3 = \frac{1}{3} \mathbf{S}_{ij} \mathbf{S}_{jk} \mathbf{S}_{ki}$$

The three-invariant yield function is delineated in terms of these three invariants and the cap hardening parameter κ , as shown by

$$5. \quad f(J_1, J'_2, J'_3, \kappa) = J'_2 - \mathcal{R}^2 F_f^2 F_c$$

Figure 7. FE models of beam: (a) impact loading stage; (b) post-impact residual resistance stage

Figure 8. General shape of the concrete model yield surface in two dimensions (Murray, 2007)

where F_f is the shear failure surface, F_c is the hardening cap and $\mathcal{R}$ is the Rubin three-invariant reduction factor. The cap hardening parameter κ is the value of the pressure invariant at the intersection of cap and shear surfaces. For the shear failure surface, the strength of concrete is modelled by the shear surface in the tensile and low confining pressure regimes. The shear failure surface F_f is defined along the compression meridian as

$$6. \quad F_f(J_1) = \alpha - \lambda \exp^{-\beta J_1} + \theta J_1$$

The values of α , β , λ and θ are selected by fitting the model surface to the strength measurements from triaxial compression tests conducted on plain concrete cylinders. The triaxial compression data are typically plotted as principal stress difference against pressure. The principal stress difference (axial stress minus confining stress) is equal to the square root of $3J_2'$. The strength of the concrete is modelled by the combination of cap and shear surfaces in the low to high confining pressure regimes. The cap is used to model plastic volume change related to pore collapse (although the pores are not explicitly modelled). The initial location of the cap determines the onset of plasticity in isotropic compression and uniaxial strain. The elliptical shape of the cap allows the onset for isotropic compression to be greater than the onset for uniaxial strain, in agreement with shear enhanced compaction data. Without ellipticity, a 'flat' cap would produce identical onsets. The motion of the cap determines the shape (hardening) of the pressure–volumetric strain curves by way of fits with data. Without cap motion, the pressure–volumetric strain curves would be perfectly plastic. Concrete usually exhibits softening (strength reduction) in the tensile and low to moderate compressive regimes and softening behaviour is modelled through a damage formulation. Without the damage formulation, the cap model predicts perfectly plastic behaviour for laboratory tests such as direct pull, unconfined compression, triaxial compression and triaxial extension. This behaviour is not realistic; although a perfectly plastic response is typical of concrete at high confining pressures, it is not representative of concrete at lower confinement and in tension. The damage formulation models both strain softening and modulus reduction. Damage initiates and accumulates when strain-based energy terms exceed the damage threshold. Moreover, the strain rate effects on concrete strength are duly considered in this model.

Figure 9 shows the approximate tensile and compressive dynamic increase factor considered for various strain rates. The default parameter generation option (by providing some key specifications) was used instead of the traditional method where all material parameters are needed. Default parameters are provided for the concrete model based on three input specifications – the unconfined compressive strength of concrete, aggregate size and activating strain rate effects. This option is valid for unconfined compressive strength from 20 MPa to 58 MPa and aggregate sizes from 8 mm to 32 mm. The unconfined compressive strength affects all aspects of the fit, including stiffness, 3D yield strength,

Figure 9. Approximate tensile (solid curve) and compressive (dotted curve) dynamic increase factors for default concrete material model (Murray, 2007)

hardening and damage-based softening, whereas the aggregate size affects only the softening behaviour of the damage formulation. Further details of the concrete model theory can be found elsewhere (Murray, 2007)

The steel reinforcement bars (longitudinal and shear reinforcement) within the beam were modelled as a strain rate sensitive uniaxial elastic–plastic material to account for their strain rate sensitivity as well as stress–strain history dependence. Material model Piecewise-Linear-Plasticity (Mat-024) was employed in this study to incorporate the strain rate effect. Wu *et al.* (2011) vividly depict details of the steel material modelling. The expressions proposed by Malvar (1998) concerning the strain rate effects on yield and ultimate stress of steel reinforcement were employed in this study.

