Dynamic behavior of reinforced concrete beams under varying rates of concentrated and impact loadings

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**DYNAMIC BEHAVIOR OF REINFORCED CONCRETE
BEAMS UNDER VARYING RATES OF
CONCENTRATED AND IMPACT LOADINGS**

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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

2014

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BEAMS UNDER VARYING RATES OF
CONCENTRATED AND IMPACT LOADINGS**

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fulfillment of the requirement for the degree of
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2014

STATEMENT OF ORIGINALITY

I, Satadru Das Adhikary, do hereby declare that the research work contained in the thesis entitled “*Dynamic behavior of RC beams under varying rates of concentrated and Impact loadings*” is entirely my original contribution. I also certify here that the thesis has been solely written by me. This thesis has not been previously submitted for any other degree in any other university.

All the resources used during the doctoral study have been duly acknowledged. All testing have been conducted solely by the author with the assistance from Prof. Kazunori Fujikake at National Defense Academy, Japan.

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Last but not least, the author would like to express his deepest gratitude to his wife Piyali and parents, for their unwavering support, encouragement and understanding. I would like to dedicate this thesis to my parents.

LIST OF PUBLICATIONS

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ABSTRACT

Reinforced Concrete (RC) structures may experience various types of loading during their lifespan, which in turn induces a wide spectrum of strain rates. The materials involved in RC structures are strain-rate sensitive and thus, the behavior of structural members can be affected by the loading rates but only significantly when the rate differs by more than one order of magnitude. Majority of past research on RC beams have focused on assessing their behavior under static and relatively slow loading rates while limited attention has been paid to the high loading rates. Therefore, to shed some light in this field, an experimental program comprising twenty-four RC beams was carried out under four different loading rates ranging from slow (4×10^{-4} m/s) to fast (2 m/s) to cover the wide range of loading scenarios (quasi-static, earthquakes and low velocity impact regime). Comparative analyses of beams under these varying loading rates highlighted several important aspects of their dynamic behavior. Furthermore, the influence of various key parameters on dynamic increase factor (DIF) of maximum resistance and failure modes of beams under these loading conditions was summarized and discussed through numerical simulation parametric studies after successful validation of numerical models in an explicit finite element (FE) program.

Although the loading rate effects covering low velocity impact regime were considered, however, it would be more practical to consider realistic impacts. Thus, a drop-weight impact test program was undertaken on thirty RC beams to evaluate their impact responses. The acquired data was then used in the development and verification of numerical and analytical methods. Two empirical equations have been proposed by analyzing a dataset which would aid in determining the required static bending and shear resistance for input impact energy by specifying the maximum midspan deflection for each limit state of beam. Moreover, to extent the knowledge beyond the range of parameters investigated experimentally, FE models of the beams were also developed. Maximum midspan deflection could be an important performance index to evaluate the damage levels of beam when subjected

to impact loadings; hence simplified analytical models were employed to predict the same in less modeling effort and computational time.

Residual performance assessment of damaged structures has been gaining enormous importance in engineering community to ensure that the damaged structures will not fail catastrophically and uphold structural integrity. However, to date no study has been documented in literature on residual performance of impact-damaged RC beams. Thus, the impact-damaged beams have been retested quasi-statically to determine their post-impact residual responses. Finally, the FE models developed earlier for drop-weight impact loading were modified by replacing the impactor with the loading plate to perform quasi-static simulation by utilizing resulting deformation and damages of beams from the impact stages. Thereafter, these models were used further to quantify the effect of various parameters on residual resistance index (RRI), which could be used effectively to delineate the extents of damage of beams after impact.

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LIST OF SYMBOLS

$\dot{\epsilon}$	Strain rate
$\dot{\epsilon}_s$	Static strain rate
$DIF_{c/t}$	Dynamic Increase Factor in compression or tension for concrete
DIF_c	Dynamic Increase Factor in compression for concrete
DIF_t	Dynamic Increase Factor in tension for concrete
DIF_s	Dynamic Increase Factor for reinforcing steel
f_{cs}	Static compressive strength of concrete
f_y	Static yield strength of reinforcement
P_{usd}, P_u	Static bending resistance
E_{kd}, E_{col}, E	Input impact energy
δ_{rd}	Residual displacement
V_{usd}	Static shear resistance
δ_{max}, D_{max}	Maximum displacement
ρ_b	Balanced reinforcement ratio
a	Shear span of beam
d	Effective depth of beam section
a/d	Shear span to effective depth ratio
ρ_L	Longitudinal reinforcement ratio
ρ_T	Transverse or shear reinforcement ratio
v_c	Concrete contribution to shear stress
V_c	Concrete contribution to shear resistance
V_s	Transverse reinforcement contribution to shear resistance
f'_c	Compressive strength of concrete
h	Overall depth of beam section
b	Width of beam section
L	Total span length of beam

l_{clear}	Clear span length of beam
$R_b(t)$	True resistance of beam without inertia force
$P_{TL}(t)$	Load measured by top load-cell
ρ	Density of reinforced concrete
$\ddot{x}_b(t)$	Acceleration at midspan of beam
ρ_{LT}	Compressive longitudinal reinforcement ratio
ρ_{LB}	Tensile longitudinal reinforcement ratio
R_s	Static shear resistance
R_b	Static bending resistance
$\dot{\delta}$	Loading rate
f_{Ly}	Yield strength of longitudinal reinforcement
f_{Ty}	Yield strength of transverse reinforcement
J_1	First invariant of stress tensor
J_2'	Second invariant of deviatoric stress tensor
J_3'	Third invariant of deviatoric stress tensor
S_{ij}	Deviatoric stress tensor
P	Pressure
κ	Cap hardening parameter
F_f	Shear failure surface
F_c	Hardening cap
$\Re$	Rubin three-invariant reduction factor
m_b	Mass of beam
m_i, M	Mass of impactor
V	Velocity just before impact
K_{bs}	Stiffness including bending and shear
K_m	Membrane stiffness
n	Contact stiffness
R_i	Radius of impactor

ν_i	Poisson's ratio of impactor
E_i	Young modulus of impactor
E_b	Young modulus of beam
W_{max}	Maximum deflection
m_e	Effective mass of beam
h	Drop-height of the impactor
R_m	Resistance force at yielding of the beam
u_e	Elastic limit displacement

CHAPTER 1

INTRODUCTION

1.1 Research Background

Reinforced Concrete (RC) structures during their service life may be subjected to various types of dynamic loading in the form of earthquakes, impact or blasts. Structures in low to moderate seismic or earthquake-prone regions could be subjected to dynamic loading due to ground oscillations. Furthermore, with the rapid development of the infrastructural project around the world, there is a chance that RC structural elements or members may be experienced from various types of impact loading due to several events or accidents. Typical examples comprise in a diversified fields ranging from transportation structures (e.g. bridge piers, guard rails, traffic signal posts and electric poles etc.) subjected to vehicle-crash impact, falling rocks on rock-sheds in mountainous regions, falling heavy loads on industrial facilities due to accidents or mishaps resulting from pipe and turbine breaks, marine and offshore structures exposed to ship and ice impact or subjected by tornado or tsunami-borne debris impact, columns in multi-story car park or bridge-pier strike by moving vehicle, protective structures subjected to projectiles or aircraft impact. Thus, in all these above-mentioned cases, it is of fundamental importance to understand the effect of loading rates on structures. Bischoff and Perry [B11] addressed the wide spectrum of strain rates corresponds to different loading conditions typically encountered in practice, as shown in **Fig. 1.1**. During creep deformations, very low strain rates ($\sim 10^{-8}$ to 10^{-7} /s) usually generate. For static loading, strain rates usually ranged from 10^{-6} to 10^{-4} /s. Moreover, for the case of seismic loading, it could be in the domain of 10^{-3} to 1 /s and for hard impact, it would be in the range of 1 to 50/s. Strain rates greater than the hard impact region fall in the blast loading range. In the earthquake range of strain rates, inertial effects on structural elements are generally considered insignificant. However, for strain rates higher than the seismic regime, inertial effects dominate the structural response (Kulkarni and Shah [K13]).

Fig. 1.1: Spectrum of strain rates corresponds to different loading environments
(Bischoff and Perry [B11])

A number of devices or machines are available to perform dynamic tests on structural elements in laboratory for a particular range of strain rates. Kulkarni [K12] summarized the details of different machines or impact loading test setups, suitable for a specific range of strain rates. Hydraulic machines are ideally used to generate the wide range of loading rates extending from 4×10^{-4} to 2 m/s. Corresponding strain rate induced in RC beams were ranging from 10^{-4} to 10/s. However, it could be possible to design hydraulic machines having piston velocities up to 12.5 m/s which would correspond to strain rate of about 50/s. In such testing, some steel plates or spreader beams are usually placed in between the specimens and piston for the case of 3-point or 4-point bending configurations respectively (**Figs. 1.2a and 1.2b**). The piston is generally maintained contact with the specimens via steel plates or spreader beams throughout the test so that sudden contact or impact could be avoided. For higher piston velocities, pneumatic machines could be utilized in which compressed air is used to drive the piston. Attainable piston velocities from this machine might be in the order of 30 m/s which would result in a strain rate of 120/s. Till now, discussions are made on the machines which could generate different range of loading rates which correspondingly produces varying range of strain rates in the structural elements. However, to precisely simulate a natural impact scenarios, real impact tests (i.e. striker or impactor strikes the structural member with a certain initial velocity) are indispensable. These experimental studies attempt to replicate the real impact loading conditions to be experienced by the actual structures to evaluate their impact performance. Strain rates develop in structural elements could be in the range of 1 to 10/s due to the aftermath of the low velocity impact event.

Fig. 1.2: Test configurations of RC beams under varying loading rates (Adhikary *et al.* [A3], Kulkarni and Shah [K13])

Impact tests can be classified mainly into two categories such as drop-weight type and pendulum type, differing in the way the corresponding hammer strikes the specimen. In the case of first type, impactor having known weight is dropped on a specimen from a certain drop-height. In a well-instrumented drop-weight impact test-setup, both global and local structural responses (i.e. impact, reaction force history, mid-span deflection and strain history of reinforcements) could be captured (**Fig. 1.3a**).

Fig. 1.3: Impact loading test setups (Kishi *et al.* [K7], Gopalaratnam *et al.* [G1])

A pendulum-type hammer is used to deliver impact for the second one by releasing the swing hammer from a certain drop-height. Conventional Charpy impact test is a standardized test which determines the amount of energy required for fracturing the specimen. Gopalaratnam *et al.* [G1] described the modified instrumented Charpy test in evaluating the dynamic behavior of cement-based composite (**Fig. 1.3b**) specimens. However, pendulum type hammer can also be used for large scale testing of structural components (Orozco [O1]). Furthermore, it is noted that impulsive loading on structures can also be generated by gas gun and contact blast loading.

RC structures are composite constructs, constituting plain concrete and steel reinforcements. It is well known that constitutive characteristics of plain concrete and steel reinforcements are strain rate dependent. Resistance, stiffness and ductility (or brittleness) of structural members can be affected by the loading rates but, in most cases, the difference only become significant when the rate alters by almost one order magnitude. Therefore, the response of RC structures to varying rates of concentrated loading depends on the magnitude of loading rates and the rate sensitivity of the materials used. Some researchers have pointed out the influence of loading rate on the modes of failure, cracking patterns and energy absorption characteristics of RC structural elements. One of the important phenomena perceived in RC beams under high rates of concentrated loading (by a servo-hydraulic machine with a maximum piston velocity of about 0.75 m/s) is the possible change in the failure mode of a beam from a flexure type of failure to a shear type (Mutsuyoshi and Machida [M11]). A beam designed to fail in flexure in a static test might fail in shear at higher rates of loading. However, an opposite phenomenon (i.e., transition in the mode of failure from shear failure at the static rate to flexure failure at the high loading rate) has been observed by Kulkarni and Shah [K13]. In this investigation, for high rates loading, piston velocity was considered to be 0.38 m/s, which can be related to strain rates expected in an earthquake event.

Drop-weight impact loading cannot be merely adjudged as an extreme case of high rate loading because of the involvement of severe inertia forces, energy transfer

mechanism and complex pattern of stress waves. In this case, several key parameters such as mass, initial velocity of the impactor and the stiffness of the contact region affect the structural behavior. Moreover, under impact loading, the structural responses are divided into two components: primary response (local response) and secondary response (overall response) and these responses are influenced simultaneously by rate effects and the propagation of stress waves. **Fig. 1.4** shows the different tendencies of increase of the stress wave generated and the rate effects incurred in the structural members for three loading cases: impact, high rates and static loading (Takeda *et al.* [T3]).

Fig. 1.4: The different tendencies between impact, high rate loading and static loading (Takeda *et al.* [T3])

It is generally admitted that there is an apparent increase in strength when the concrete-like material is subjected to high strain rate. The dynamic increase factor ($DIF_{c/t}$) defined by the ratio of dynamic strength to quasi-static strength has been widely adopted as an important parameter to measure the rate-dependent material behavior. There are several hypotheses on physical mechanism that explains the concrete material response under varying strain rates. One of them states that the strain rate effects observed is due to the presence of water that may influence the response of the concrete under high strain rates (Rossi *et al.* [R5]). It is attributed that Stefan effect on free water within the concrete may change the cracking patterns under high loading rates. Under quasi-static loading, cracks occur through the cement matrix and when it encounters the coarse aggregates, the cracks will continue to propagate around the boundaries of the aggregate. On the other hand in dynamic loading, due to Stefan effect cracks would penetrate through the coarse

aggregates and it may be the reason behind the strength enhancement. Another angle of view is the lateral inertia force effect which causes an apparent increase in $DIF_{c/t}$ for concrete. Li and Meng [L2] reported that the increase of dynamic compressive strength could be only caused by the lateral confinement when the strain rate is higher than around 10^2 s^{-1} . They termed this lateral inertia confinement as pseudo-strain-rate-effect and further opined that the acceptance of this strain rate effect in design and numerical models may overestimate the dynamic compressive strength of concrete. Zhou and Hao [Z1] developed homogeneous and mesoscale model with strain rate sensitive material model to analyze concrete-like material under high strain-rate compression which corroborate reasonably with test results. Comparison of DIF_c caused by lateral confinement and the DIF_c obtained from dynamic tests shows that the inertial confinement is only one of the two sources that contribute to the DIF_c and this contribution becomes more significant when the strain rate is higher than 1000 s^{-1} . Material strain rate effect cannot be neglected in modeling concrete material response to high loading rates, especially when the strain rate is less than 200 s^{-1} . Most recently Ozbolt *et al.* [O5] performed some numerical studies of compact tension specimen (CTS) loaded by varying rates. After validating the numerical results with experimental results they commented that for strain rate lower than approximately 50 s^{-1} , the structural response is controlled by rate-dependent constitutive law. However, for higher strain rate crack branching and progressive increase in resistance is observed. This is attributed to the effect of structural inertia and not the rate dependent strength of concrete. From numerical point of view, assuming micro and mesoscale analysis, the effects of the rate dependency of the growing micro cracks (e.g. influence of inertia at micro crack level) and viscous behavior of bulk material between the cracks (e.g. viscosity due to water content) can be accounted by the constitutive law. Whereas, the structural inertia effect would be automatically accounted through dynamic analysis (Ozbolt *et al.* [O4])

1.2 Objectives and Scopes

- Experimental outcomes are essential to develop and verify finite element (FE) and analytical methods and these approaches could be employed further to

investigate the structural behavior for various case studies. Previous studies (Mutsuyoshi and Machida [M11] and Kulkarni and Shah [K13]) on high loading rates of RC beams were confined up to the earthquake loading regime. Fujikake *et al.* [F4] considered a much higher piston velocity for high loading rates which could be linked to the impact loading regime; however consider only one test parameter (i.e. flexural reinforcement ratio). Thus, there is a lack of systematic research on RC beams under wide range of varying loading rates to cover static loading to hard impact regime. To fulfill this, hydraulic machine was used to generate the wide range of loading rates (4×10^{-4} m/s to 2 m/s) and the effects of these loading rates on RC beams (in three-point bending configurations) have been studied. Corresponding strain rate would be in the range of 10^{-4} /s to 10/s (enclosed by an ellipse in **Fig. 1.1**). An effort has been made by collecting data (through comprehensive literature review) on beams under varying loading rates to observe the overall trend of dynamic increase factor (DIF) of maximum resistance. Furthermore, to extend the findings from tests, FE model is developed to study the behavior of beams under varying loading rates. It is worthwhile to mention that all tests were monotonic in nature and cyclic deflection reversal case is out of scope of this research. Moreover, the influence of the characteristics of load itself (the relationship between the magnitude of load and time) is not taken into account.

