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<td><strong>Author(s)</strong></td>
<td>Adhikary, Satadru Das; Li, Bing; Fujikake, Kazunori</td>
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State-of-the-art review on low-velocity impact response of reinforced concrete beams

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Based on a comprehensive literature review, a state-of-the-art report on the strain rate dependent mechanical properties of materials involved in reinforced concrete (RC) structures and the structural response of RC beams under low-velocity impact is presented. Due to the prevalence of plentiful equations to calculate the dynamic increase factor of concrete strength in compression and tension, future research is needed to reach a general consensus. Two empirical equations were derived based on previous test data, and the applicability of the proposed equations is demonstrated. With the interpretation of previous data in the light of authors’ test results, the issue related to a change in failure mode from flexural failure under static loading to shear failure under impact loading is discussed. Finally, several issues related to the impact response of beams are raised, and the need for future research is identified.

Notation

\( a \) shear span of beam
\( b \) width of beam
\( d \) effective depth of beam
\( E \) input impact energy
\( f_c' \) compressive strength of concrete
\( f_{yl} \) yield strength of longitudinal reinforcements
\( f_{ty} \) yield strength of transverse reinforcements
\( h \) total height of beam
\( L \) total length of beam
\( R_b \) bending resistance of RC beam
\( R_s \) shear resistance of RC beam
\( \delta_{\text{max}} \) maximum midspan deflection

Introduction

It is generally admitted that there is an apparent increase in strength when a concrete-like material is subjected to a high strain rate. The dynamic increase factor (DIF), defined as the ratio of dynamic strength to quasi-static strength, is widely accepted as an important parameter in the measurement of strain rate dependent material behaviour. However, there are several hypotheses concerning the physical mechanism explaining the concrete material response under varying strain rates. One of them states that the observed strain rate effect is due to the presence of water, which may influence the response of concrete under high strain rates (Rossi et al., 1994). It is known that the Stefan effect on free water within concrete can change the cracking patterns under high loading rates. Under quasi-static loading, cracks occur in the cement matrix, and when these cracks encounter coarse aggregates they propagate around the boundaries of the aggregate. On the other hand, in dynamic loading, due to the Stefan effect, cracks would penetrate through the coarse aggregates and this may be the reason behind the strength enhancement.

Another perspective is the lateral inertia force effect, which causes an apparent increase in the DIF of concrete. Li and Meng (2003) reported that an increase in dynamic compressive strength could only be caused by lateral confinement when the strain rate is higher than around 100 s\(^{-1}\). They termed this lateral inertia confinement ‘the pseudo strain rate effect’ and further opined that acceptance of this strain rate effect in design and numerical models may overestimate the dynamic compressive strength of concrete. On the other hand, Zhou and Hao (2008) developed a homogeneous meso-scale model with a strain rate sensitive material model to analyse concrete-like material under high strain rate compression, and the model was found to corroborate reasonably well with test results.

Comparison of the DIF caused by lateral confinement and the DIF obtained from dynamic tests shows that inertial confinement is only one of the two sources that contribute to the DIF.
and this contribution becomes more significant when the strain rate is higher than 1000 s\(^{-1}\). Material strain rate effects cannot be neglected in modelling the concrete material response to high loading rates, especially when the strain rate is less than 200 s\(^{-1}\).

Kim et al. (2010) performed numerical simulations on concrete specimens under high strain rates in compression to see whether the experimentally (i.e. split Hopkinson pressure bar (SHPB) test) obtained strain rate effects were a purely material response or not. It was concluded that a strain rate dependent model should not be employed in numerical simulation as strain rate effects already account for the effects of lateral confinement. Their simulation results also showed the effects of friction on observed strain rate effects when the aspect ratio of specimens was considered. Furthermore, for future experimental and simulation-based research, they suggested examining the specific contribution of the confinement effect. Cotsovos and Pavlovic (2008) raised the question of the usage of strain rate dependent mechanical properties of concrete under high rates of uniaxial tensile loading, as they concluded they were structural rather than material effects. Lu and Li (2011) performed numerical simulations of direct dynamic tensile, dynamic splitting and spalling tests based on a strain rate independent constitutive model of concrete to check whether the tensile DIF is attributed to strain rate effects or structural effects. It was concluded that the strain rate enhancement of tensile strength is purely a material effect rather than a structural effect at macroscopic level. However, at the micro-level, the strain rate dependent tensile strength increment can be attributed to the inertia effects of microcracks.

More recently, Ozbolt et al. (2013) performed numerical studies of compact tension specimens loaded to varying rates. After validating the numerical results with experimental results they commented that, for strain rates lower than approximately 50 s\(^{-1}\), the structural response is controlled by a strain rate dependent constitutive law. However, for higher strain rates, crack branching and a progressive increase in resistance were observed. This was attributed to the effect of structural inertia and not the rate dependent strength of concrete. From a numerical point of view, assuming micro- and meso-scale analysis, the effects of the rate dependency of growing microcracks (e.g. the influence of inertia at microcrack level) and the viscous behaviour of the bulk material between the cracks (e.g. viscosity due to water content) can be accounted for by the constitutive law, whereas the structural inertia effect would be automatically accounted for through dynamic analysis (Ozbolt et al., 2011).