Mat-Rigid (Mat-020) was used from the LS-Dyna (LSTC, 2007) material library to model the impactor, support rollers and triangular plates. Realistic values of Young's modulus and Poisson's ratio of the rigid material should be defined since unrealistic values may contribute to numerical problems in contact. The Young's modulus and Poisson's ratio of the steel material were used for the rigid material in the numerical simulation.

Comparison with test results

The numerical simulation results of RC beams subjected to drop-weight impact loading was first validated with the experimental outcomes. Then, the numerical results of an impact-damaged beam were compared with the test results. Verification of impact loading history under impact and residual resistance against

mid-span deflection profile of impact-damaged specimens and cracks patterns is presented in the following two sections.

Impact loading history and residual resistance against mid-span deflection of RC beams

Two types of tested beams were considered for validation of the numerical results. Comparisons of the impact load history and post-impact residual resistance against mid-span deflection of DR3-8-0-8-0-11 for a drop height of 0.60 m are shown in Figures 10(a) and 10(b), respectively. Similarly, these two responses were validated for specimen DR5-7-1-6-0-20 for 0.45 m drop height (Figures 10(c) and 10(d)). The results from the 3D non-linear FE model match the test results with acceptable accuracy.

Crack profiles of beams after impact and post-impact quasi-static loading

An assessment of crack pattern or the failure mode of a beam under drop-weight impact and post-impact quasi-static loading is of utmost importance. Damage to the beams obtained from

numerical simulation is shown by plotting the fringes of effective plastic strain. These effective plastic strain contours reveal the strain localisation where failure propagates. Figure 11 shows a comparison of damage plots from the experiment and numerical simulation. For brevity, comparative damage plots of only two types of beams are shown, but it can be concluded that the numerical simulation damage plot captures the experimental crack profiles (after impact and residual tests) quite satisfactorily.

Numerical simulation case studies

After verification of the FE model with the experimental results, a parametric investigation was conducted to elucidate the residual resistance of beams after impact damage. Key parameters such as the mass ratio (ratio of beam mass to impactor mass) ($\alpha = 0.26, 0.42$ and 0.49), longitudinal reinforcement ratio ($\rho_L = 0.8\text{--}2.4\%$), transverse reinforcement ratio ($\rho_T = 0.12\text{--}0.67\%$) and concrete compressive strength ($f'_c = 30, 40$ and 50 MPa) were taken into account to study their influence on residual performance under various impact energies ($0.88\text{--}3.43$ kJ). The

Figure 10. Validation of impact force history and post-impact residual resistance against mid-span deflection

Figure 11. Comparison of cracking patterns of beams after impact and residual tests

parameter variation for various case studies is shown in Table 3. The beams used for the numerical case studies had the same geometric properties (span length and cross-sections) as the experimental specimens.

Effect of mass ratio

Figure 12 illustrates the effect of mass ratio α on the RRI of beams under varying impact energies. The mass of the impactor was kept constant (300 kg), but different mass ratios were obtained due to variations in beam span lengths and cross-sections. Considering the same longitudinal reinforcement ratio and two transverse reinforcement ratios, two cases are presented in Figures 12(a) and 12(b). It is evident that residual resistance will be on the higher side for the case of higher α under the same impact energy.

Effect of longitudinal reinforcement ratio

Figure 13 shows the variation of RRI under various impact energies. To understand the influence of longitudinal reinforcement ratio (ρ_L) on RRI, two cases are considered. Under low impact energies, the effect of ρ_L seems to be insignificant. However, for high impact energies, specimens with low ρ_L perform better in terms of achieving residual resistance. Beams with lower amounts of longitudinal reinforcement exhibited only overall flexure failure, whereas flexure as well as local damage at the impact point occurred in beams with a relatively higher amount of longitudinal reinforcement. This local damage initiated the further development of diagonal cracks during residual tests, which successively reduced the residual resistance. In the case of overall flexural failure types, a much more ductile response was