- Another objective of this research is to enhance the understanding of RC beams under drop-weight impact loading which can able to replicate the actual impact scenarios to be experienced by the structures. Although, numerous researches (Hughes and Beeby [H3]; Banthia [B1]; Kishi *et al.* [K5-K9]; Chen and May [C3]; Fujikake *et al.* [F4]; Bhatti *et al.* [B8]; Saatci and Vecchio [S1]; Tachibana *et al.* [T1]; Kishi and Mikami [K10]) have been conducted on evaluating the impact response of RC beams; however till now limited efforts have been sought towards determining the residual performance of beams after impact damage. So, at first, well-instrumented experimental program was planned to impart impact loading to RC beams by drop-hammer and their corresponding responses for different drop-heights have been summarized. Thereafter, the impact-damaged specimens were shifted to 3-point bending test

setup to obtain their quasi-static residual performances. Subsequently, some undamaged specimens were tested under monotonic static loading to compare their response to the impact-damaged specimens. Moreover, set of data (RC beam under drop-weight impact) is assembled and analyzed to propose empirical equations which would aid in determining the required static bending and shear resistance for input impact energy by specifying the maximum midspan deflection for each limit state of beam. Maximum midspan deflection would be an important performance index to evaluate the damage levels (i.e. from no damage to moderate damage, severe damage and complete collapse) of beam when subjected to impact loading (Fujikake *et al.* [F4]). Hence, two analytical approaches such as energy-balance and single-degree-of-freedom (SDOF) model have been employed to predict the maximum midspan deflection of beam. Moreover, a numerical procedure has also been developed in explicit finite element (FE) program LS-DYNA [L1] to reproduce the two stage experiments (the impact and 3-point quasi-static loading tests) which could be valuable in providing some insight about the impact responses as well as the post impact residual performances of damaged specimens. Finally, it is essential to enumerate that impulsive loading on structures caused by gas gun and contact blast loading is not within the scope of the research.

1.3 Structure of Thesis

The thesis comprises of nine chapters and three appendices. The contents of the chapters are briefly illustrated as follows:

Chapter 2 presents a comprehensive survey of available literature in the field of RC beams subjected to varying rates of concentrated and drop-weight impact loading. To understand the structural response under these above-mentioned loading conditions, knowledge on mechanical properties of steel and plain concrete under various ranges of strain rates is essential. Therefore, a brief review of strain rate effects on constitutive characteristics of steel and plain concrete was accomplished in parallel.

Chapter 3 focuses on experimental investigation of loading rate effects on RC

beams which consists of the detailing of the specimens, experimental program, material properties, instrumentations and test results and discussions. Comparative analyses of beams under these varying loading rates highlighted several key aspects of their dynamic behavior.

Three dimensional non-linear FE simulations of beams under varying loading rates are presented in Chapter 4. This FE model is then used to conduct some case studies to quantify the effects of various parameters on DIF of maximum resistance and failure modes.

Chapter 5 describes the experimental assessment of RC beam under drop-weight impact loading. This chapter elaborates the beam details, test program, instrumentation; material properties and results. In addition, empirical equations have been formulated by analyzing a pool of data which would aid in determining the required static bending and shear resistance for input impact energy by specifying the maximum midspan deflection for each limit state of beam.

Chapter 6 discusses the development of the numerical model of beams subjected to drop-weight impact loadings and parametric studies are carried out to determine the effects of several parameters on impact responses.

Chapter 7 presents some analytical models aimed at attaining the key impact responses of the beams in less modeling efforts and computational time.

Chapter 8 demonstrates both experimental and numerical evaluation of residual performance of impact-damaged beams. A numerical procedure comprising two steps is developed in explicit FE program to capture the residual resistance of damaged specimens. These models are used further to quantify the effect of various parameters on residual resistance index (RRI), which could be used effectively to delineate the extents of damage of beams after impact loading.

Chapter 9 summarizes the conclusions drawn from the research work and provides the recommendations for future study. Appendix A tabulates the database of RC beams under various loading rates, accumulated from the literature. Similarly, databank corresponds to RC beams subjected to drop-weight impact loading is

summarized in Appendix B and C.

CHAPTER 2

LITERATURE REVIEW

2.1 Overview

To understand the behavior of RC beams under varying rates of concentrated and drop-weight impact loading, efforts are devoted for both experimental and numerical studies. This chapter presents a comprehensive survey of the strain rates effect on the material behavior (i.e. plain concrete and steel reinforcement) and the response of structural components (i.e. RC beam only). At first, a brief review of available literature on strain-rate dependent mechanical properties of plain concrete and reinforcing steel is performed as the behavior of these materials under dynamic conditions differ significantly from their behavior under static conditions. Influence of strain rates on bond properties are reviewed successively. This is followed by the summary of the behavior of RC beams under varying rates of concentrated and drop-weight impact loading.

2.2 Strain Rate Effect on Mechanical Properties of Materials

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

2.2.1 Plain concrete

Three factors mainly influence the behavior of structures under varying loading rates. These are (a) the rate dependency of growing micro crack; (b) viscous behavior of bulk material between cracks; (c) structural inertia forces which can significantly change the state of stresses and strains of the material [O3]. There are various methodologies for accounting rate-dependent material behavior. Mihashi

and Wittmann [M8] employed the rate theory to explain the first two above-mentioned reasons to explain the rate-sensitivity. This theory asserts that crack initiation on an atomic scale is controlled by the activation energy. More bond breaking occurs than bond healing when an external force acts on the material. Long duration loading causes more bond-rupture than short duration loading as the number of bond-breaking steps is assumed to be constant over time. This means that sustained loading causes a reduction in strength whereas very short duration loading causes an increase in strength. This is the logical explanation behind the increase of strength of the material under high rates. The following expression has been given to express increased strength in compression, tension and flexure:

$$f_d / f_s = (\dot{\sigma} / \dot{\sigma}^0)^\alpha \quad (2-1)$$

where, f_d is dynamic strength; f_s is static strength under monotonic loading; $\dot{\sigma}$ is stress rate under dynamic load; $\dot{\sigma}^0$ is stress rate under static load; α is the parameter depends on load, types of material and way of loading. Reinhardt and Weerheijm [R1] took into account the inertia effect at material level. The authors look at a cluster of penny shaped cracks in a plane which is perpendicular to the tensile loading direction. Cracks face move when the cracks propagate into the material with a certain velocity and for this movement energy balanced is computed. The calculation showed that the rate of energy supplied becomes too high to be absorbed in the fracture process and thus major part of energy supplied is stored at the crack tip. As a result, stress distribution around the crack tip changes and the stress intensity factor decreases with increasing loading rates. This resulted in the strength increases of material with increasing loading rates. In the model of Bazant *et al.* [B5, B6], the influence of loading rates is considered in two parts: the effect of viscosity is accounted by simple visco-elastic model and the influence of strain-dependent growth of micro-cracks is based on the theory of activation energy. Moreover, this model did not consider the influence of inertia forces. The expression proposed is as follows:

$$\sigma(\varepsilon) = \sigma^0(\varepsilon) \left[1 + C_1 \ln \left(\frac{2\dot{\varepsilon}}{C_2} \right) \right] \quad (2-2)$$

where, σ is stress at dynamic loading; σ^0 stress at static loading; $\dot{\varepsilon}$ is strain rate ; C_1 and C_2 are constant determine from experiments. Ozbolt *et al.* [O3] compared the theoretical models with experimental results and from their plot, it can concluded that Reinhardt and Weerheijm [R1] model is better supported by test data than others. This has been recently demonstrated that for low and medium strain rate up to 10 s^{-1} , the total resistance is accounted by viscosity and strain dependent crack growth. However, for high strain rates above 10 s^{-1} , the inertia forces dominate and cause progressive increase of structural resistance. Moreover, dominant influence on failure mode and crack propagation is observed under high rates [O4, O5].

Over the last several decades, numerous studies have been carried out on the compressive behavior of concrete over a wide range of strain rates. Fu *et al.* [F2] and Bischoff and Perry [B11] provides excellent synopsis on compressive behavior of plain concrete subjected to varying strain rates by reviewing the existing literature on it. Some key findings are summarized as follows: both compressive strength and stiffness increase with increasing strain rates; however disharmony prevails among researchers regarding the effect of strain rates on ultimate strain and strain at peak stress; higher strain rates appear to have a more profound effect on normal strength concrete than on high-strength concrete; dry concrete is less rate sensitive than wet concrete; slope of the descending branch in the stress-strain diagram increases with increasing rate of straining.

As compared to the compressive behavior of concrete over a wide range of strain rates, test data of the dynamic behavior of concrete in tension is rather limited. Several methods (split-cylinder tests, direct uniaxial tension tests, Split-Hopkinson Pressure Bar [SHPB] tests etc.) usually are employed in laboratory to determine the rate sensitivity of concrete in tension. Sauris and Shah [S3] investigated the rate sensitivity of concrete in tension, compression and flexure using drop-weight impact machine. It was concluded that the tensile strength of concrete is more

strain-rate sensitive than its compressive strength. Zielinski and Reinhardt [Z2] used the Split-Hopkinson-Bar technique to examine the tensile behavior of concrete and mortar at high stress rates (5000-30000 MPa/sec). It was concluded that the significant increase in the tensile strength in mortar and concrete was due to the excessive micro cracking in the whole volume of stressed specimens. Larger impact strain was observed than static ones which confirmed the above-mentioned hypothesis. Moreover, in impact loading, the specimens had fractures into three pieces. The higher impact tensile strength of concrete than mortar was explained by direct crack arresting action of the tougher aggregates which increased the amount of energy absorbed in the impact tensile fracture process. Furthermore, comprehensive review of strain rate effects for concrete in tension has been performed by Malvar and Ross [M5].

The most comprehensive model for strain rate enhancement of concrete both in tension and compression is presented by the CEB model code [C5]. Strain rate effect on compression and tension is typically reported as dynamic increase factor ($DIF_{c/t}$) - i.e. ratio of dynamic to static strength. In compression CEB model equations come out to be properly fit with the available data. The DIF_c for compressive strength is given by:

$$DIF_c = \begin{cases} (\dot{\epsilon}/\dot{\epsilon}_s)^{1.026\alpha_s} & \dot{\epsilon} \leq 30s^{-1} \\ \gamma_s (\dot{\epsilon}/\dot{\epsilon}_s)^{1/3} & \dot{\epsilon} > 30s^{-1} \end{cases} \quad (2-3)$$

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

Soroushian *et al.* [S8] proposed an equation for DIF_c of concrete in compression by fitting the test results reported by different researchers. The DIF_c is depicted as follows:

$$DIF_c = 1.48 + 0.160 \log_{10} \dot{\epsilon} + 0.0127 (\log_{10} \dot{\epsilon})^2 \quad (2-4)$$

where $\dot{\epsilon}$ is the strain rate (s^{-1}) greater than 10^{-5} . However, after observing the wide spectrum of test data, the authors tried to figure out the main source of scatter.