This paper is organised as follows. First, an overview of the strain rate effect on concrete mechanical properties (compression and tension) and steel reinforcement behaviour (yield and ultimate stress) is presented. This is followed by an assessment of the structural response of reinforced concrete (RC) beams subjected to low-velocity impact (<10 m/s) at midspan through the assembly of a database from the literature. Limited data were used by Kishi and Mikami (2012) to propose empirical formulae for designing beams (e.g. only statically flexure-critical) under impact loading. This limitation is overcome by compiling data from different researchers, and successively similar empirical formulae are presented for both statically shear-critical and flexure-critical beams under low-velocity impact loading. Finally, some issues related to the failure modes of beams under impact loading are discussed, and the need for future research is identified.

The strain rate effect on mechanical properties of materials

Since the mechanical properties of plain concrete and steel are strain rate dependent, the behaviour of structural members under impact loading conditions can only be accurately predicted by considering the strain rate dependent properties of materials. Therefore, to evaluate structural performance in terms of resistance and behaviour, the constitutive properties of concrete and steel over a wide range of strain rates are required.

Plain concrete

The DIF is often used to characterise the rate sensitivity behaviour of material and the strain rate effect on compression and tension of concrete is typically reported as the DIF (i.e. ratio of dynamic strength to static strength).

Compression

CEB model code 1990 (CEB, 1993) gives the DIF for compressive strength as

\[ \text{DIF} = \begin{cases} (\dot{\varepsilon}/\dot{\varepsilon}_s)^{0.026\alpha} & \text{for } \dot{\varepsilon} \leq 30 \text{ s}^{-1} \\ \gamma_s(\dot{\varepsilon}/\dot{\varepsilon}_s) & \text{for } \dot{\varepsilon} > 30 \text{ s}^{-1} \end{cases} \]

where \( \dot{\varepsilon} \) is the strain rate in the range of \( 30 \times 10^{-6} \text{ s}^{-1} \) to \( 300 \text{ s}^{-1} \), \( \dot{\varepsilon}_s = 30 \times 10^{-6} \text{ s}^{-1} \) is the static strain rate and \( \log \gamma_s = 6.156\alpha_s - 2 \), in which \( \alpha_s = 1/(5 + 9\ f_{cd}/f_{cs}) \) where \( f_{cd} = 10 \text{ MPa} \) and \( f_{cs} \) is the static compressive strength of concrete.

Soroushian et al. (1986) proposed an equation for the DIF of concrete in compression by compiling test results reported by different researchers. This DIF is

\[ \text{DIF} = 1.48 + 0.160\log_{10}\dot{\varepsilon} + 0.0127(\log_{10}\dot{\varepsilon})^2 \]

where \( \dot{\varepsilon} \) is a strain rate greater than \( 10^{-5} \text{ s}^{-1} \). However, after observing the wide spectrum of test data, the authors tried to figure out the main source of scatter and noted that the moisture content of concrete is the reason for the variability in results. It was concluded that the strain rate effect on
increasing the compressive strength of concrete becomes more significant as the concrete moisture content increases. The available test data did not show any considerable influence of strain rate on the static compressive strength of concrete. Furthermore, the effect of strain rate on concrete compressive strength was found to be independent of the age of the specimens if their moisture contents were identical. DIFs for dry and wet concrete were suggested as follows.

For dry concrete

3. \[ \text{DIF} = 1.48 + 0.206 \log_{10} \dot{\varepsilon} + 0.0221 (\log_{10} \dot{\varepsilon})^2 \]

and for wet concrete

4. \[ \text{DIF} = 2.54 + 0.580 \log_{10} \dot{\varepsilon} + 0.0543 (\log_{10} \dot{\varepsilon})^2 \]

Ross et al. (1995, 1996) and Tedesco and Ross (1998) conducted a series of SHPB tests to investigate the effect of strain rate and moisture content on concrete strength. The DIF equations for compression suggested by Tedesco and Ross (1998) are

5. \[ \text{DIF} = 0.00965 \log_{10} \dot{\varepsilon} + 1.058 \geq 1.0 \text{ for } \dot{\varepsilon} \leq 63.1 \text{ s}^{-1} \]

6. \[ \text{DIF} = 0.758 \log_{10} \dot{\varepsilon} - 0.289 \leq 2.5 \text{ for } \dot{\varepsilon} > 63.1 \text{ s}^{-1} \]

An experimental examination of the dynamic behaviour of concrete and mortar at very high strain rate and under high hydrostatic pressure was reported by Grote et al. (2001). Quasi-static compression, SHPB and plate impact experiments were used involving strain rates from \(10^{-3} \text{ s}^{-1}\) to \(10^3 \text{ s}^{-1}\) and confining pressure from 0 to 1.5 GPa. The following formulae were suggested to measure the strain rate dependent DIF.

7. \[ \text{DIF} = 0.0235 \log_{10} \dot{\varepsilon} + 1.07 \text{ for } \dot{\varepsilon} \leq 266 \text{ s}^{-1} \]

8. \[ \text{DIF} = 0.882 (\log_{10} \dot{\varepsilon})^2 - 4.4 (\log_{10} \dot{\varepsilon})^2 + 7.22 (\log_{10} \dot{\varepsilon}) - 2.64 \text{ for } \dot{\varepsilon} > 266 \text{ s}^{-1} \]

Li and Meng (2003) examined SHPB application to determine the dynamic strength of concrete-like materials whose compressive strength is dependent on hydrostatic stress. They showed that the apparent dynamic strength enhancement beyond a strain rate of \(10^2 \text{ s}^{-1}\) is strongly influenced by the hydrostatic stress effect due to lateral inertia confinement. The following equations were derived to calculate DIF in compression.