a/d	α	Longitudinal reinforcement ratio, ρ_L : %	Transverse reinforcement ratio, ρ_T : %	Impact energy: kJ	f'_c : MPa
3.3	0.42	1.62, 2.40	0.12, 0.19, 0.38, 0.56	2.35, 3.53, 4.70	30, 40, 50
3.8	0.49	0.80, 1.50	0.11, 0.15, 0.24, 0.35, 0.50	2.35, 3.53, 4.70	30, 40, 50
5.7	0.26	0.90, 1.60	0.15, 0.20, 0.31, 0.47, 0.67	1.17, 1.77, 2.35	30, 40, 50

Table 3. Parameter variations in case studies

Figure 12. Effect of mass ratio α on RRI under various impact energies: (a) $\rho_L = 1.60\%$, $\rho_T = 0.20\%$; (b) $\rho_L = 1.60\%$, $\rho_T = 0.50\%$

observed due to the strain hardening of the longitudinal tensile rebars.

Effect of transverse reinforcement ratio

The effect of transverse reinforcement on RRI under varying amounts of impact energies is presented in Figure 14. It is

Figure 13. Effect of longitudinal reinforcement ratio ρ_L on RRI under various impact energies: (a) $a/d = 3.8$; (b) $a/d = 5.7$

evident that specimens with a higher amount of transverse reinforcement possess higher residual resistance. Under increasing drop heights, the difference in RRI between beams with low and high amounts of shear reinforcement is significant. A higher amount of transverse reinforcement means the beam can resist excessive impact damage and can produce a much higher residual resistance due to confinement to the core concrete and supplementing lateral restraint capacity against the buckling of the longitudinal reinforcement.

Figure 14. Effect of transverse reinforcement ratio ρ_T on RRI under various impact energies: (a) $a/d = 3.3$, $\rho_L = 1.62\%$; (b) $a/d = 3.8$, $\rho_L = 1.50\%$

Effect of concrete compressive strength

Beams of concrete compressive strength of 30, 40 and 50 MPa were used to determine the effect of f'_c on RRI under varying impact energies. Three case studies are shown in Figure 15. Under low impact energy, the effect of f'_c on RRI is quite minimal. However, for increasing impact energies, specimens with high compressive strength perform better (i.e. higher residual resistance) than those of low strength. A beam made from high-strength concrete was beyond the scope of this research.

Conclusions

Based on the experimental and numerical studies conducted in this research, the following conclusions were reached.

- Two indices – residual resistance index (RRI) and residual stiffness index (RSI) – were defined and discussed in detail to

Figure 15. Effect of compressive strength of concrete f'_c on RRI under various impact energies: (a) $a/d = 3.3$, $\rho_L = 1.62\%$, $\rho_T = 0.56\%$; (b) $a/d = 3.8$, $\rho_L = 1.50\%$, $\rho_T = 0.50\%$; (c) $a/d = 5.7$, $\rho_L = 1.60\%$, $\rho_T = 0.67\%$

show their variation under various impact energies. These indices could be used effectively to delineate the extent of damage (i.e. resistance and stiffness degradation) to beams after impact loading.

- A methodology was developed to numerically simulate drop-weight impact tests and post-impact quasi-static residual resistance tests of damaged beams. The FE model was successfully validated with the test results.
- After successful verification of the FE model with the experimental results, a parametric study was carried out to quantify the effect of various parameters on the RRI. The RRI will be on the higher side for the case of a higher mass ratio under the same impact energy. Under high impact energies, specimens with a lower amount of longitudinal reinforcement perform better in terms of achieving residual resistance. Specimens with a relatively higher amount of transverse reinforcement possess higher residual resistance after impact damage. Under increasing impact energies, specimens of high concrete compressive strength perform better (i.e. have higher residual resistance) than beams with low concrete compressive strength.

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