Finally, they pointed out that the moisture content of concrete is the reason for the variability of 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 rates on 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 are identical. DIF_c for both dry and wet concrete were suggested as follows:

For dry concrete,

$$DIF_c = 1.48 + 0.206 \log_{10} \dot{\epsilon} + 0.0221 (\log_{10} \dot{\epsilon})^2 \quad (2-5)$$

For wet concrete,

$$DIF_c = 2.54 + 0.580 \log_{10} \dot{\epsilon} + 0.0543 (\log_{10} \dot{\epsilon})^2 \quad (2-6)$$

Ross *et al.* [R2, R3] and Tedesco and Ross [T5] conducted a series of SHPB test to investigate the effect of strain rates and moisture content on concrete strength. The DIF_c equations for compression suggested by Tedesco and Ross [T5] are as follows:

$$DIF_c = 0.00965 \log_{10} \dot{\epsilon} + 1.058 \geq 1.0 \quad \text{for} \quad \dot{\epsilon} \leq 63.1 \text{ s}^{-1} \quad (2-7)$$

and

$$DIF_c = 0.758 \log_{10} \dot{\epsilon} - 0.289 \leq 2.5 \quad \text{for} \quad \dot{\epsilon} > 63.1 \text{ s}^{-1} \quad (2-8)$$

An experimental examination of the dynamic behavior of concrete and mortar at very high strain rate and under high hydrostatic pressure was reported by Grote *et al.* [G2]. Quasi-static compression, SHPB and plate impact experiments were used involving strain rate from 10^{-3} to 10^4 s^{-1} and confining pressure from 0 to 1.5 GPa. Following formulae is suggested to measure the strain-rate dependent DIF_c .

$$DIF_c = 0.0235 \log_{10} \dot{\epsilon} + 1.07 \quad \text{for} \quad \dot{\epsilon} \leq 266 \text{ s}^{-1} \quad (2-9)$$

and

$$DIF_c = 0.882 (\log_{10} \dot{\epsilon})^3 - 4.4 (\log_{10} \dot{\epsilon})^2 + 7.22 (\log_{10} \dot{\epsilon}) - 2.64 \quad \text{for} \quad \dot{\epsilon} > 266 \text{ s}^{-1} \quad (2-10)$$

Li and Meng [L2] examined the application of SHPB to determine the dynamic strength of concrete-like materials whose compressive strength is hydrostatic-stress-dependent. It was shown that the apparent dynamic strength enhancement beyond the strain-rate of 10^2 s^{-1} is strongly influenced by the hydrostatic stress effect due to lateral inertia confinement. Following equations were derived to calculate DIF_c in compression.

$$DIF_c = 1 + (\log_{10} \dot{\varepsilon} + 3) \times 0.03438 \quad \text{for} \quad \dot{\varepsilon} \leq 100 \text{ s}^{-1} \quad (2-11)$$

and

$$DIF_c = 8.5303 - 7.1372 \log_{10} \dot{\varepsilon} + 1.729 (\log_{10} \dot{\varepsilon})^2 \quad \text{for} \quad \dot{\varepsilon} > 100 \text{ s}^{-1} \quad (2-12)$$

Katayama *et al.* [K1] demonstrated a numerical simulation methodology to analyze the local damage and dynamic response of structures composed of RC and /or other geological material subjected to high explosive loads. Moreover, they simulated the dynamic behavior of the RC structures subjected to impulsive loading (i.e. velocity > 100 m/s) and numerical analysis results were compared to those reference experiments. To incorporate strain rate effect into Drucker-Prager's equation for compressive strength in concrete, they used the DIF_c formula proposed by Yamaguchi *et al.* [Y1]. The corresponding equation is as follows:

$$DIF_c = 1.021 + 0.05076 \log_{10} \dot{\varepsilon} + 0.2583 (\log_{10} \dot{\varepsilon})^2 \quad (2-13)$$

Shkolnik [S6] demonstrated the influence of high strain rates on the stress-strain relationships, strength and modulus of elasticity using thermo fluctuation theory, principle of accumulation and development of damages as well as nonlinear behavior of concrete. The obtained equations describe and unify the influence of the high strain rates on constitutive properties of concrete under various uniaxial stresses. Comparative study has been performed with existing experimental results and the general equation given by CEB model code [C5]. It was concluded that substantial agreement found among them. The proposed DIF_c formula in compression is as follows:

$$DIF_c = 1 + (2kT/W_a) \varepsilon^2 / \varepsilon_c \ln \dot{\varepsilon}_d / \dot{\varepsilon}_s (E_0 \varepsilon - (E_0 / 2 \varepsilon_c) \varepsilon^2) \quad (2-14)$$

where, k is Boltzmann constant; $kT = 404 \times 10^{-23}$ J at the usual temperature of test ($T = 293$ K); ε_c is the strain of breaking the interatomic bonds; E_0 is tangent modulus of elasticity; W_a is volume of atom; $\dot{\varepsilon}_d, \dot{\varepsilon}_s$ are strain rates at dynamic and quasi-static loading.

Zhou and Hao [Z1] developed homogenous and mesoscale model to analyze the behavior of concrete-like material under high strain-rate compression. Both strain rate insensitive and strain rate sensitive materials were considered in the numerical model to quantify the relative contribution of inertia effect and strain rate effect on the compressive strength DIF_c . The proposed compressive DIF_c is as follows:

$$DIF_c = 0.0225 \log_{10} \dot{\varepsilon} + 1.12 \quad \text{for} \quad \dot{\varepsilon} \leq 10 \text{ s}^{-1} \quad (2-15)$$

and

$$DIF_c = 0.2713(\log_{10} \dot{\varepsilon})^2 - 0.3563(\log_{10} \dot{\varepsilon}) + 1.2275 \quad \text{for} \quad 10 \leq \dot{\varepsilon} \leq 2000 \text{ s}^{-1} \quad (2-16)$$

Fujikake et al. [F5] carried out tri-axial rapid compressive loading tests on concrete specimens to formulate the dynamic constitutive model for concrete. DIF_c in compression was proposed as follows:

$$DIF_c = \left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_{sc}} \right)^{0.006[\log(\dot{\varepsilon}/\dot{\varepsilon}_{sc})]^{1.05}} \quad (2-17)$$

where, $\dot{\varepsilon}_{sc} = 12 \times 10^{-5} \text{ s}^{-1}$ and $\dot{\varepsilon} < 10 \text{ s}^{-1}$.

Uniaxial compressive test on concrete cubes with grade C30 and C50 was performed by Li and Li [L3] using electro-hydraulic servo-controlled testing machine (displacement-controlled load). Four strain rates (e.g., 10^{-5} , 10^{-4} , 10^{-3} and 10^{-2}) were considered in this test program. Based on results, the DIF_c of compressive strength of grade C30 is reported as follows:

$$DIF_c = 1.0 + 0.0648 \log \left(\frac{\dot{\varepsilon}_c}{\dot{\varepsilon}_{so}} \right) \quad (2-18)$$

Similarly, the DIF_c of compressive strength of C50 was obtained as follows:

$$DIF_c = 1.0 + 0.0314 \log \left(\frac{\dot{\epsilon}_c}{\dot{\epsilon}_{so}} \right) \quad (2-19)$$

where, $\dot{\epsilon}_c$ implied the strain rate of concrete and $\dot{\epsilon}_{so}$ denotes the quasi-static strain rate of concrete.

Moreover, the DIF_t for tension as per CEB model code [C5] is furnished as:

$$DIF_t = \begin{cases} (\dot{\epsilon}/\dot{\epsilon}_s)^{1.016\delta_s} & \dot{\epsilon} \leq 30s^{-1} \\ \beta_s (\dot{\epsilon}/\dot{\epsilon}_s)^{1/3} & \dot{\epsilon} > 30s^{-1} \end{cases} \quad (2-20)$$

where $\dot{\epsilon}$ is the strain rate in the range of $3 \times 10^{-6} s^{-1}$ to $300 s^{-1}$; $\dot{\epsilon}_s = 3 \times 10^{-6} s^{-1}$ (static strain rate); $\log \beta_s = 7.11\delta - 2.33$; $\delta_s = 1/(10 + 6f_{cs}/f_{co})$; $f_{co} = 10$ MPa; f_{cs} is the static compressive strength of concrete.

However, it was found by Malver and Ross [M5] that the available data in literature and their additional new data on concrete in tension differed somewhat from the CEB [C5] recommendations, mostly for strain rates beyond $1 s^{-1}$. Thus, they modified the DIF_t equations and the change in slope occurs to strain rate of $1 s^{-1}$ instead of $30 s^{-1}$. The proposed formulations then becomes

$$DIF_t = \begin{cases} (\dot{\epsilon}/\dot{\epsilon}_s)^\delta & \dot{\epsilon} \leq 1s^{-1} \\ \beta (\dot{\epsilon}/\dot{\epsilon}_s)^{1/3} & \dot{\epsilon} > 1s^{-1} \end{cases} \quad (2-21)$$

where $\dot{\epsilon}$ is the strain rate in the range of 10^{-6} to $160 s^{-1}$; $\dot{\epsilon}_s = 10^{-6} s^{-1}$ (static strain rate); $\log \beta = 6\delta - 2$; $\delta = 1/(1 + 8f_{cs}/f_{co})$; $f_{co} = 10$ MPa; f_{cs} is the static compressive strength of concrete.

A series of dynamic tensile tests were conducted by Tedesco *et al.* [T4] and based on the results they proposed following equations:

$$DIF_t = 0.1425 \log_{10} \dot{\epsilon} + 1.833 \geq 1.0 \quad \text{for} \quad \dot{\epsilon} \leq 2.32 s^{-1} \quad (2-22)$$

and

$$DIF_t = 2.929 \log_{10} \dot{\epsilon} + 0.814 \leq 6 \quad \text{for} \quad \dot{\epsilon} > 2.32 s^{-1} \quad (2-23)$$

Direct tension, splitting tensile and direct compression test of mortar and concrete were performed by Ross et al. [R4] at strain rate 10 and 10^2 s^{-1} on a SHPB. Quasi-static tests on the same kind of specimens were conducted using a standard material testing machine so that DIF_t values can be computed. The suggested DIF_t in tension is as follows:

$$DIF_t = \exp \left[0.00126 \log_{10} \left(\frac{\dot{\epsilon}_d}{\dot{\epsilon}_s} \right)^{3.373} \right] \quad (2-24)$$

where, $\dot{\epsilon}_s = 10^{-7} \text{ s}^{-1}$ and d, s = subscript for dynamic and static loading.

An experimental investigation on the strain rate dependent behavior of concrete in tension was carried out by Yan and Lin [Y3] in a servo-hydraulic testing machine. The strain rates ranged from 10^{-5} to $10^{-0.3} \text{ s}^{-1}$. Specimens were grouped according to their water contents and temperature environment. By least square curve fitting of the experimental data, the DIF_t has been expressed as a linear function of strain rate on the semi-log scale. Formulas for various groups are listed below:

For Group A:

$$DIF_t = 1.0 + 0.134 \log(\dot{\epsilon}_t / \dot{\epsilon}_{ts}) \quad (2-25)$$

For Group B:

$$DIF_t = 1.0 + 0.135 \log(\dot{\epsilon}_t / \dot{\epsilon}_{ts}) \quad (2-26)$$

For Group C:

$$DIF_t = 1.0 + 0.265 \log(\dot{\epsilon}_t / \dot{\epsilon}_{ts}) \quad (2-27)$$

For Group D:

$$DIF_t = 1.0 + 0.115 \log(\dot{\epsilon}_t / \dot{\epsilon}_{ts}) \quad (2-28)$$

where, $\dot{\epsilon}_t$ is strain rate in the range from 10^{-5} to $10^{-0.3} \text{ s}^{-1}$; $\dot{\epsilon}_{ts}$ is the quasi-static strain rate, 10^{-5} s^{-1} . Comparison between Eq. (2-21) and Eq. (2-23) reveals that the strength increases in fully saturated concrete are much greater than those experienced by concrete with normal moisture content.

The proposed tensile DIF_t as per Zhou and Hao [Z1] is as follows:

$$DIF_t = 0.0225 \log_{10} \dot{\epsilon} + 1.12 \quad \text{for} \quad \dot{\epsilon} \leq 0.1 \text{ s}^{-1} \quad (2-29)$$

and

$$DIF_t = 0.7325(\log_{10} \dot{\epsilon})^2 + 1.235(\log_{10} \dot{\epsilon}) + 1.6 \quad \text{for} \quad 0.1 \leq \dot{\epsilon} \leq 50 \text{ s}^{-1} \quad (2-30)$$

Xiao *et al.* [X1] performed some dynamic tensile testing of plain concrete specimens with axial strain rate ranging from 10^{-5} to 10^{-1} s^{-1} . As compared to the quasi-static strain rate of 10^{-5} s^{-1} , the dynamic tensile strength of concrete at strain rates of 10^{-4} , 10^{-3} , 10^{-2} and 10^{-1} s^{-1} increase 6.37%, 13.08%, 20.48% and 25.47% respectively. They concluded that this result is almost similar to the several researchers. From the test results, they suggested following equation:

$$DIF_t = 1.0 + 0.0653 \log(\dot{\epsilon}_t / \dot{\epsilon}_{ts}) \quad (2-31)$$

where, $\dot{\epsilon}_t$ is dynamic strain rate; $\dot{\epsilon}_{ts}$ is quasi-static strain rate (10^{-5} s^{-1})

Therefore, it is seen that there are plentiful of equations to calculate the $DIF_{c/t}$ of concrete in compression and tension. Numerous factors may affect the constitutive behavior and $DIF_{c/t}$ of concrete under varying strain rates such as mix proportion, cement content, aggregate shape and size, water-cement ratios, age and curing conditions etc. Thus, more test data are indeed needed in future to estimate the $DIF_{c/t}$ more precisely.

2.2.2 Steel reinforcement

Several studies have been documented on the effect of strain rate on reinforcing bars, structural steel and steel wires (Keenan and Feldman [K2]; Wakabayashi *et al.* [W1]; Soroushian and Choi [S7]. A detailed review of the available work has been conducted by Fu *et al.* [F3] and Malvar [M2]. According to Wakabayashi *et al.* [W1], yield stress of steel bar increases with increasing strain rate but the behavior in the strain hardening region is not affected largely by a strain rate. Soroushian and Choi [S7] concluded that the yield strength of steel is more strain-rate sensitive than the ultimate strength. The modulus of elasticity is independent of rate of straining.

According to them, the most important factor influencing the strain rate effects is the static yield strength. The mechanical properties of steel with lower yield strength are more strain rate-sensitive than higher one. Malvar [M2] narrated that the DIF_s of yield and ultimate stress is inversely related to the yield stress itself. A formulation was proposed to find out the DIF_s as a function of strain rate and yield stress by fitting the available data in literature. This formulation is valid for yield stresses in between 290 and 710 MPa and for strain rates in between 10^{-4} s^{-1} and 10 s^{-1} . The formulation which gives the DIF_s for both yield and ultimate stress is as follows:

$$DIF_s = (\dot{\epsilon}/10^{-4})^\alpha \quad (2-32)$$

where for yield stress, $\alpha = \alpha_{fy}$; $\alpha_{fy} = 0.074 - 0.04 f_y / 414$; for ultimate stress, $\alpha = \alpha_{fu}$; $\alpha_{fu} = 0.019 - 0.009 f_y / 414$; $\dot{\epsilon}$ is the strain rate in s^{-1} and f_y is static yield strength of reinforcement in MPa.

2.3 Influence of Loading Rates on Bond Properties

Bond behavior in between reinforcement and concrete plays a vital role in evaluating the response of RC structures. Hitherto, plentiful research has been conducted to understand the bond phenomenon under static loading conditions. However, for the case of bond behavior under varying strain rates, very less information exists. Von and Reinhardt [V1], Takeda [T2], Yan [Y2] and Weathersby [W2] made important contributions in this regard. Neither the bond resistance nor the shape of the bond stress vs. displacement of plain reinforcing bars is significantly affected by the loading rates. However, there is an immense influence of the loading rate on the bond resistance and the bond stress vs. displacement behavior of deformed bars. Bond stiffness also increases with increasing strain rates (Von and Reinhardt [V1]). The following expression has been given by Von and Reinhardt [V1] by analyzing the test data.

$$\frac{\tau}{\tau_0} = \left\{ \frac{\dot{\tau}}{\dot{\tau}_0} \right\}^\eta \quad (2-33)$$

where, τ and τ_0 are the average bond stress belonging to certain displacement; $\dot{\tau}$ and $\dot{\tau}_0$ are the respective loading rates; τ_0 and $\dot{\tau}_0$ can be conceived as static reference value; η is a parameter which depends upon material and environmental conditions and in this case η is a function of concrete quality and the reinforcement displacements; $\eta = \frac{0.7(1-2.5\delta)}{f_c^{0.8}}$ with δ is the rebar displacement in mm and f_c is cube compressive strength in MPa and the relation is valid within the limits $0 < \delta < 0.2$ mm.