9. \[ \text{DIF} = 1 + (\log_{10} \dot{\varepsilon} + 3) \times 0.03438 \text{ for } \dot{\varepsilon} \leq 100 \text{ s}^{-1} \]

10. \[ \text{DIF} = 8.5303 - 7.1372 \log_{10} \dot{\varepsilon} + 1.729 (\log_{10} \dot{\varepsilon})^2 \text{ for } \dot{\varepsilon} > 100 \text{ s}^{-1} \]

Zhou and Hao (2008) developed a homogenous and meso-scale model to analyse the behaviour of concrete-like material under high strain rate compression. Both strain rate insensitive and sensitive materials were considered in the numerical model to quantify the relative contribution of the inertia effect and the strain rate effect on the compressive strength DIF. The compressive DIF was proposed as

11. \[ \text{DIF} = 0.0225 \log_{10} \dot{\varepsilon} + 1.12 \text{ for } \dot{\varepsilon} \leq 10 \text{ s}^{-1} \]

12. \[ \text{DIF} = 0.2713 (\log_{10} \dot{\varepsilon})^2 - 0.3563 (\log_{10} \dot{\varepsilon}) + 1.2275 \text{ for } 10 \leq \dot{\varepsilon} \leq 2000 \text{ s}^{-1} \]

The dynamic compressive strength of concrete under high strain rates is given by CEB model code 2010 (CEB, 2010) as

13. \[ \frac{f_{c,\text{int},k}}{f_{c\text{m}}} = (\dot{\varepsilon}_c/\dot{\varepsilon}_0)^{0.014} \text{ for } \dot{\varepsilon}_c \leq 30 \text{ s}^{-1} \]

\[ \frac{f_{c,\text{int},k}}{f_{c\text{m}}} = 0.012 (\dot{\varepsilon}_c/\dot{\varepsilon}_0)^{1/3} \text{ for } \dot{\varepsilon}_c > 30 \text{ s}^{-1} \]

where \(f_{c\text{m}}\) is the mean compressive strength of concrete, \(\dot{\varepsilon}_c\) is the concrete compressive strain rate, valid in the range of \(30 \times 10^{-6} \text{ s}^{-1}\) to \(300 \text{ s}^{-1}\) and \(\dot{\varepsilon}_0 = 30 \times 10^{-6} \text{ s}^{-1}\) is the static compressive strain rate.

**Tension**

The DIF for tension as per CEB model code 1990 (CEB, 1993) is

14. \[ \text{DIF} = \begin{cases} (\dot{\varepsilon}_c/\dot{\varepsilon}_0)^{0.016} & \text{for } \dot{\varepsilon}_c \leq 30 \text{ s}^{-1} \vspace{0.5em} \\ \beta_s (\dot{\varepsilon}_c/\dot{\varepsilon}_0)^{1/3} & \text{for } \dot{\varepsilon}_c > 30 \text{ s}^{-1} \end{cases} \]

where \(\dot{\varepsilon}_c\) is the strain rate in the range of \(3 \times 10^{-6} \text{ s}^{-1}\) to \(300 \text{ s}^{-1}\), \(\dot{\varepsilon}_0 = 3 \times 10^{-6} \text{ s}^{-1}\) is the static strain rate and \(\log \beta_s = 7.11 \delta - 2.33\), in which \(\delta = 1(10 + 6 f_{c}\overline{f}_{c\text{m}})\) where

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$f_{co} = 10$ MPa and $f_{cs}$ is the static compressive strength of concrete.

However, Malvar and Ross (1998) found that the available data in the literature and their additional new data on concrete in tension differed somewhat from the CEB model code 1990 (CEB, 1993) recommendations, mostly for strain rates beyond 1 s$^{-1}$. They thus modified the DIF equations with a change in slope occurring at a strain rate of 1 s$^{-1}$ instead of 30 s$^{-1}$. The proposed formulations are

15. \[
\text{DIF} = \begin{cases} \frac{\dot{\epsilon}}{\dot{\epsilon}_s} & \text{for } \dot{\epsilon} \leq 1 \text{ s}^{-1} \\ \frac{\beta(\dot{\epsilon}/\dot{\epsilon}_s)^{1/3}}{1+\delta} & \text{for } \dot{\epsilon} > 1 \text{ s}^{-1} \end{cases}
\]

where $\dot{\epsilon}$ is the strain rate in the range of $10^{-6}$ s$^{-1}$ to 160 s$^{-1}$, $\dot{\epsilon}_s = 10^{-6}$ s$^{-1}$ is the static strain rate and $\log \beta = 6\delta - 2$, with $\delta = 1/(1 + 8 f_p/f_{co})$ and $f_{co}$ and $f_{cs}$ as before.

Tedesco et al. (1997) conducted a series of dynamic tensile tests and, based on the results, proposed the following equations.

16. \[
\text{DIF} = 0.1425 \log_{10} \dot{\epsilon} + 1.833 \geq 1.0 \quad \text{for } \dot{\epsilon} \leq 2.32 \text{ s}^{-1}
\]

17. \[
\text{DIF} = 2.929 \log_{10} \dot{\epsilon} + 0.814 \leq 6 \quad \text{for } \dot{\epsilon} > 2.32 \text{ s}^{-1}
\]

The tensile DIF proposed by Zhou and Hao (2008) is

18. \[
\text{DIF} = 0.0225 \log_{10} \dot{\epsilon} + 1.12 \quad \text{for } \dot{\epsilon} \leq 0.1 \text{ s}^{-1}
\]