Yan [Y2] designed a testing program aimed at attaining the bond behavior between reinforcement and concrete under impact loading. It was concluded that for smooth bars, the bond resistance is due to the chemical adhesion and the frictional force at the interface and there exists a linear bond stress-slip relationship under both static and high rate loading. Whereas, for deformed bars, bond stress-slip relationship under high rates varies with time and is different at various points along the reinforcements. High loading rates significantly increases the bond resistance capacity. Not only high loading rates but also higher compressive strength of concrete significantly increases the bond resistance capacity and fracture energy at bond failure. At the same time, these factors greatly influence the stress distribution in concrete, the slips at the interface between reinforcements and concrete, and the crack development. Moreover, under high rates, the stress distribution along the reinforcements is not uniform, and not even linear; there is more stress concentration along the reinforcements than under static loading.

2.4 Research on RC Beams under Varying Loading Rates

This section will provide an overview on limited experimental and analytical investigation of on loading rate effects on RC beams.

2.4.1 Experimental investigations

Mutsuyoshi and Machida [M11] tested twenty seven singly and doubly RC beams under different loading rates, noting that the final mode of failure shifted from

flexure at a low rate to shear at high loading rate for two pairs of specimens having a/d - 4 and 5.7, where a - shear span and d - effective depth of the beams. Seven pairs of singly reinforced beams were tested by Kulkarni and Shah [K13]. For each pair, one beam was tested at a ‘static’ rate (7.1×10^{-6} m/s) while the other at a ‘high’ rate (3.8×10^{-1} m/s). In this study, the strain rates generated in the reinforcements of beams by high loading rates can be correlated to the earthquake-induced strain rates. At the high rate, there was no recognizable ‘yield point’ and ‘yield plateau’ in the load-midspan deflection curves of a beam failing in flexure. Furthermore, for the three pairs of beams (a/d - 4, 4.5 & 5) tested by them, an opposite phenomenon (i.e. transition in the mode of failure from shear failure at the static rate to flexure failure at the high loading rate) has been observed. It was concluded that the difference in the observed failure tendencies in these studies might be attributed to the strain rate effect on the yield strength of steel used in the two studies (Mutsuyoshi and Machida [M11] and Kulkarni and Shah [K13]). From **Fig. 2.1**, it is apparent that the flexural strength of RC beams used by Mutsuyoshi and Machida [M11] increased at a much higher rate than the shear strength, indicating an increased tendency towards shear failure at high rates. However, the converse is true for RC beams used by Kulkarni and Shah [K13]. Two different grades of steel used in these studies affects the relative capacity (e.g. flexure and shear strength) of beams under varying strain rates which would be the reason behind the opposite phenomena observed in changes of failure mode from static to high rates. Furthermore, Fujikake *et al.* [F4] performed the testing of three pairs of doubly RC beams with varying amount of longitudinal reinforcements, but the shear reinforcement ratios were kept identical for all specimens. All beams were designed as under-reinforced and with a shear to bending resistance ratio greater than one. For each pair, one beam was tested at a “static” rate (5×10^{-4} m/s) while the other at a “high” rate (2×10^0 m/s). As compared to Mutsuyoshi and Machida [M11] and Kulkarni and Shah [K12], Fujikake *et al.* [F4] considered a much higher piston velocity for high loading rates which could be linked to the impact loading regime. The main intention of this test program was to validate loading rate dependent analytical model which would aid in to acquire the spring characteristics (i.e. load-midspan deflection relationship of RC beam with loading rate effects) for two-

degree-of- freedom model. All specimens failed in flexure under both static and high loading rates. The ultimate resistance of the beams was enhanced by the increment of longitudinal tensile reinforcements for both static and high loading rates. A database of RC beams under different loading rates has been assembled from the literature (Mutsuyoshi and Machida [M11]; Kulkarni and Shah [K13]; Fujikake *et al.* [F4]; Adhikary *et al.* [A3 and A4]). The database containing the particulars and response of the specimens (i.e. geometric characteristics of beam, reinforcement ratios, material properties, peak resistance and failure modes etc.) is succinctly tabulated in the Appendix A.

Fig. 2.1: Relative enhancement of flexural and shear strengths of RC beams
(Kulkarni and Shah [K13])

2.4.2 Analytical and numerical studies

A simple sectional analysis was first attempted by Kulkarni and Shah [K13] to predict the load-midspan deflection of RC beams under high rates. However, standard sectional analysis with rate-dependent material properties did not capture the shape of the curves adequately. Extreme localized yielding of reinforcing bars due to enhanced bond properties at high rates could be the reason behind that. Thus, the sectional analysis was augmented by incorporating the shape of the average stress-strain curve of reinforcing bars consistent with the characteristics of localized

yielding which leads to improved prediction of load-midspan deflection curves.

Fujikake *et al.* [F4] proposed a section based nonlinear analytical model to capture the load-midspan deflection relationship of RC beams under rapid flexural loading. Load-midspan deflection relationship was obtained through moment-curvature relationship by sectional analysis technique, in which the strain rate effects of concrete and steel reinforcement were duly considered.

2.5 Study on RC Beams under Drop-weight Impact Loading

Impacts are generally classified as hard and soft impact. For the case of soft impact, the missile itself also deforms significantly whereas in hard impact, the missile undergoes almost no deformation compared to impacted structures. Depending on the nature of impact, the impacted structure may respond in several ways: (a) it may suffer local damage only, dissipating the majority of impact energy at or around the impact zone; (b) it may respond to the impact loading globally through the deformation of the entire member; (c) it may respond in such a way that it suffers a combination of both local and global damage. Local damage is usually categorized as follows: (a) penetration of the missile in the front face and scabbing of small pieces of concrete at the back face; (b) significant scabbing of the concrete at both faces; (c) perforation of the element, with the missile existing from the back face with a residual velocity. Local and overall impact phenomena for hard missile impact are schematically presented in **Fig. 2.2**.

Fig. 2.2: Missile impact phenomena (Kennedy [K3])

This section gives a brief summary of research on the global response of RC beam under drop-weight impact loads. It is worthwhile to mention that research on the use of special materials, such as high strength concrete or fibre reinforced concrete for increasing the impact resistance and the local impact response of beam are omitted in this review for brevity.

2.5.1 Laboratory tests

One of the earliest impact test program was carried out by Mylrea [M12], in which 254 x 406.4 mm (width by depth) RC beams with 2.44 m spans having varying amounts of reinforcing steels of various grades subjected to various falling weights (254 kg and 925 kg drop weight hammer were used). The beams contained different amounts and grades of longitudinal reinforcement and no shear reinforcement. Although the beams were severely damaged with diagonal shear cracks, it was concluded that the impact resistance of beams was quite significant in the presence of even small amounts of reinforcement, since it was impossible to rupture any of the longitudinal reinforcement. The failure mode of beams is shown in **Fig. 2.3**.

Fig. 2.3: Failure mode of the RC beams (Mylrea [M12])

The factor of safety against rupture of the reinforcement was too high. Moreover, there was enormous reserve strength against collapse after the yielding of the reinforcing bars. Similar to the impact performance of beam containing the structural steel reinforcement, the impact behavior of beam containing the rail-steel and other brittle steel reinforcement was quite satisfactory.

Kishi *et al.* [K5] conducted drop-weight impact tests on eight RC beams having clear span length of 2 m to establish a rational impact-resistant design procedure for flexural-failure-types specimens. Impact load was imparted onto the midspan of specimen by free-falling 200 kg steel-impactor, as exhibited in **Fig. 2.4(a)**. Cross-sectional dimensions, reinforcement ratios and impact velocity were considered as experimental variables. Ultimate resistance of flexural-failure type beams subjected to drop-weight impact loading was estimated by using the maximum reaction force at failure. Configuration of the hysteretic loop between reaction force and the mid-span displacement at failure was approximated by a parallelogram, as shown in **Fig. 2.4(b)**.

(a) Impact test set up

(b) Hysteretic loop

Fig. 2.4: Impact test set up and hysteretic loop between reaction force and mid-span displacement of beam (Kishi *et al.* [K5])

This is suggested that flexure-failure type beams under impact load may be designed with a margin of safety by assuming dynamic response ratio as 2.0 and ratio of absorbed energy to input kinetic energy as 0.7. A simple equation was proposed to calculate the required static bending resistance of beams against impact loading. The formulation to estimate the required static bending resistance is as follows:

$$P_{usd} = 0.35 \frac{E_{kd}}{\delta_{rd}} \quad (2-34)$$

where P_{usd} is static bending resistance, E_{kd} is input kinetic energy and δ_{rd} is residual displacement.

Kishi *et al.* [K7] carried out falling-weight impact tests on shear-failure type RC beams to establish a rational impact-resistant design procedure. In total, twenty-seven simply supported rectangular beams (150 mm width and 250 mm depth) without shear reinforcement were constructed to perform that test program. Longitudinal reinforcements, shear span to effective depth ratios, static shear to bending resistance ratio and clear span length were taken as test variables. An impact load was applied at the mid span of the beam by dropping a free-falling 300 kg steel-weight, as depicted in **Fig. 2.5**. The striking face of the steel-weight was spherical with a radius of curvature of 1407 mm. This has been confirmed by the

same author that the effects of the striking face of the impactor on the dynamic response and failure mode of the beam are very small under similar impact velocity.

Fig. 2.5: Impact test setup (Kishi *et al.* [K7])

Fig. 2.6 presents the crack patterns of two series specimens under various impact velocities. It was addressed that if the static shear to bending resistance ratio is less than unity (e.g. 0.64 for beam A36), the beams clearly collapse in a shear-failure mode under impact loading. On the contrary, if the static shear to bending resistance ratio is greater than one (e.g. 1.03 for beam B36); the RC beams collapse in a bending-failure mode under low-velocity impact loading but may collapse in a shear-failure mode under high-velocity impact loading. The shape of the hysteretic loop between the reaction force and the mid span displacement was assumed to be triangular as shown in **Fig. 2.7** when the beams just collapsed in shear-failure mode. Kishi *et al.* [K7] 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 absorbed input energy ratio of 0.6. Required static shear resistance for beams against impact loading could be evaluated by a simple equation as follows:

$$V_{usd} = 0.8 \frac{E_{kd}}{\delta_{rd}} \quad (2-35)$$

where V_{usd} is static shear resistance, E_{kd} is input kinetic energy and δ_{rd} is residual displacement.

Fig. 2.6: Crack patterns for beams (a) A36 and (b) B36 (Kishi *et al.* [K7])

Fig. 2.7: Simplified loop for reaction force-midspan displacement of RC beams being collapsed in shear-failure mode (Kishi *et al.* [K7])

Bhatti *et al.* [B8] reported a test program of shear-failure type beams, aimed at developing a simple elasto-plastic impact response analysis by finite element (FE) method. The test program consisted of twelve simply supported beams, having 200 mm wide, 400 mm deep and 2400 mm long. Impact load was applied at mid span of RC beam by dropping a 400 kg steel-weight from predefined height. Following experimental results were compared with numerical analysis results to check the

reliability of the FE method: 1) time histories of impact force, reaction force and mid-span displacement; 2) hysteretic loops of impact force vs. mid-span displacements and reaction force vs. mid-span displacements; 3) crack profiles on side surface of beam.

Fujikake *et al.* [F4] examined the impact responses of twelve RC beams through an experimental study which involves a drop-hammer impact test. The beams were of 250 mm in depth, 150 mm in width and 1700 mm in length. The specimens used in this research consisted of under-reinforced sections with sufficient amount of transverse reinforcements to allow for an overall flexural failure. Investigation was carried out to evaluate the influence of drop-height and the amount of longitudinal reinforcement ratio to the structural response. The static shear to bending resistance ratios of beam were varied from 1.5 to 2.6. **Fig. 2.8** displays the drop-hammer impact test setup where a hammer with a mass of 400 kg was dropped freely onto the top surface of the beam at midspan from four different heights. The striking head of the hammer had a hemispherical tip with a radius of 90 mm.

Fig. 2.8: Drop-hammer impact test setup (Fujikake *et al.* [F4])

Typical failure modes obtained from this test program are shown in **Fig. 2.9**. Beams of series S1616 exhibited an overall flexural failure at all the drop-heights considered for this test program. However, for specimens of series S1322 and S2222, the overall flexural failure was observed up to a drop-height of 0.6 m. Local

failure with extensive concrete crushing near the impact region was monitored at a drop height of 1.2 m and 2.4 m.

Fig. 2.9: Failure modes: (a) S1616 series; (b) S1322 series; (c) S2222 series
(Fujikake *et al.* [F4])

It was concluded that the amount of longitudinal reinforcement ratio significantly affected the failure modes of beams. Beams with comparatively lower amounts of longitudinal steel reinforcement exhibited only overall flexural failure, while beam having comparatively higher amount of longitudinal reinforcement not only exhibited the overall flexural failure but also local failure located near 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, the impulse, the duration of impact load, the maximum midspan deflection and the time taken for the maximum midspan deflection increased as the drop-height was increased. However, the duration of impact load, the maximum midspan deflection and the time taken for the maximum midspan deflection were affected by the flexural rigidity of beams.

Chen and May [C3] accomplished a test program to investigate the high mass and low velocity impact behavior of RC beams. Fourteen 2.7 m and four 1.5 m clear-span length specimens were tested under impact loads using drop-weight facility. All the tests were conducted under a drop-weight of 98.7 kg with an impact velocity of 7.3 m/s, as shown in **Fig. 2.10**. Support conditions (e.g. pin-ended and simply supported), type of impactors (e.g. hemispherical and flat) and impact interface (e.g. plywood placed in between beam and impactor and direct impact) were taken as

test variables in this investigation. Measurements were taken during testing which includes transient impact force-history, acceleration-history, strain-history of longitudinal reinforcements and video footage of crack profiles generated from the images captured by high-speed camera.

Fig. 2.10: Drop-weight impact test set up (May *et al.* [M6])

Three types of failure modes were observed by the authors:

Failure mode – a: Flexural failure with some crushing beneath the impactor and some shear cracking in the impact zone were observed. Vertical cracks starting from the top of a beam were found along the beam section away from the impact zone as presented in **Figs. 2.11(a) and (b)** for beams A1 and A2 respectively. Further shear cracking and short vertical cracking occurred in beam A3, as shown in **Fig. 2.11(c)**. There was less damage in this type of failure than in modes ‘b’ and ‘c’, described below, owing to some impact energy being absorbed in deforming the plywood.

Failure mode – b: Local failure at the impact zone with excessive concrete crushing below the impactor and yielding of the tensile reinforcements were noticed as shown in **Figs. 2.11(d) and (e)** for beams B1 and B2 respectively. Away from the

impact zone the cracking was similar to that of mode ‘a’ failure. This failure mode was observed in beams impacted directly with the hemispherical impactor.