19. \[
\text{DIF} = 0.7325(\log_{10} \dot{\epsilon})^2 + 1.235(\log_{10} \dot{\epsilon}) + 1.6
\quad \text{for } 0.1 \leq \dot{\epsilon} \leq 30 \text{ s}^{-1}
\]

Xiao et al. (2010) performed dynamic tensile testing of plain concrete specimens with axial strain rate ranging from $10^{-5}$ s$^{-1}$ to $10^{-3}$ s$^{-1}$. Compared with the quasi-static strain rate of $10^{-5}$ s$^{-1}$, the dynamic tensile strength of concrete at strain rates of $10^{-4}$ s$^{-1}$, $10^{-3}$ s$^{-1}$, $10^{-2}$ s$^{-1}$ and $10^{-1}$ s$^{-1}$ was found to increase by 6.3%, 13.1%, 20.5% and 25.5% respectively. They concluded that this result was similar to the findings of several other researchers. From the test results, they suggested

20. \[
\text{DIF} = 1.0 + 0.0653 \log(\dot{\epsilon}_d/\dot{\epsilon}_s)
\]

where $\dot{\epsilon}_d$ is the dynamic strain rate and $\dot{\epsilon}_s$ is the quasi-static strain rate ($10^{-5}$ s$^{-1}$).

The dynamic tensile strength of concrete under high strain rates was proposed as

21. \[
\begin{align*}
f_{ct,10} &= (\dot{\epsilon}_{ct}/\dot{\epsilon}_{co})^{0.018} & \text{for } \dot{\epsilon}_{ct} \leq 10 \text{ s}^{-1} \\
f_{ct,10} &= 0.0062 (\dot{\epsilon}_{ct}/\dot{\epsilon}_{co})^{1/3} & \text{for } \dot{\epsilon}_{ct} > 10 \text{ s}^{-1}
\end{align*}
\]

where $f_{co}$ is the mean compressive strength of concrete, $\dot{\epsilon}_{ct}$ is the concrete compressive strain rate, valid in the range of $1 \times 10^{-6}$ s$^{-1}$ to $300$ s$^{-1}$ and $\dot{\epsilon}_{co} = 1 \times 10^{-6}$ s$^{-1}$ is the static compressive strain rate.

It is thus quite clear there are plentiful equations to calculate the DIF of concrete in compression and tension. The compression and tension DIFs calculated using various equations proposed by different researchers are plotted in Figure 1 and Figure 2, and variability among the proposed equations is obvious. Numerous factors may affect the constitutive behaviour and DIF of concrete under varying strain rates, such as

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**Figure 1.** DIF in compression of concrete

**Figure 2.** DIF in tension of concrete
Steel reinforcement

Several studies on the effect of strain rate on reinforcing bars, structural steel and steel wires have been documented (Keenan and Feldman, 1960; Soroushian and Choi, 1987; Wakabayashi et al., 1980) and detailed reviews of available work have been presented by Fu et al. (1991) and Malvar (1998). According to Wakabayashi et al. (1980), the yield stress of steel bars increases with increasing strain rate, but the behaviour in the post-yielding and post-hardening region is not largely affected by strain rate. Soroushian and Choi (1987) concluded that the yield strength of steel is more sensitive to strain rate than the ultimate strength, and that the modulus of elasticity is dependent of the rate of straining. According to Soroushian and Choi, the most important factor influencing strain rate effects is the static yield strength, with the mechanical properties of steel of lower yield strength being more sensitive to strain rate than higher yield strength steel. Malvar (1998) reported that the DIF of yield and ultimate stress is inversely related to the yield stress itself. A formulation was proposed to determine the DIF as a function of strain rate and yield stress by fitting available data in the literature. This formulation is valid for yield stresses between 290 MPa and 710 MPa and for strain rates between $10^{-2}$ s$^{-1}$ and 10 s$^{-1}$ and is given by

$$\text{DIF} = (\dot{\varepsilon}/10^{-3})^\alpha.$$  

where, for yield stress, $a = a_{0y} (a_{fy} = 0.074 - 0.04f_y/414)$ and, for ultimate stress, $a = a_{0u} (a_{fu} = 0.019 - 0.009f_y/414)$, $\dot{\varepsilon}$ is the strain rate in s$^{-1}$ and $f_y$ is the static yield strength of the reinforcement in MPa.

Reinforced concrete beams under impact loading: experimental investigations

Hughes and Beeby (1982) conducted a drop-weight impact test programme comprising 80 pin-ended and 12 simply supported beams. Two impactors of masses 58.5 kg and 98 kg were used, and the impact velocity was varied in the range 2.1–7.9 m/s. Various pads (steel, rubber and plywood) were placed at the impact zone to vary its stiffness. Most of the tested beams failed in flexure at midspan, while two beams (those provided with fewer transverse reinforcements than recommended by the British standard) failed in shear at approximately third points. This was a consequence of the high shear at these points caused by excitation of the third mode. The test results indicated that a high impact velocity (the maximum in the considered range) and a stiff (steel) pad resulted in shear dominant behaviour.

Kishi et al. (2001) conducted drop-weight impact tests on eight RC beams to establish a rational impact-resistant design procedure for flexural failure type specimens. Impact load was imparted onto the midspan of specimens by a free-falling 200 kg steel impactor. This work suggested that flexural failure type beams under impact load may be designed with a margin of safety by assuming a dynamic response ratio of 2.0 and a ratio of absorbed energy to input kinetic energy of 0.7. Here, the reaction force versus displacement loop at failure was simplified as a parallelogram, as shown in Figure 3. A simple equation was proposed to calculate the required static bending resistance of beams against impact loading

$$P_{usd} = 0.35 \frac{E_{kd}}{\delta_{kd}},$$

in which $P_{usd}$ is the static bending resistance, $E_{kd}$ is the input kinetic energy and $\delta_{kd}$ is the residual displacement.

Kishi et al. (2002) carried out falling-weight impact tests on shear failure type RC beams to establish a rational impact-resistant design procedure. An impact load was applied at the midspan of the beam by dropping a free-falling 300 kg steel weight. The authors reported that the effects of the striking face of the impactor on the dynamic response and failure mode of the beam were very small under similar impact velocities. It was recommended that shear failure type beams without shear reinforcement under impact loading may be designed with a certain safety margin by assuming a dynamic response ratio of 1.5 and an absorbed input energy ratio of 0.6. The required static shear resistance for beams against impact loading was evaluated by the simple equation

$$V_{usd} = 0.8 \frac{E_{kd}}{\delta_{kd}},$$

in which $V_{usd}$ is the static shear resistance, $E_{kd}$ is the input kinetic energy and $\delta_{kd}$ is residual displacement.
Fujikake et al. (2009) examined the impact responses of 12 RC beams through an experimental study involving drop-hammer impact tests. A hammer of mass 400 kg was dropped freely onto the top surface of a beam at midspan from four different heights. It was concluded that beams with comparatively lower amounts of longitudinal steel reinforcement exhibited only overall flexural failure, while beams with higher amounts of longitudinal reinforcement not only exhibited overall flexural failure but also showed local failure (i.e. crushing of concrete) near the impact loading point. Local failure was substantially reduced when the beam contained heavy longitudinal compression reinforcement. Impact response characteristics such as the maximum impact load, impulse, duration of impact load, maximum midspan deflection and the time taken for maximum midspan deflection increased as the drop height was increased. However, the duration of the impact load, the maximum midspan deflection and the time taken for the maximum midspan deflection were affected by the flexural rigidity of the beams. Moreover, design guidelines were provided for RC beams under impact loading in the form of a flowchart, as shown in Figure 4.

Chen and May (2009) designed a test programme to investigate high-mass and low-velocity impact behaviour of RC beams. All the tests were conducted with a drop-weight of 98·7 kg with an impact velocity of 7·3 m/s. The test variables in this investigation were the support conditions (e.g. pin-ended or simply supported), type of impactor (e.g. hemispherical or flat) and the impact interface (e.g. plywood placed between the beam and impactor or direct impact). The test results revealed that the support conditions had less influence on the impact force than the span length. Moreover, it was concluded that the plywood interface distributed the impact force in a similar manner to the use of a flat impactor.

Saatci and Vecchio (2009) reported on a well-instrumented experimental programme that aimed to contribute towards understanding of the effects of shear mechanisms on the behaviour of RC beams under impact loading. Two different drop-weights (211 kg and 600 kg) were employed in the impact testing. Regardless of the projected static behaviour of beams, all the specimens developed severe diagonal cracks, originating at the impact point and propagating downwards with an angle of approximately 45°, forming shear plugs. It was thus suggested that shear mechanisms must be considered during the development of methods to predict impact responses. Impact forces at the initial stages of response were mainly resisted by the inertia of the specimens before the forces reached the supports. Therefore, the mass and geometric properties of a structure, such as the beam span length, are important factors in resisting impact forces. Finally, based on experience from the test programme, some recommendations for future experimental studies on impact loading were provided.

Tachibana et al. (2010) documented a series of low-speed impact experiments on RC beams with varying span lengths, cross-sections and longitudinal reinforcements. Steel weights of various masses (150, 300 and 450 kg) were employed in the test programme. The following equation was proposed to calculate the maximum midspan deflection of the beam based on impact energy and static ultimate flexure resistance

\[ \delta_{\text{max}} = 0.522 \frac{E_{\text{col}}}{P_u} \]

where \( \delta_{\text{max}} \) is maximum displacement (mm), \( E_{\text{col}} \) is kinetic energy (J) and \( P_u \) is ultimate flexural resistance (kN). The validity of the proposed equation was corroborated through comparisons with other experimental results and numerical results from finite-element simulations. The static bending resistance of the considered beams ranged from 16·7 kN to 66·7 kN and the impact energies varied from 150 J to 5400 J based on variations of mass and impact velocity.

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<th>Design condition</th>
<th>Non-linear analysis</th>
<th>Impact response analysis</th>
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<tr>
<td>✓ Structural size</td>
<td>✓ Load–deflection relation</td>
<td>✓ Max. impact load, ( P_{\text{max}} )</td>
</tr>
<tr>
<td>✓ Material properties</td>
<td>✓ Ultimate deflection, ( \delta_{\text{ultimate}} )</td>
<td>✓ Max. deflection, ( \delta_{\text{max}} )</td>
</tr>
<tr>
<td>✓ Mass and speed of impact body</td>
<td></td>
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**Figure 4.** Design flow chart of RC beam subjected to impact loadings (Fujikake et al. 2009)
Kishi and Mikami (2012) developed an impact-resistance design methodology for RC beams following performance-based design based on falling-weight impact test results. Three different drop-hammer masses were used (300, 400 and 500 kg) and impact velocities were in the range 3.1–7.7 m/s. Flexural cracks were observed not only in the bottom fibre but also in the upper fibre throughout the clear span. Moreover, diagonal shear cracks formed around the midspan area, mainly below the impact point, and these cracks became prominent with increasing impact velocity. On the basis of the relationships between maximum and residual deflections per unit input impact energy $E$, the following empirical design formulae were proposed

$$26. \quad P_{usc} = 0.63 \frac{E}{D_{max}}$$

$$27. \quad P_{usc} = 0.42 \frac{E}{\delta_{rs}}$$

in which $P_{usc}$ is the static flexural load carrying capacity (kN), $E$ is input impact energy (J), $D_{max}$ is the maximum displacement and $\delta_{rs}$ is the residual displacement (mm). These two equations allow for a significant simplification in design approach for structures subjected to impact loading. However, there are some limitations in the application—an input impact energy $E < 15$ kJ and applicability to RC beams having a static flexural load carrying capacity $P_{usc} < 240$ kN and static shear flexural capacity ratio $a > 1.5$.