Failure mode - c: This is similar to mode ‘a’ failure but was accompanied by loss of the concrete cover at the bottom of the beam owing to scabbing as shown in **Figs. 2.11(f) and (g)** for beams B3 and B4 respectively. It was occurred on beams impacted directly with the flat impactor.

Figure 2.11: Post-test crack pattern of beams (a) [A1]; (b) [A2]; (c) [A3]; (d) [B1]; (e) [B2]; (f) [B3] and (g) [B4] (Chen and May [C3])

The beam test results revealed that the supports conditions had less influence on the impact force than the span length. Moreover, this has been noticed that the plywood interface distributed the impact force in a similar manner like a flat impactor.

Saatci and Vecchio [S1] reported a well instrumented experimental program to contribute to the understanding of the effects of shear mechanisms on the behavior of RC beams under impacts loading. Eight specimens were tested under free-falling drop-weights impact loading at their midspan. Two different drop-weights were used for the impact testing: a lighter weight of 211 kg and a heavier weight of 600 kg. The contact surface of the drop-weights were flats and a 50 mm thick, 305mm square steel plate was placed on the top of the beam at impact zone to obtain a well-distributed impact force. The weights were dropped from a clear height of 3.26 m above the specimen, resulting in an 8 m/s impact velocity. For the a-series specimens, the beams were first tested once with the smaller drop-weight, followed by two tests with the larger one. However, for b-series specimens, the order was

reversed; they were tested twice with the larger drop-weight and the one last time with the smaller drop-weight. All the specimens had identical longitudinal reinforcement but varying amount of shear reinforcement ratios, intended to investigate the effects of shear resistance on the impact behavior. **Fig. 2.12** displays the test setup which was specially devised to prevent the uplift of the specimen without creating restrain moment at the supports.

Fig. 2.12: Impact test set up (Saatci and Vecchio [S1])

Crack patterns of a-series and b-series specimens are shown in **Fig. 2.13**. Static test results divulged that SS3 and SS2 exhibited a ductile flexure-critical behavior, whereas, SS1 and SS0 were shear-critical. During impact testing, regardless of their projected static behavior, all specimens developed severe diagonal cracks, originating at the impact point and propagating downward with an angle approximately 45 degrees, forming shear plugs. Additionally, numerous diagonal cracks parallel to shear-plug cracks also formed, along with some vertical flexural cracks at the midspan and the supports. In the flexure-critical beams, shear-plugs developed faster than the formation of support cracks. On the other side, failure of shear-critical specimens occurred due to the development of shear-plug as well as the formation of diagonal cracks from loading point to support point. The shear resistance of flexural-critical specimens was sufficient to carry the shear forces beyond the shear plug to the supports. Subsequent impact loading pressed the shear-plug down further without causing significant damage in the other region. However, shear-critical members did not have enough strength to transmit the shear forces to

the supports. As a result, the shear forces beyond the shear-plug region caused massive damage between the shear-plug and the supports, finally failing the beams in that region.

Fig. 2.13: Final crack pattern of a-series and b-series beams (Saatci and Vecchio [S1])

Shear characteristics of the specimens played a pivotal role in their overall impact response. All specimens, regardless of their shear capacity, developed severe diagonal cracks and shear-plug cracks under the impact point. Specimens with higher shear capacity were able to sustain more impacts and absorb more energy, whereas the ones with lower shear capacity suffered extensive damage under the same or smaller impact loads. Shear mechanism must be considered during the method development in predicting the 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 span length of a beam, are important factors in resisting the impact forces. After gaining some experience from their test program, Saatci and Vecchio provided some recommendations for future experimental studies on impact. To cover wider range of parameters (i.e. impact parameters, such as mass and velocity of impactor; specimen parameters, such as geometry, reinforcement ratios and material properties etc.) that would affect the overall behavior, higher number of specimens should be considered. Measurement of impact responses should be recorded with a higher sampling rate. To measure the displacement during impact test, optical technique should be utilized. Moreover, to better understand the crack formation and propagation mechanisms, high-speed camera

should be employed during testing.

Tachibana *et al.* [T1] documented an investigation on a series of low-speed impact experiments of RC beams with varying span length, cross-section and longitudinal reinforcements. Beams were impacted by a steel-weight which was dropped from a specific height, as shown in **Fig. 2.14**. The steel-weight used in this experiment had a curved contact surface with a length and radius of 565 and 75 mm respectively. Steel-weights having various masses such as 150, 300 and 450 kg were employed in this test program. Static shear to bending resistance ratios for all beams were greater than one; that means flexure-failure will precede shear-failure under static loading conditions.

Fig. 2.14: Impact test set up (Tachibana *et al.* [T1])

Fig. 2.15 shows the crack pattern of type A2 beam after a 300 kg impactor struck the beam with a velocity of 5 m/s. This is a typical crack pattern for flexure-failure with little concrete fragmentation at the impact region.

Fig. 2.15: Crack pattern of type A2 beam (Tachibana *et al.* [T1])

The impulse resulting from impact loading is proportional to the momentum of the impacting mass. Similarly, the duration of impact force is proportional to the ratio of the momentum of impacting mass and static ultimate flexure resistance of beam. Moreover, an equation was proposed (Tachibana *et al.* [T1]) to calculate the maximum mid-span deflection of beam based on impact energy and static ultimate flexure resistance. The proposed formulation is as follows:

$$\delta_{max} = 0.522 \frac{E_{col}}{P_u} \quad (2-36)$$

where δ_{max} is maximum displacement (mm), E_{col} is kinetic energy (J) and P_u is ultimate flexure resistance (kN). The validity of the proposed equation was corroborated by comparing with other experimental results and numerical results from FE simulations, as shown in **Fig. 2.16**.

Fig. 2.16: Proposed equation by Tachibana *et al.* [T1]

The static bending resistance of considered beams ranged from 16.7 to 66.7 kN and the impact energies varied from 150 to 5400 J based on the variation of mass and impact velocity. This has been concluded that the proposed equation to estimate the maximum displacement of beam under impact loading, allows for a performance-

based design of protective structures.

Kishi and Mikami [K10] developed an impact-resistance design methodology for RC beams following performance-based design based on falling-weight impact test results. In total, 36 beams were tested and cross-sectional dimensions, clear span length, amount of longitudinal and shear reinforcement ratios, compressive strength of concrete, yield strength of longitudinal reinforcements, mass and velocity of the drop-hammer were the test variables. All beams had rectangular cross-section, 150 to 250 mm wide, 200 to 400 mm deep and 2 to 3 m clear span length. The amount of longitudinal reinforcements was varied from 0.8 to 3.17% and the shear reinforcements were placed with varying spacing of 100, 125 and 150 mm. For all beams, the static shear to bending resistance ratio was greater than 1, signifying that they were statically flexure-critical. In this study, three different mass of the drop-hammer which were 300, 400 and 500 kg used and impact velocities were ranged from 3.1 to 7.7 m/s. All drop-weights had identical shape of spherical bottom with 1407 mm radius and 2 mm taper. Crack pattern of beams corresponding to series G2L, G5 and G10 is shown in **Fig. 2.17**. 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 mid-span area, mainly below the impact point and these cracks became prominent with increasing impact velocity. The maximum and residual deflections are proportional to the input impact energies and the gradients are empirically formulated using the inverse of static flexural load-carrying capacity of the beam.

Fig. 2.17: Crack patterns in beams of series G2L, G5 and G10 (Kishi and Mikami [K10])

The maximum and residual deflections per unit input impact energy E (coefficient of maximum and residual deflection and static flexural load-carrying capacity P_{usc} are α_{def} and α_{rs}) plotted in **Fig. 2.18**. On the basis of these relationships, empirical design formulae were proposed which are as follows:

$$\alpha_{def} = \frac{0.63}{P_{usc}} \quad (2-37)$$

$$\alpha_{rs} = \frac{0.42}{P_{usc}} \quad (2-38)$$

Rearranging the above two equations, the followings can be obtained

$$P_{usc} = 0.63 \frac{E}{D_{max}} \quad (2-39)$$

$$P_{usc} = 0.42 \frac{E}{\delta_{rs}} \quad (2-40)$$

where P_{usc} is static flexural load-carrying capacity (kN), E is input impact energy (J), D_{max} is maximum displacement and δ_{rs} is the residual displacement (mm).

Fig. 2.18: Regression curves: (a) regression curve between coefficient of maximum deflection α_{def} and static flexural load-carrying capacity P_{usc} ; (b) regression curve between coefficient of residual deflection α_{rs} and static flexural load-carrying capacity P_{usc} by Kishi and Mikami [K10]

These above-mentioned two equations allow for a significant simplification in design approach for structures subjected to impact loading. If the input impact energy and the static flexural load-carrying capacity in both equations are the same, the ratio of the maximum deflection to residual deflection would be 1.5, which could be adjudged as an important design parameter. However, there are some limitations in the application which are: 1) input impact energy $E < 15$ kJ; 2) Eqs. (2-8) and (2-9) are applicable for RC beams having static flexural load-carrying capacity $P_{usc} < 240$ kN and static shear-flexural capacity ratio $\alpha > 1.5$.

A database of RC beams under drop-weight impact loading has been accumulated from the literature (Kishi [K7], Fujikake *et al.* [F4], Bhatti *et al.* [B8], Chen and May [C3], Saatci and Vecchio [S1], Tachibana *et al.* [T1], Kishi and Mikami [K10]). The database containing the particulars and the impact responses of the

specimens (i.e. geometric characteristics of beam, reinforcement ratios, material properties, mass & velocity of the impactor, maximum impact load, maximum mid-span displacements and failure modes etc.) is concisely tabulated in the Appendix B and C.

2.5.2 Numerical and analytical studies

Bhatti *et al.* [B8] demonstrated a simple elasto-plastic finite element (FE) analysis method for simulating the experimental impact behavior. Concrete material model was assumed by using a bilinear model in compression side and a cut-off model in tension side. The steel reinforcing bars were modeled as elasto-plastic with isotropic hardening. Moreover, strain rate effect on concrete and reinforcements was not considered in this study. Finally the numerical results were compared to the experimental outcomes such as impact, reaction force history, mid-span displacement history, impact force vs. mid-span displacement loops and reaction force vs. mid-span displacement loops. Moreover, energy absorption and crack patterns were predicted by this method within a reasonable accuracy.

Saatci and Vecchio [S1] proposed a nonlinear finite element analysis (NLFEA) procedure to numerically simulate the impact responses of RC beam. To express the effectiveness of the procedure, the computed responses (i.e., time histories of mid-span displacements, crack profiles and longitudinal reinforcement strains) were compared to the measured responses from an experimental program. The methodology based on distributed stress field model (DSFM) was successful in predicting the shear-dominant behavior of the specimens under impact loads. However, accurate prediction of test specimens was obtained in this research by considering strain rate dependent material properties; thus, this issue needs further investigation.

Fujikake *et al.* [F4] modeled the RC beam under drop-hammer impact loading as two-degree-of-freedom mass-spring-damper system. This model could represent not only global response of the beam but also the local response at the contact point between the drop-hammer and beam. Here, the impact scenario has been presumed as a perfectly plastic collision. The spring property corresponding to the overall

response of the beam was determined from the loading-rate dependent load vs. mid-span deflection relationships whereas the contact spring stiffness value was assumed from Hertz's contact theory. The global damping coefficient was considered to be zero and contact damping coefficient was assumed to be one-half of the critical damping coefficients. The analytical results in terms of impact force history and mid-span deflection history were in good agreement with the experimental results when the beams predominantly failed in flexure. However, large discrepancies were observed in mid-span deflection responses when both local failure at impact point and flexure failure occurred.

2.6 Summary

Based on literature review, some conclusions can be summarized as follows:

- (1) Review of literature related on the performance of RC beams under varying loading rates reveals a paucity of works in this field. A limited number of tests on beams show that the increased flexural strength due to the increase in yield strength of steel reinforcement is a primary reason for change in failure mode of beams subjected to loading rate expected in an earthquake event. This change in failure mode phenomenon is more pronounced in beams of intermediate slenderness. Moreover, investigation on effects of loading rates corresponds to low velocity impact on RC beam is found to be inadequate in literature. Thus, a systematic test program on intermediate-slender beams subjected to wide spectrum of concentrated loading rates at their mid-span is worthy of investigation.
- (2) There is a lack of numerical research on identifying the effects of various parameters on the behavior of beams under a wide range of varying loading rates. Therefore, numerical model (three-dimensional) is essential to validate the experimental results first and further utilized to expand the knowledge beyond the range of parameters investigated experimentally.
- (3) Assessment of the behavior of RC beams under drop-weight impact loading has been well documented in the literature; however, evaluation of the residual performance of impact-damaged beams is a lacking area. To contribute towards a better understanding in this area, an organized experimental program (i.e. impact

tests followed by quasi-static tests) is essential.

(4) Due to the scarcity of test results, it is obvious that there will no numerical research on residual performance of impact-damaged beams. Therefore, development of three-dimensional FE model to numerically simulate these two-stage experiments is valuable to examine.

CHAPTER 3

EXPERIMENTAL INVESTIGATION OF LOADING RATE EFFECTS ON RC BEAMS

3.1 Introduction

Over the past few decades, there has been a growing interest among the researchers and designers in analyzing and designing RC structures against extreme loads, such as earthquakes, blasts and impact. These loading scenarios induce varying strain rates in the structural elements. In this study, the strain-rate effect on RC beams falls in the wider range of static to hard impact (10^{-4} to 10/s) regime. A variety of devices such as hydraulic machines, pneumatic machines and impact systems (mass swings or falls from specific heights to strike a specimen) have been constructed to perform high loading rate tests on structural elements in laboratory. Previous studies (Mutsuyoshi and Machida [M11] and Kulkarni and Shah [K13]) on high loading rates of RC beams were confined up to the earthquake loading regime. Fujikake *et al.* [F4] considered a much higher piston velocity for high loading rates which could be linked to the impact loading regime; however consider only one test parameter (i.e. flexural reinforcements ratio). There is a lack of systematic research on RC beams under wide range of varying loading rates to cover static loading to hard impact regime. Therefore, a well-instrumented experimental program was initiated to investigate the effects of varying loading rates on the structural behavior of intermediate slender RC beams with limited amount of flexural reinforcements (approximately $0.3\rho_b$ where ρ_b refers to balanced reinforcement ratio in percentage). In this study, hydraulic machine was used to generate the wide range of loading rates extending from 4×10^{-4} m/s to 2×10^0 m/s. Corresponding strain rate induced in the reinforcements of RC beams were ranging from 10^{-4} /s to 10/s.