An impact test programme on 30 RC beams was conducted by the current authors (Adhikary et al., 2015) to examine impact response and failure modes. The test results showed that no shear failure occurred under impact loading in statically flexure-critical beams (i.e. shear to bending resistance ratio greater than one). However, with increasing drop height, more localised failure with extensive concrete crushing at the impact region was observed. On the contrary, a transition in the mode of failure of RC beams from flexural failure at static loading to shear failure at low-velocity impact has been reported in the literature (Hughes and Beeby, 1982; Saatci and Vecchio, 2009). The impact interface (i.e. direct impact or with some interface such as a steel or plywood plate between the impactor and the beam) could be one reason why this change in failure mode was not observed in the test programme conducted by Adhikary et al. (2015). A harder and stiffer contact zone (e.g. when a steel plate is placed between the impactor and the beam) produces more inertia forces (i.e. the majority of impact energy transfers to the beam through the steel plate, which accelerates the beam in the direction of the impact force generating more inertia force), which helps the beam to fail in shear under impact loading. However, in the test programme of Adhikary et al. (2015), due to the direct contact of the impactor with the beam during the impact event, the majority of the impact energy was dissipated during localised crushing of concrete in the impact region, thus developing less inertia force due to less energy transfer to the entire span of the beam.

### Analysis of experimental database

A database of RC beams tested under drop-weight impact loading was compiled from the literature (Bhatti et al., 2009; Chen and May, 2009; Fujikake et al., 2009; Kishi et al., 2001, 2002; Kishi and Mikami, 2012; Saatci and Vecchio, 2009; Tachibana et al., 2010) and is concisely tabulated in Appendix 1 and 2.

Tachibana et al. (2010) proposed an equation to estimate the maximum midspan deflection of beams based on impact energy and static flexural resistance. Similarly, empirical design formulae following the performance-based design concept were suggested by Kishi and Mikami (2012), which involved the static flexural resistance, maximum and/or residual deflection and input impact energy. As limited databases were employed in the above-mentioned research studies, efforts were devoted here to finding relationships among static resistance (both flexure and shear), maximum midspan deflection and input impact energy by using data documented by several research studies reported in the literature.

The compiled database comprises 174 beams tested under drop-weight impact loading at their midspan (details of some of the beams are not available in the literature). The beam details, material properties and static shear to bending resistance ratio are tabulated in Appendix 1. Out of the 174 specimens, 53 were of static shear failure type and the remaining 121 were static flexural failure type beams. All impact responses were sorted based on their first impact. The mass of the impactor, impact velocity and impact responses are succinctly presented in Appendix 2. The geometry of the test specimens, amounts of longitudinal and shear reinforcement, the compressive strength of the concrete and the yield strength of the longitudinal reinforcement span a wide range. All the beams were of rectangular cross-section, with dimensions in the range 100–300 mm width, 150–560 mm height and 1–4 m clear span length. The longitudinal reinforcement ratio varied from 0.84% to 2.75%, whereas the shear reinforcement ratio was in the range 0.11–0.75%. The concrete strength of the specimens was mostly within the range 24–50 MPa. The diameter of longitudinal reinforcements in the test specimens varied from 13–35 mm and the diameter of transverse reinforcements for 90% of the beams was 6 mm. The yield strength of longitudinal reinforcement varied from 345 MPa to 520 MPa. Moreover, various types and shapes of impactors and beam–impactor interfaces were used by different researchers, as shown in Table 1.

Figure 5 shows the relationship between maximum midspan deflection ($\delta_{max}$) versus input impact energy over static flexural
resistance ($E/R_b$). About 90 out of the 121 experimental specimens of static flexural failure type are plotted in Figure 5 as there was insufficient experimental data given for the remaining 31 specimens. The static bending resistance of the considered beams varied from 11.4 kN to 237.5 kN and the input impact energy varied from 0.1 kJ to 19.2 kJ due to the variations in impactor mass (98.7–600 kg) and impact velocity (1–8 m/s). Although there were variations in beam geometry, type of impactor and beam–impactor interface, the values of maximum midspan deflection fit well with the linear line. However, the proposed equation (FC9) can only be applied up to $E/R_b = 150$. More data is thus required in order to extend the applicability of the equation beyond this value of $E/R_b$. Similarly, for statically shear-critical beams, the relationship between maximum midspan deflection ($\delta_{\text{max}}$) and input impact energy over static shear resistance ($E/R_s$) is shown in Figure 6. As mentioned earlier, there was insufficient impact response data for some shear failure type beams from the database, thus the data points shown in Figure 6 only cover 42 out of the 53 specimens. The input impact energy varied from 0.15 kJ to 19.2 kJ due to variations in mass of the impactor (98.7–600 kg) and impact velocity (1–9.3 m/s). It was observed

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<th>Investigators</th>
<th>Type of impactor</th>
<th>Impact interface</th>
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<td>Kishi et al. (2002)</td>
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<td>Direct contact</td>
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<td>Bhatti et al. (2009)</td>
<td>Spherical steel weight with radius of curvature of 1407 mm</td>
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<td>Fujikake et al. (2009)</td>
<td>Drop hammer with hemispherical tip of radius 90 mm</td>
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<td>Chen and May (2009)</td>
<td>Stainless steel with a 90 mm diameter and hemispherical profile of 125 mm radius</td>
<td>12 mm plywood and direct contact</td>
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<td>Mild steel with a 100 mm diameter and flat contact surface</td>
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<td>Saatci and Vecchio (2009)</td>
<td>305 mm square hollow structural steel weight</td>
<td>50 mm thick 305 mm square steel plate</td>
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<tr>
<td>Tachibana et al. (2010)</td>
<td>Steel weight with a curved contact surface, length 565 mm, radius 75 mm</td>
<td>Direct contact</td>
</tr>
<tr>
<td>Kishi and Mikami (2012)</td>
<td>Steel weight with a spherical bottom of radius 1407 mm and 2 mm taper</td>
<td>Direct contact</td>
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Table 1. Type of impactor and beam–impactor interface used in the experimental works

![Figure 5. Relationship between maximum midspan deflection and input impact energy over static flexural resistance](image)

![Figure 6. Relationship between maximum midspan deflection and input impact energy over static shear resistance](image)
that the maximum displacement can be accurately predicted using the equation proposed in the figure, but more data are essential in order to extend the applicability of the equation beyond \( E/R_b = 50 \).

**Empirical formulae for impact-resistant design**

Two empirical equations can be proposed from the aforementioned analysis of the experimental database.

For static flexural failure type beams

\[ R_b = 0.574 \frac{E}{\delta_{\text{max}}} \]

and for static shear failure type beams

\[ R_s = 0.614 \frac{E}{\delta_{\text{max}}} \]

where \( R_b \) and \( R_s \) are in kN, \( E \) is in J and \( \delta_{\text{max}} \) is in mm.

Therefore, by specifying the maximum midspan deflection \( (\delta_{\text{max}}) \) for each limit state of the beam, the required static bending \( (R_b) \) or shear resistance \( (R_s) \) for impact-resistant design can be determined by applying Equations 28 and 29.

To demonstrate the applicability of the proposed equations, comparisons were made with the test results provided by Adhikary et al. (2015). Figure 7 (for flexure-critical beams) and Figure 8 (for shear-critical beams) show the comparative plots of the experimental results of maximum deflection with the proposed equations. The figures show good agreement, and the two proposed equations could thus be used in design with the aforementioned limitations.

The equations proposed for static flexural failure type beams by Tachibana et al. (2010) and Kishi and Mikami (2012) are similar to the proposed equation (Equation 28) apart from the constant values, which were specified as 0.522 and 0.63 by Tachibana et al. (2010) and Kishi and Mikami (2012) respectively. The constant in Equation 28 is 0.574, which is very close to the average of the two previously proposed values; therefore Equation 28 gives a less conservative but more economical calculation of required static bending resistance compared with Kishi and Mikami (2012) and vice versa for Tachibana et al. (2010). Although there are some variations in beam dimensions, test setup, type of impactor and impactor interface in the research works studied, it can be concluded that the required static bending resistance for impact-resistant design is highly dependent on input impact energy and the limit state of maximum midspan deflection.

The required static shear resistance for a static shear failure type beam against impact loading can be evaluated by the simple equation proposed by Kishi et al. (2002)

\[ V_{\text{uid}} = 0.8 \frac{E_{\text{kd}}}{\delta_{\text{rd}}} \]

where \( V_{\text{uid}} \) is static shear resistance, \( E_{\text{kd}} \) is input kinetic energy and \( \delta_{\text{rd}} \) is residual displacement. The constant value in Equation 30 is 0.8 whereas the proposed equation (Equation 29) suggests a constant value of 0.614. The discrepancy is due to the fact that Kishi et al. (2002) considered the limit state of residual deflection \( (\delta_{\text{rd}}) \) as an input parameter while Equation 29 considers the limit state of maximum midspan deflection \( (\delta_{\text{max}}) \). It is therefore difficult to directly compare these two equations.
Conclusions and future research directions

(a) A comprehensive review of the strain rate effects on concrete mechanical properties has been presented and it has been shown that there are numerous equations in the literature to calculate the dynamic increase factor (DIF) of concrete in compression and tension. Numerous factors may affect the constitutive behaviour and DIF of concrete under varying strain rates, such as mix proportion, cement content, aggregate shape and size, water/cement ratio, age, curing conditions and so on. More test data are thus needed to estimate the DIF of concrete strength more precisely and make a comparison with existing data in the literature. Moreover, there seems to be a lack of general consensus on whether the strain rate enhancement of concrete strength is a material effect or a structural effect. Further research is needed to examine this issue.

(b) Two empirical equations are proposed from analysis of an extensive dataset of RC beams under drop-weight impact loading documented by several researchers in the literature. By specifying the maximum midspan deflection for each limit state of a beam, the required static bending and shear resistance can be determined for designing a beam subjected to input impact energy. To demonstrate the applicability of the proposed equations, a comparison was made with results from tests conducted by the authors.