3.2 Analysis of Experimental Database

A database of RC beams under different loading rates has been assembled from the literature to conduct the parametric studies with an objective of gaining some insight on the influence of various parameters such as shear span to effective depth ratio (a/d), longitudinal reinforcement ratios (ρ_L), shear reinforcement ratios (ρ_T) and failure modes on the dynamic increase factor (DIF) of maximum resistance. Due to the scarcity of data, it was very difficult to compare the data which could be grouped in such a way that they would present respective variable as the only variable, i.e. beam with identical properties except varying a/d . The database containing the particulars of the specimens is tabulated in the Appendix-A.

Most of the RC beams shown in the Appendix-A, can be classified into singly RC beams without shear reinforcement (SR) and doubly RC beams with shear reinforcement (DR). Thus, parametric study was carried out for SR and DR beams separately to determine the effect of several parameters on their corresponding DIF of maximum resistance. **Fig. 3.1 (a)** shows the influence of shear span to effective depth ratio on the trend of variation of DIF of SR beams. Effects of longitudinal reinforcement ratios and failure mode on DIF under different loading rates are shown in **Figs. 3.1 (b) and (c)**. Due to scattering of data points, it is very tough to ascertain the effects of single parameters on DIF of SR beams. Failure modes (shear or flexure) and the time of yielding of tensile reinforcements (before or after the peak resistance of shear critical beams) have some significant effect on DIF. In **Fig. 3.1 (c)** for 0.1 to 1 m/s, few DIF are higher than the clustering data points. In these cases, yielding of tensile reinforcements occurred after their peak resistance. Similarly, influence of shear span to effective depth ratio (a/d), longitudinal reinforcement ratios (ρ_L) and shear reinforcement ratios (ρ_T) and failure mode on DIF of DR beams are shown in **Figs. 3.2 (a), (b), (c) and (d)** respectively. From the trend of DIF, it can be concluded that beams with lower a/d ratios (1.9) have higher DIF under varying loading rates as compared to slender beams (a/d -3.3 and 4.4). Beams of low shear reinforcement had a higher DIF value as compared to other specimens.

Fig. 3.1: Influence of (a) shear span to effective depth ratio (a/d); (b) longitudinal reinforcement ratios (ρ_L); (c) failure modes on DIF of singly reinforced (SR) beams

When the specimen contains less shear reinforcements, it may fail in shear where the tensile strength of concrete (high rate sensitive than compressive strength) plays a significant role in increasing the ultimate resistance under high loading rates and correspondingly increases the DIF. Finally based on the comparative plot of DIF vs. failure modes, it can be concluded that shear critical beams produced higher DIF as compared to flexure critical for all loading rates.

Fig. 3.2: Influence of (a) shear span to effective depth ratio (a/d); (b) longitudinal reinforcement ratios (ρ_L); (c) shear reinforcement ratios (ρ_T); (d) failure modes on DIF of doubly reinforced (DR) beams

The review of existing literature revealed variety of high loading rate tests and it is very difficult to make effective comparisons due to the involvement of numerous parameters at a time. As such, it will be beneficial if the researchers can agree on a preferred way to perform high loading rate testing of RC beams. There are some

major differences in the testing methods worth notable. At first, the test setup may be designed for 3- or 4-point bending depending on the objective of the test program. For high loading rate testing using hydraulic machines, 3-point configurations were recommended by Kulkarni and Shah [K13] from the point of view of reducing inertial effects. Furthermore, attempt should be made to test identical specimens (i.e., similarities not only in structural level but also in material level, such as aggregate size, compressive strength of concrete and yield strength of reinforcements) using different test machines. Time to reach peak displacement in loading history (for a particular loading rate in displacement control) during experiment should be kept nearly unvarying. The data acquisition system (particularly the frequency response characteristics) should be nearly identical to facilitate better analysis and comparisons. In addition, the type of anchorage (i.e., by nuts, welded with plates or development length by hooking) at the end of flexural reinforcements has a profound effect on the shear failure mechanism of beams (which will be governed by arch or beam action) under varying loading rates and should as such be kept identical across the tests.

3.3 Objective of Test Program

Review of literature in the field of RC beam under varying loading rates reveals that there is a need to assess the behavior of beam under wide range of varying loading rates. The primary objective of this research is to study the loading rate effect on the load vs. mid-span displacement relationships, concrete contribution to the shear stress, and peak strain rate at longitudinal tensile and shear reinforcements. Moreover, Dynamic increase factors (DIF) of maximum resistance, cracking stiffness, energy-absorption and failure mechanism of beams are also discussed by comparative analysis. In addition, this test program will supplement the literature with detailed test data that can assist further in the development and verification of numerical models.

3.4 Experimental Overview

Till date, limited research has been attributed to explore the behavior of RC beams under wide range of loading rates. Meticulous inspection of the database provided in the appendix reveals that the behavior of RC beams with intermediate slenderness under varying loading rates is still deficient. With an intention of contributing to that scarce research area, an experimental program was undertaken for RC beams with limited amount of flexural reinforcements ($0.31\rho_b$ and $0.3\rho_b$ for 3.3 and 4.4 series respectively; where ρ_b refers to balanced reinforcement ratio in percentage) subjected to a wide range of loading rates (Static: 4×10^{-4} , Low: 4×10^{-2} , Medium: 4×10^{-1} and High: 2×10^0 m/s). Comparative analyses of specimens under these varying loading rates highlighted several key aspects of their dynamic behavior.

The experimental program consisted of 24 RC beams which were divided into two groups in terms of their shear span-to-effective depth ratios (a/d -3.3 and 4.4). Each group had three types of specimens distinguished by their shear reinforcement ratio. Each type had four identical specimens subjected to four types of varying rates of concentrated loading at their mid-span. The entire test program was summarized in the form of a flow chart as shown in **Fig. 3.3**. It is noted that all test

specimens have been designed and cast at NTU, Singapore however; the testing was conducted at NDA, Japan. The specimen denoted by SR3.3_0.84 corresponds to ‘singly-reinforced’ beam (SR) followed by the shear span-to effective depth ratio and percentage of longitudinal tensile reinforcement ratios. Again, DR3.3_0.84_0.11 refers to ‘doubly-reinforced’ beam (DR) followed by the shear span-to effective depth ratio, percentage of longitudinal tensile and compressive reinforcement ratio and percentage of shear reinforcement ratio.

Fig. 3.3: Flow chart of test program

The layout of the longitudinal reinforcements, spacing of shear reinforcements and the measuring points (acceleration and strain for steel reinforcing bars) are shown in **Fig. 3.4**. Here, ‘SA’ refers to the strain gauges attached at the midspan of longitudinal tensile reinforcements (e.g. two strain gauges for two bottom tensile reinforcements) where ‘SS’ refers to the strain gauges attached to transverse reinforcements (e.g. one strain gauge at one leg of stirrups). Longitudinal reinforcements consisted of deformed steel bars of T13 (diameter-13mm) and T10 (diameter-10 mm) for a/d -3.3 and 4.4 series specimen respectively while R6 (diameter- 6 mm) plain round bars were used as shear reinforcements for all

specimens. Target concrete compressive strength at 28 days was 40 MPa with maximum aggregate size of 10 mm for all specimens. The yield strengths of T13, T10 and R6 were 520, 510 and 310 MPa respectively. All specimens were cast at the same time using ready mix concrete of same batch; hence, all specimens had an identical concrete strength. The compressive strength at 28 days obtained from three concrete cylinders was 39.0, 39.6 and 42.2 MPa respectively. The specimen details, theoretical static capacities, static expected and observed failure modes are listed in **Table 3.1**, where the static flexural and shear resistances are calculated according to ACI 318-08 [A2]. According to this table, except SR3.3_0.84, all specimens are theoretically flexure-dominant (shear to bending resistance ratio is greater than one) under monotonically increasing static loads. However, for DR3.3 series shear failure mode (e.g. Shear*1 and Shear*2) was observed under static loading. Shear cracks were developed just after the yielding of tensile reinforcements and after the crushing of compression concrete for the case of shear*1 and shear*2 respectively. Thus, this type of failure could be visualized as mixed type failure mode. The reason of inconsistency between the expected and the observed failure modes in DR3.3 specimens is the wider interval of the stirrups. The maximum spacing of vertical stirrups as the smaller of $d/2$ or 610 mm is set by ACI 318-08 [A2]. Following ACI Code, the vertical stirrups surely intersect at a diagonal shear crack formed in the RC beam. Thus, stirrups were not effective at all in preventing shear failure.

Fig. 3.4: Dimensions of RC beams, layout of reinforcements, location of strain gauges and accelerometers

Table 3.1: Specimen details, theoretical static capacities, static expected and observed failure modes

Specimens	a/d	Longitudinal rebar ratio (%)	Shear rebar ratio (%)	Flexural resistance (kN)	Shear resistance (kN)	Shear to bending resistance ratio	Expected failure modes	Observed failure modes
SR3.3_0.84	3.3	0.84 ^T	0.00	77.1	66.2	0.86	Shear	Shear*1
DR3.3_0.84_0.11		0.84 ^{T&C}	0.11	77.6	87.1	1.12	Flexure	Shear*2
DR3.3_0.84_0.15		0.84 ^{T&C}	0.15	77.6	95.5	1.23	Flexure	Shear*1
SR4.4_0.82	4.4	0.82 ^T	0.00	33.7	40.2	1.19	Flexure	Flexure
DR4.4_0.82_0.13		0.82 ^{T&C}	0.13	34.0	56.1	1.65	Flexure	Flexure
DR4.4_0.82_0.19		0.82 ^{T&C}	0.19	34.0	62.5	1.83	Flexure	Flexure

Shear*1=Shear failure following yielding tensile reinforcements

Shear*2=Shear failure following flexure compression failure

Note:- T-only tensile reinforcements; T&C- Tensile and compressive reinforcements

To construct the specimens, steel formwork was used. Before the placement of reinforcing cages, oiling was carried out on the inside surfaces of formwork so that specimens can be removed easily and the surface of the specimens would not

damage. Before casting, concrete spacer blocks were placed in the underside as well as the side faces of transverse reinforcements to achieve clear concrete cover. Lifting hooks were mounted on both sides of the specimens to facilitate lifting process during testing. **Fig. 3.5** shows the photos of the casting of beams and the preparation of concrete cylinders (150mm×300 mm). After casting, damp hessian fabrics were employed to cure the specimens for two weeks.

Fig. 3.5: (a) Casting of beams; (b) preparation of concrete cylinders

All specimens were well instrumented to capture the loads, displacements, accelerations and the strains of steel reinforcement. The instrumentation for this test program included a load cell, strain gauges and accelerometers. Strain gauges with 3 mm gauge length were installed in the mid-span of the longitudinal tensile reinforcements and in the mid-point of the two legs of the shear reinforcements to measure the local strain. These are general purpose foil strain gauges (KFG types) with resistance of 120Ω and linear expansion coefficient of $11 \times 10^{-6}/^{\circ}\text{C}$. Soldering to the terminal of the gauges was not required due to the pre-attachment of three parallel vinyl-coated lead wires with 10m long. Before the installation of strain gauges at the specific locations, the reinforcement surfaces were grinded lightly and cleaned with alkaline and acidic chemicals. **Fig. 3.6** shows the photos the glued strain gauges in longitudinal tensile and transverse reinforcements. Although in **Fig. 3.6**, the strain gauge is shown in top face of the tensile reinforcements however, the reinforcement cages were inverted during the placement into the moulds; then the gauges would be in the bottom face of the tensile reinforcements. A load cell of

capacity of 980 kN and measuring frequency of 5 kHz was attached to the actuator to measure the load. Five accelerometers (capacity of 1000 times gravity and resonance frequency greater than 70 kHz) were mounted on the specimens for each test (except for static loading) to measure the accelerations for low, medium and high rates of concentrated loading. The midspan deflection of RC beams was measured by laser-type variable displacement transducers (LVDTs) which have a measuring range of 80 mm and sampling rate of 50 kHz. Data from the sensors were collected by a digital data acquisition system which has a sampling rate of 100 Hz, 10 kHz, 100 kHz, and 200 kHz for static, low, medium and high loading rates respectively. A steel plate having 40 mm thickness was placed on the top of the beam in between loading bar and specimen to transfer well-distributed force to the specimen. The use of digital photography and high-speed video recording proved to be valuable in providing insights into the cracking patterns and failure modes. A schematic diagram of three point bending test setup is shown in **Fig. 3.7**. All experiments were conducted in displacement control as this is known to be indispensable for obtaining the entire post-peak response of structural elements. Moreover, in varying loading rates testing, displacement control is even effective to reduce the inertia effects during the pre-peak stage.

Fig. 3.6: (a) Strain gauging in tensile longitudinal reinforcements; (b) Strain gauging in transverse reinforcement

Fig. 3.7: Schematic diagram of test set up

3.5 Experimental Results and Discussions

Following the experimental overview sections which consist of specimen design, material properties, test-setup and instrumentation; experimental results and major observations are comprehensively presented and discussed in this section.

3.5.1 Load-midspan displacement relationships

The input load by the actuator was measured by the load cell attached to it and the reaction forces were measured by two load cells placed at the two pin supports of the beams. For static, low and medium loading rates, almost identical responses (i.e. load-midspan deflection) were obtained from the top and two bottom load cells. However, for high loading rates, higher response was obtained from two bottom load cells as compared to the top load cell. The 100 kg mass of each supporting apparatus attached to each support load cell might induce additional responses. Besides, the wider range of frequency of the load acting on the lower load cells as compared to its natural frequency could be the another reason. So, the reaction force (summation of two bottom load cells) obtained from high loading rates might

overestimate the true resistance of the beam as shown in **Fig. 3.8**. This figure displays the typical variation of force history obtained from top and bottom load-cells under high loading rates. Moreover, there is a time lag (2.98 ms) in between the two responses which is clearly visible in the figure. This is due to the propagation time required of stress wave to travel from loading point to support region.