(c) An increased tendency towards shear failure of RC beams under impact loading has been mentioned by several researchers (Hughes and Beeby, 1982; Saatci and Vecchio, 2009) although, under static loading, the beams failed in flexure. The reason for this is the presence of inertia forces, which are responsible for different deflected shapes and subsequently a higher shear to moment ratio (i.e. in higher vibration modes). The impact interface plays an important role in developing the severity of inertia forces. A harder and stiffer contact zone (e.g. when a steel plate is placed between the impactor and the beam) produces more inertia forces (i.e. the majority of impact energy transfers to the beam through the steel plate, which accelerates the beam in the direction of the impact force, generating more inertia force), which will assist the beam to fail in shear under impact loading. In the test programme conducted by the authors (Adhikary et al., 2015), no shear failure occurred in statically flexure-critical beams, but more localized failure with extensive concrete crushing below the impactor was observed with increasing drop-height. Due to the direct contact of the impactor with the beam during the impact event, the majority of impact energy was dissipated during localized crushing of the concrete in the impact region, thus developing less inertia force due to reduced energy transfer to the entire span of the beam. Future research should focus on the issue of whether the failure mode of statically flexure-critical beams under impact loading (i.e. direct contact) changes to shear or not (or localized failure governs).

(d) For a more thorough comparative study, research initiatives should also be undertaken to test identical specimens (i.e. similar not only at the structural level but also at the material level (e.g. aggregate size, compressive strength of concrete and yield strength of reinforcements)) using different drop-weight impact machines (i.e. also keeping the impactor shape and size, impact interface and boundary conditions almost identical). Moreover, the data acquisition systems used should be almost identical in order to facilitate better analysis and comparison of the test results. In general, more efforts are needed to determine the impact resistance of RC beams, considering various structural masses and geometries and various impactor masses and velocities.
Appendix 1 Beam geometry and reinforcement details for drop-weight impact loading

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Kishi et al. (2001)

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Kishi et al. (2002)

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| C       | 24 250 150 210 1000 2400 | 4·76 | 9·6 | 16 1·277 345 16 1·277 345 | 6 | 0·75 | 295 | 1·9 |
| D       | 24 250 150 210 1000 2400 | 4·76 | 9·6 | 10 0·499 345 10 0·499 345 | 6 | 0·75 | 295 | 4·3 |
| E       | 24 400 150 360 1000 2400 | 2·78 | 6 | 13 0·492 345 13 0·492 345 | 6 | 0·75 | 295 | 2·4 |
| F       | 24 400 150 360 1000 2400 | 2·78 | 6 | 10 0·291 345 10 0·291 345 | 6 | 0·75 | 295 | 4 |

Kishi and Mikami (2012)

| G1-1    | 33·7 300 200 260 1500 3400 | 5·77 | 11·33 | 19 1·1 379 19 1·1 379 | 6 | 0·07 | 295 | 2·81 |
| G1-1S   | 33·7 300 200 260 1500 3400 | 5·77 | 11·33 | 19 1·1 379 19 1·1 379 | 6 | 0·07 | 295 | 2·81 |
| G2-1    | 32·2 250 150 210 1000 2400 | 4·76 | 9·6 | 13 0·8 373 13 0·8 373 | 6 | 0·13 | 295 | 3·67 |
| G2-2    | 32·2 250 150 210 1000 2400 | 4·76 | 9·6 | 13 0·8 373 13 0·8 373 | 6 | 0·13 | 295 | 3·67 |
| G2-3    | 32·2 250 150 210 1000 2400 | 4·76 | 9·6 | 13 0·8 373 13 0·8 373 | 6 | 0·13 | 295 | 3·67 |
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### Appendix 2 Impact response characteristics

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Bhatti et al. (2009)

| A37    | 384                  | 0.7                   | 3.67                  | 2.7                      | 765                      | 657                      | 8.13                     | Diagonal cracks and shear plug from impactor point to bottom portion of beam |
|        | A46                  | 1.08                  | 4.58                  | 4.2                      | 900                      | 732                      | 10.13                    | More diagonal cracks, formation of shear plug and flexural cracks |
| A56    | 384                  | 1.6                   | 5.61                  | 6.3                      | 1007                     | 938                      | 13.13                    | Diagonal cracks and more pronounced shear plug |
| A65    | 384                  | 2.15                  | 6.52                  | 8.5                      | 1125                     | 929                      | 16                       | Shear plug and diagonal tension failure |
| A74    | 384                  | 2.79                  | 7.42                  | 11                       | 1250                     | 1125                     | 22                       | More diagonal cracks and shear plug |
| A84    | 384                  | 3.6                   | 8.40                  | 14.1                     | 1350                     | 1088                     | 24.38                    | Diagonal tension failure and spalling of concrete on top of the beam |
| B37    | 384                  | 0.7                   | 3.67                  | 2.7                      | 782                      | 635                      | 7.5                      | Diagonal cracks and flexural cracks |
| B46    | 384                  | 1.08                  | 4.58                  | 4.2                      | 900                      | 816                      | 11.25                    | More diagonal and flexural cracks |
| B65    | 384                  | 2.15                  | 6.52                  | 8.5                      | 1219                     | 954                      | 16                       | Diagonal cracks and formation of shear plug |
| B74    | 384                  | 2.79                  | 7.42                  | 11                       | 1350                     | 1160                     | 19.8                     | Width of diagonal crack in one side increases and spalling of concrete in impact zone starts |
| B84    | 384                  | 3.6                   | 8.40                  | 14.1                     | 1382                     | 1214                     | 24                       | One-sided diagonal cracks and number of cracks increases in impact zones |

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Tachibana et al. (2010)

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Kishi and Mikami (2012)

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n/p = not provided

Continued