Fig. 3.8: Typical variation of force history from top and bottom load-cells under high loading rates

Thus, inertia force should be deducted from the top load cell data (i.e., using following equation) by considering the beam as a SDOF system to evaluate its true resistance.

$$R_b(t) = P_{TL}(t) - M_b \times \ddot{x}_b(t) \quad (3-1)$$

where, $R_b(t)$: true resistance of RC beam without inertia force; $P_{TL}(t)$: load measured by top load-cell; M_b : equivalent mass of RC beam (i.e. $17\rho A l_{clear} / 35$; ρ =density of RC beam; A =cross-sectional area of beam; l_{clear} = span length in between supports); $\ddot{x}_b(t)$: acceleration at midspan obtained from accelerometers; moreover, another 25 kg extra mass (considering the mass of the loading bar) has been added to the equivalent mass of the beam to calculate the true resistance. Banthia *et al.* [B2] proposed two approaches (i.e. linear for plain and sinusoidal for conventionally reinforced beams) to calculate the inertia force based on the

distribution of accelerations along the beam. Efforts have been made to compare the current methodology and the approaches provided in literature. **Fig. 3.9** shows the comparative plot of true resistance history obtained from Eqtn. 3-1 and approaches recommended by Banthia *et al.* [B2]. From this figure, it can be concluded that true resistance history obtained from current approach is almost similar to the sinusoidal case coined by Banthia *et al.* [B2]

Fig. 3.9: Comparative plot of true resistance history obtained from Eqtn. 3-1 and approaches proposed by Banthia *et al.* [B2]

The comparisons of the load-midspan displacement relationship of each specimen to the corresponding loading rates are illustrated in **Fig. 3.10**. Due to some unforeseen problem (i.e., fault in hydraulic machine), the response of the beam naming DR4.4_0.82_0.19 under low loading rates could not be captured and hence, the corresponding beam response has been omitted. Load-carrying capacity of all specimens increases with incremental loading rates. Dynamic increase factors (DIF) of a/d -3.3 and 4.4 series specimens are shown in **Figs. 3.11 (a) and (b)** respectively. DIF is calculated as a ratio of maximum resistance of beam at any loading rates (i.e. low, medium and high) to the corresponding static maximum resistance at a loading rate of 4×10^{-4} m/s. From the comparative analysis of the above-mentioned two figures, DIF for 3.3 series seems to be higher than 4.4 series due to their contrast nature of failure mode. It is noteworthy that DR beams had less DIF as compared to SR; however for DR3.3_0.84_0.15, resistance for low loading rate has been used to calculate DIF. As there is no sign of ductile behavior with strain hardening (i.e. sudden drop in load-deformation curves after peak load

although the longitudinal tensile reinforcements yielded before the beam attained peak resistance) in the load vs. mid-span deflection curves of DR3.3_0.84_0.15, thus the peak resistance obtained from testing under static loading would overestimate the DIF.

Fig. 3.10: Load vs. mid-span deflection of (a) a/d -3.3; (b) a/d -4.4 series specimens under varying loading rates (continued)

Fig. 3.10: Load vs. mid-span deflection of (a) a/d -3.3; (b) a/d -4.4 series specimens under varying loading rates

Fig. 3.11: Dynamic Increase Factor (DIF) of maximum resistance of (a) a/d -3.3; (b) a/d -4.4 series specimens under varying loading rates

3.5.2 Concrete contribution (v_c) to the shear stress

For evaluating the concrete contribution to the shear resistance, only SR beams were considered and collected from the published database and the current testing data. ACI318-08 [A2] only provides the coefficients of concrete contribution to shear stress normalized by the square root of concrete compressive strength (i.e. 0.17 for SI unit system) under static loading only. ACI methodology of concrete contribution to shear stress for static loading has been extending further to incorporate the loading rate effects on the same. The testing shear resistance values obtained from varying loading rates are normalized by the square root of concrete compressive strength, the width and the effective depth of beams. **Fig. 3.12** displays the variation (e.g. increasing trend under high rates) of concrete contribution to shear stress normalized by the square root of concrete compressive strength ($v_c / \sqrt{f'_c}$) under different loading rate ($\dot{\delta}$). Few erratic data was observed in the data provided by Mutsuyoshi and Machida [M10]. Yielding of tensile reinforcement after the shear failure of the beam could be the reason behind this inconsistent behavior.

Fig. 3.12: Concrete contribution to the shear stress

3.5.3 Concrete (V_c) and shear reinforcement (V_s) contribution to shear resistance

Fig. 3.13 shows the concrete and shear reinforcement contribution to shear strength of DR beams of a/d -3.3 series, as the specimens of this series failed in shear irrespective of loading rates. Some crude estimation has been done to calculate the corresponding contribution to shear resistance. Concrete contribution has been taken from SR beams of the same series as shear force was solely resisted by concrete. To account for the stirrup contribution, shear resistance of the SR beams has been deducted from the shear resistance of DR beams of same series by neglecting the contribution of compression reinforcement to shear strength. From **Fig. 3.13**, it is evident that the shear reinforcement contribution as compared to concrete contribution is quite small under these four varying loading rates.

Fig. 3.13: Concrete and shear reinforcement contribution to shear resistance

3.5.4 Cracking stiffness

Cracking stiffness of the beams was calculated by using the secant of the load vs. midspan deflection curve passing through the point from which the curve changes its slope. Then, it was simply computed by taking the quotient of the resistance and the deflection of that corresponding point. **Fig. 3.14** displays the variation of cracking stiffness of the two series (i.e. a/d -3.3 and 4.4) of specimens. Enhancement in cracking stiffness of a/d -3.3 series specimens was observed when

the loading rate increased from static to low and medium rates. Similarly, for a/d - 4.4 series specimens, stiffness increases as the loading rate progresses from static to medium. However, the above-mentioned figure lacks the stiffness under high rates as the load vs. midspan deformation curves was obtained in this case by deducting the inertia force from the top load cell (described in section 3.5.1). Due to the time-lag between the top and bottom load-cell signals, the load vs. midspan deformation response obtained from the bottom load cells under high rates may not be similar to the same from the top load cell. As a result, the computed cracking stiffness for high rates is not compared with other loading rates. The reason behind this time-lag could be due to the inertia forces, as the initial force applied is solely resisted by the inertia of the beam before the bottom load-cell attained any load.

Fig. 3.14: Cracking stiffness of (a) a/d -3.3; (b) a/d -4.4 series beams under varying loading rates

3.5.5 Energy absorption

To calculate the amount of energy absorption of the specimens under these varying loading rates, the area under the load vs. midspan deflection curves needs to be calculated. Before that, the generalized mid-span displacement range should be defined to impose the same conditions for all the specimens. The deformation range was defined up to peak load displacement (for shear-failure type beams) or three times of the yield deformation (for flexure-failure type beams), solely depending on

the characterization of their load vs. midspan deflection curves. However, for medium and high loading rate, the load-midspan deformation curves did not displayed the ‘yield plateau’. For these cases, peak load displacement has been taken as benchmark point. **Fig. 3.15** clearly illustrates the variation of the energy absorption capacity of RC beams for static, low, medium and high loading rates. The energy absorption capacity of the RC beams increased with the enhancement of loading rates from static to high.

Fig. 3.15: Energy absorption of (a) a/d -3.3; (b) a/d -4.4 series beams

3.5.6 Comparison of time to reach peak load, yielding of tensile and shear reinforcements & peak strain rate

Table 3.2 represents the comparison between the time to reach peak load and yielding of tensile and shear reinforcements for 3.3 series specimens as these series specimens failed in shear at their ultimate resistance. The term ‘Average’ in **Table 3.2** refers to the mean value of two strain gauge (e.g., SA1,2 and SS1,2) readings. It is worthwhile to know that whether the longitudinal and shear reinforcements yielded before or after attainment of the peak resistance. From this table, it is clear that except static loading, the tensile reinforcements yielded at the very early stages for all loading rates. When the tensile reinforcements yielded before the shear failure, the DIF would be less than the case of yielding of tensile reinforcements after shear failure. However, only few cases were seen where shear reinforcement yielded before shear failure or not yielded at all. Due to the larger spacing of shear

reinforcements, diagonal shear cracks did not intersect (i.e. propagated in between stirrups) them. Additionally, strain rate was calculated by differentiating the strain history data of longitudinal and shear reinforcements. To define the materials (e.g. concrete and steel) behavior, most of the formulations were developed for certain strain rate intervals; therefore, it is essential to determine the range of strain rates.

Table 3.2: Comparison between the time to reach peak load and the yielding of tensile and shear reinforcements

Specimens	Time	Static (s)	Low (s)	Medium (ms)	High (ms)
SR3.3_0.84	T_{pl}	362.5	0.22	20.0	3.30
DR3.3_0.84_0.11		181.4	0.50	34.0	5.70
DR3.3_0.84_0.15		188.4	0.57	36.0	5.40
		SA1,2	SA1,2	SA1,2	SA1,2
		Average			
SR3.3_0.84	T_{ylt}	359.3	0.125	11.4	1.84
DR3.3_0.84_0.11		174.9	0.084	12.2	1.49
DR3.3_0.84_0.15		183.6	0.095	11.6	1.68
		SS1,2	SS1,2	SS1,2	SS1,2
		Average			
DR3.3_0.84_0.11	T_{ys}	152.3	0.32	NY	4.20
DR3.3_0.84_0.15		NY	NY	NY	1.21
Note:- T_{pl} -Time to reach peak load; T_{ylt} -Time to yield longitudinal tensile reinforcement; T_{ys} -Time to yield shear reinforcement; NY-Not Yielded

Fig. 3.16: Peak strain rates of (a) longitudinal (b) shear reinforcements for all specimens

Thus, the variations of peak strain rates (i.e. choosing the maximum value from the differentiation of strain history data) over these loading rates for longitudinal and shear reinforcements are indicated in **Figs. 3.16 (a) and (b)** respectively.

3.5.7 Deflected shapes

As mentioned previously, accelerometers were placed to the test specimens along the entire length on their top face in between the support points. Assuming zero displacement at support, the displacement time history of each point could be determined by double integrating the acceleration data.

Fig. 3.17: Deflected shape of (a) a/d -3.3; (b) a/d -4.4 series specimens under high loading rates

For a particular time, these data points are plotted and joined together by smoothened lines to get the deflected shape of the beams. **Fig. 3.17** displays the deformed shapes of the beams in 2.5 ms interval for high loading rates. Due to the damage of some data (i.e., data from accelerometers), two types of specimens from each series are presented here. These figures distinctly show the one-sided shear failure of a/d -3.3 series specimens and flexure-type failure of a/d -4.4 series specimens. So, the failure pattern can be observed in the deflected shapes of the beams in any time of the loading history. However, to get more precise deflected shape of beams, closely-placed accelerometers need to be installed.

3.5.8 Crack patterns

Fig. 3.18 shows the crack patterns of all specimens except DR4.4_0.82_0.19_L under these loading rates. SR beams of 3.3 series exhibited brittle shear-critical behavior irrespective of loading rates. Diagonal shear cracks originated from the mid-height of beam and propagated towards loading and support points for all loading rates. However, for DR specimens, diagonal shear cracks under medium and high loading rates, propagated with much steeper angles towards the bottom surface of the beam as compared to static and low loading rates. In these cases, diagonal cracks under medium and high loading rates tried to punch the mid-part of the specimens towards bottom surface of the beam at steep angles as the shear reinforcements gave resistance to transmit these cracks towards supports. In general, for 3.3 series of specimens, diagonal tension and/or shear tension type failure was observed in SR beams whereas diagonal tension and/or shear compression were the dominant failure mode for DR specimens. It was observed that all 3.3 series specimens failed in shear at their peak resistance, however measurement of steel strain data exhibited the yielding of tensile reinforcements before peak load or shear failure occurred for low, medium and high loading rates. Development and propagation of cracks around different points (yielding of tensile reinforcement, first hairline shear crack, peak load etc.) in load vs. time curves were portrayed in **Fig. 3.19** for SR3.3_0.84 under high loading rates. Similarly, development and propagation of cracks around different points in the load vs. time curves were presented in **Fig. 3.20** and **Fig 3.21** for DR3.3_0.84_0.11 and

DR3.3_0.84_0.15 respectively, under high loading rates. Two explanations could be used to justify the shear failure of DR3.3 series specimens under high loading rate. One is the wider spacing of web reinforcements and the other one is the increment of flexural resistance of beams under low, medium and high loading rates due to strain rate sensitivity of yield stress of tensile reinforcements.

Fig. 3.18: Crack patterns of (a) a/d -3.3; (b) a/d -4.4 series specimens under varying loading rates

Considering the DIF of yield stress of tensile reinforcements, it was calculated that flexural resistance boosted around 18.5, 21.3 and 24.8% for the above-mentioned loading rates as compare to assuming static yield stress for all rates. However, strain rate sensitivity of yield stress of stirrups did not contribute much (in majority of cases stirrups did not yield) in their shear resistance. As a result, the shear to bending resistance ratios become in the range of 0.9-1 which was 1.1-1.2 at the initial design stage (considering static yield stress).

Fig. 3.19: Development of cracks in SR3.3_0.84 under high loading rates

For 4.4 series specimens, ductile flexure type failure with some spalling and crushing of concrete below the loading plate was observed for all loading rates. In

these cases, flexural cracks usually started from the bottom surface of the beam and propagated vertically upwards and successively crushed the compression concrete.

Fig. 3.20: Development of cracks in DR3.3_0.84_0.11 under high loading rates

Fig. 3.21: Development of cracks in DR3.3_0.84_0.15 under high loading rates

3.6 Summary

This chapter presents the experimental investigation on RC beams under a wide spectrum of loading rates. Based on the observation and analyses of experimental results, the following conclusions can be drawn:

(1) With increasing loading rates, the ultimate load resistance, cracking stiffness and energy absorption of RC beams were found to increase. When the loading rate progressed from low to high, an increasing trend in DIF was observed. In addition, specimens having smaller a/d ratio produced higher DIF as compared to the specimens having higher a/d ratio for low, medium and high loading rates. For instance, 3.3 series specimens had higher DIF as compared to 4.4 series. Moreover, for low, medium and high loading rates if the tensile reinforcements yielded before the shear failure, the DIF would be lesser than the case where yielding of tensile reinforcements was followed by the shear failure. For the first case, strain rate sensitivity of yield stress of reinforcements controls the magnitude of DIF whereas strain rate sensitivity of the tensile strength of concrete controls the magnitude of DIF for the second case. As, the tensile strength of concrete is more rate sensitive than yield stress of reinforcements, thus the DIF is higher for second case.

(2) High loading rates seem to have beneficial effect on shear resistance as concrete contribution increases with increasing loading rates. Analysis response revealed that compared to shear reinforcement contribution, concrete contribution governs in shear resistance contributions under these loading environments. Moreover, peak strain rate was amplified in one order of magnitude (approximately, 10 times) as the loading rates progressed from low to high.

(3) For 3.3 series specimens, diagonal tension and/or shear tension type failure was observed in SR beams whereas diagonal tension and/or shear compression were the dominant failure mode for DR specimens. In all loading rates of the above-mentioned series, diagonal shear cracks originated from the mid-height of beam and propagated towards loading and support points. However, for DR specimens, diagonal shear cracks under medium and high loading rates propagated with much steeper angles towards the bottom surface of the beam as compared to static and

low loading rates. On the other hand, for 4.4 series specimens, ductile flexure type failure was observed for all loading rates.

(4) After observing the wide scatter of results, it is being realized that there is a great need of consensus among researchers in high-rate testing of RC beams. Test set up, frequency response of measurement system, beam geometry, reinforcing detailing and material properties should be kept nearly identical to facilitate better analysis and effective comparisons.

CHAPTER 4

NUMERICAL SIMULATION OF LOADING RATE EFFECTS ON RC BEAMS

4.1 Introduction

From the literature review and experimental investigation chapter on loading rate effects of RC beams, it is quite evident that research in this field in terms of experimental and numerical means is quite limited. Although the experimental investigation presented in the previous chapter can provide significant information about the tested specimens; however it could be a time-consuming and costly option. Therefore, it is worthwhile to develop reliable numerical model that can serve as an alternative to experimental studies. The objective of this chapter is to numerically predict the experimental behavior (e.g. load-mid-span deflection characteristics and failure modes etc.) of beams under varying loading rates to investigate the potentiality of explicit finite element (FE) program. Adhikary *et al.* [A3] developed three-dimensional non-linear FE model to analyze the structural response of beams under these loading conditions. Although this FE model can capture the structural response up to medium loading rate within reasonable accuracy; however for the case of high loading rates, the prediction deviates from the test results. Moreover, they recommended for a numerical parametric study to investigate the influence of various grade of steel reinforcement and concrete. Some researchers suggested for the investigation of the effects of longitudinal reinforcement ratio. It could be of some interest to observe the behavior mainly the failure mode when the longitudinal reinforcement is increased in an under-reinforced beam with intermediate-slenderness. Thus, to expand the knowledge beyond the range of parameters investigated experimentally and their effects on the structural response, three-dimensional nonlinear FE model is developed. LS-DYNA [H1] was employed in this study because of its proven effectiveness in geometric modeling and analysis capability for high rates of concentrated loading. However, prior to trusting the results of any numerical model, it is of paramount interest to validate the numerical results with the test results. Therefore, the proposed FE

model is validated by the test results and then employed further to conduct parametric studies.

4.2 Finite Element Modeling Techniques of Reinforced Concrete

Finite element models of RC structures have generally been based on replacing the composite continuum by an assembly of elements representing the concrete and the steel reinforcement. Three alternative approaches are mainly used for modeling reinforcement in a three-dimensional finite element model of a concrete structure: the discrete model, the embedded model, and the smeared model (El-Mezaini and Citipitiouglu [E1]). In discrete model, reinforcement is modeled using beam element connected to the nodes of concrete mesh. In this model, perfect bond is assumed in between the concrete and reinforcement. Hence, concrete and reinforcement mesh share the common nodes. This model is simple in concept and it has advantage of representing different material properties more precisely. The only shortcoming of using the discrete model is that concrete mesh patterns are restricted by the location of reinforcement. To overcome the problem of mesh dependency in discrete model, the embedded model could be used where the concrete mesh could be chosen independently. The embedded formulation is based on the idea of assessing the stiffness of reinforcing elements individually from the concrete elements. The strain compatibility is managed by an assumed bond-slip relationship. To model the bond-slip effects, reinforcing bars are modeled using special elements connected to concrete through fictitious springs. However, the additional nodes requirement for the reinforcement increases the total number of degrees of freedom (DOF) and the computational efforts. In the smeared formulation, the reinforcing bars are assumed to be uniformly distributed over the concrete elements. Assuming perfect bond in between concrete and reinforcement, the constitutive relations of the elements are derived from the composite theory. In large structural model where the reinforcement details are not so critical on overall response, this model could be employed.

4.3 Finite Element Model

The nonlinear explicit FE program LS-DYNA [L1] was employed in the numerical simulation process. The advantage of using explicit method is that there is no need to calculate stiffness and mass matrices for the entire system. The solution can be carried out at the element level and thus, relatively less storage is required as compared to implicit method. However, explicit methods are conditionally stable and thus small time step would be needed. To make the computation stable, the selection of time step should be such that it is less than the time required for a stress wave to travel through the shortest element of a FE model. Hence, excessive computational time will be needed when the level of discretization increases. The description of modeling includes structural geometry of beam, loading plate, support roller, boundary conditions, application of load, and relevant material properties.

4.3.1 Structural configuration

To simulate the same experimental conditions, it is essential to develop a FE model as close to the physical system as possible. Therefore, a three-dimensional model is the obvious choice. The three-dimensional FE mesh was developed which consist of solid elements to represent concrete, support rollers and loading plate and beam elements to characterize longitudinal and transverse reinforcements. Different mesh sizes have been considered to study mesh sensitivity during the initial stage of numerical study. Here, a mesh size of 15 mm was used to create the solid elements which seemed to be ideal as further decrease in mesh size almost generated the same results however augmented the computational time. **Table 4.1** presents total number of elements and nodes for each beam model. **Fig. 4.1** shows the three-dimensional FE model of beams (i.e. one type from each series), support rollers and loading plate. Moreover, **Fig. 4.1** also displays the schematic of the reinforcing cages of the beam. Eight node solid elements with a single integration point were used to represent concrete in the models. One of the disadvantages of single-point integration is the appearance of zero energy modes which are also called hourglass modes. LS-DYNA [L1] has several hourglass control types to avert the occurrence

of these modes. The Flanagan-Belytschko integration form was used in this study. The 2-node Hughes-Liu beam element formulation with 2×2 Gauss quadrature integration was used to model all reinforcing bars (i.e. longitudinal and transverse reinforcements). This element formulation is simple and allows for computational efficiency and robustness. It is also compatible with the brick elements, because the element is based on a degenerated brick element formulation and moreover, it includes finite transverse shear strains. This element contains six translational and six rotational degrees of freedom. Furthermore, a reference node is required for each element to determine the initial orientation of the cross-section. The steel reinforcing bars were modeled explicitly using beam elements connected to the concrete mesh nodes. The nodes that linked the concrete and reinforcement mesh were shared and therefore unable to slip. 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 [W2]). Moreover, there is not enough time to develop extensive bond slip along the length of the reinforcements during high loading rates. Thus the assumption of perfect bond in between the reinforcement and concrete is quite reasonable. Moreover, loading plate and support rollers were also modeled by solid elements.

Table 4.1: Total number of elements and nodal points for FE model

Beam designation	Solid elements for concrete	Beam elements	Nodal points
SR3.3_0.84	21317	228	35060
DR3.3_0.84_0.11	22967	634	37327
DR3.3_0.84_0.15	22967	702	37395
SR4.4_0.82	11856	228	14718
DR4.4_0.82_0.13	12063	594	25353
DR4.4_0.82_0.19	12063	646	25405

Fig. 4.1: Three-dimensional FE model of RC beams (a) $a/d = 3.3$; (b) $a/d = 4.4$
series

4.3.2 Boundary conditions, application of load and contact algorithm

To simulate the actual experimental conditions, the beams were supported on two rigid cylinders made of solid elements. Constraints were defined to the support cylinder, so that it could rotate about its own longitudinal axis but would not be able to translate. The load was prescribed (by BOUNDARY_PRESCRIBED_MOTION_RIGID keyword) to the beam through a rigid plates located at the top of the beam at its mid-span by specifying the same vertical displacement-history used during testing. The rigid loading plate was allowed to move only in the vertical direction. The prescribed displacement-history was similar to the displacement-history applied during experiment which successively generated the desired rate of loading. The corresponding load due to the prescribed displacement was then determined by monitoring the vertical reaction forces at the concrete nodes in contact with the support solid cylinders. The algorithm 'CONTACT AUTOMATIC SURFACE TO SURFACE' in LS-DYNA [L1] was used to model the contact between the support cylinders, loading plate and beam. This algorithm automatically generates slave and master surfaces and uses a penalty method where nominal interface springs were used for interpenetration between element and surfaces.

4.3.3 Constitutive model for concrete

One of the main challenges in modeling the behavior of RC structures is the difficulty of incorporating realistic material model that can precisely represent the behavior of the physical system. Concrete behaves quite differently from the other homogeneous material such as metals, due to its heterogeneous composition and inelastic behavior. Clear distinction exists in behavior between metals and concrete due to the random distribution of fine and coarse aggregate particles throughout the hardened cement matrix in concrete and the nonlinear behavior of it during the process of loading. To understand with the complex behavior of concrete, extensive research over the past two decades has resulted in a variety of constitutive models that are capable of representing the various aspects of concrete behavior (Bazant *et al.* [B4]; Kim and Lee [K4]; Chan *et al.* [C1]; Foster *et al.* [F1]; Kwak and Kim

[K14]). Despite the extensive research efforts, especially in the field of concrete materials, still there is a demand for a reliable material model capable of representing the complex behavior of concrete, and therefore it is an active research area. Concrete material formulation can be categorized into elasticity-based models, plasticity-based models, plastic-fracturing models, elasto-plastic damage models and endochronic models. This is beyond the scope of this research to discuss all material formulations in details and only the material model employed in the current simulation will be demonstrated.

A number of material models from LSDYNA [L1] material library can be used to represent concrete, such as Material type 005 (MAT SOIL AND FOAM); Material type 014 (MAT SOIL AND FOAM FAILURE); Material type 016 (MAT PSEUDO TENSOR); Material type 025 (MAT GEOLOGIC CAP MODEL); Material type 072 (MAT CONCRETE DAMAGE); Material type 084 (MAT WINFRITH CONCRETE); Material type 096 (MAT BRITTLE DAMAGE); Material type 159 (MAT CSCM CONCRETE); Material type 072 R3 (MAT CONCRETE DAMAGE REL3). In this study, MAT CSCM CONCRETE was employed to model concrete. This model was developed mainly for roadside safety application, such as concrete bridge rail and portable barrier (made of normal strength concrete) impacted by vehicles, but it could also be applicable to other dynamic applications. This is a cap model with a smooth intersection between the shear yield surface and hardening cap, as shown in **Fig. 4.2**. The initial damage surface coincides with the yield surface and the rate effects are modeled with viscoplasticity. The yield surface is formulated in terms of three stress invariants: J_1 , the first invariant of 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, S_{ij} and pressure, P as follows:

$$J_1 = 3P \quad (4-1)$$

$$J'_2 = \frac{1}{2} S_{ij} S_{ij} \quad (4-2)$$

$$J'_3 = \frac{1}{3} S_{ij} S_{jk} S_{ki} \quad (4-3)$$

Fig. 4.2: General shape of the concrete model yield surface in two dimensions
(Murray [M10])

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

$$f(J_1, J'_2, J'_3, \kappa) = J'_2 - \Re^2 F_f^2 F_c \quad (4-4)$$

Here F_f is the shear failure surface; F_c is the hardening cap; and $\Re$ 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 modeled by the shear surface in the tensile and low confining pressure regimes. The shear failure surface F_f is defined as follows:

$$F_f(J_1) = \alpha - \lambda \exp^{-\beta J_1} + \theta J_1 \quad (4-5)$$

Here the values of α , β , λ and θ are selected by fitting the model surface to the strength measurements from tri-axial compression tests conducted on plain concrete cylinders. The strength of the concrete is modeled 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 modeled). 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 via 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 behavior is modeled through a damage formulation. Without the damage formulation, the cap model predicts perfectly plastic behavior for laboratory tests such as direct pull, unconfined compression, triaxial compression and triaxial extension. This behavior is not realistic although 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. Default parameter generation option (by providing some key specifications) has been used herein instead of the traditional method where all material parameters were needed. Default parameters are provided for the concrete model based on three input specifications: the unconfined compressive strength of concrete, aggregate size and units. This option is valid for unconfined compressive strength from 20 to 58 MPa and aggregate sizes from 8 to 32 millimeters. Further details of this material model can be found somewhere (Murray [M10]).

4.3.4 Constitutive model for reinforcing steel

The steel reinforcement bars (longitudinal and transverse reinforcements) within the beam were modeled as a strain sensitive uniaxial elastic-plastic material to account for its strain rate sensitivity as well as stress-strain history dependence. Material model MAT PIECEWISE LINEAR PLASTICITY (MAT 024) was used in this study to model reinforcing bars where the strain rate effects was duly considered. The expressions proposed by Malvar [M2] on strain rate effects on yield and ultimate stress of steel reinforcements were utilized in this study. **Fig. 4.3** shows this proposed DIF_s for both yield and ultimate stress. The DIF_s for steel can

be expressed as follows:

$$DIF_s = (\dot{\epsilon}/10^{-4})^\alpha \quad (4-6)$$

where for yield stress, $\alpha = \alpha_{fy}$; $\alpha_{fy} = 0.074 - 0.04 f_y / 414$; for ultimate stress, $\alpha = \alpha_{fu}$; $\alpha_{fu} = 0.019 - 0.009 f_u / 414$; $\dot{\epsilon}$ is the strain rate in s^{-1} and f_y is the reinforcement static yield strength in MPa. Eq. (4-6) is valid with yield stresses between 290 and 710 MPa, and for strain rates between 10^{-4} and $10 s^{-1}$.

Fig. 4.3: Proposed Dynamic Increase Factor (DIF_s) for reinforcing steel (Malvar [M2])

4.3.5 Constitutive model for supports and loading plate

MAT RIGID (MAT 020) was used from LS-DYNA [L1] material library to model the supports and loading plate. 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. Young's modulus and Poisson's ratio of steel material were used for the rigid material in the numerical simulation.

4.4 Validation of Finite Element Analysis Results

Before proceed further to carry out any case studies with the developed FE model,

it is essential to validate or verify the FE results with the experimental results to establish the reliability of the model. Therefore, this section provides the comparison of FE results with the test results presented in the previous chapter. Load vs. midspan deflection characteristics and crack pattern of the beams for various loading rates (e.g. static, low, medium and high) was used here for verification purpose.

4.4.1 Load vs. midspan deflection curves

The comparison of load vs. midspan deflection curves between test results and FE predictions is illustrated in **Fig. 4.4**. From this figure, it can be concluded that the FE models can predict the load vs. midspan deflection response of beam under varying loading rates within a reasonable accuracy.

Fig. 4.4: Comparison of load vs. midspan deflection of beams under varying loading rates: (a) 3.3 series; (b) 4.4 series (continued)

Fig. 4.4: Comparison of load vs. midspan deflection of beams under varying loading rates: (a) 3.3 series; (b) 4.4 series (continued)

Fig. 4.4: Comparison of load vs. midspan deflection of beams under varying loading rates: (a) 3.3 series; (b) 4.4 series

Although, the FE models predicts the peak resistance for all loading rates quite accurately; however, some deviation in stiffness is observed mainly for some cases in high loading rates (e.g. DR3.3_0.84_0.15, DR4.4_0.84_0.13 and DR4.4_0.84_0.19). This may be due to the simplification made in the previous

chapter that for high loading rates inertia force was deducted from the top load cell data by considering the beam as a SDOF system to evaluate its true resistance. Accurate calculation of the inertia force is a challenging issue as it is the distribution of forces along the length of the specimens. Moreover, for DR3.3_0.84_0.15 under static loading, the load vs. midspan deflection characteristic predicted by FE analysis did not project the experimental behavior. Although the longitudinal tensile reinforcements yielded before the beam attained peak resistance, still sudden drop in experimental load-midspan deformation curves after peak load was observed. After yielding, the tensile reinforcements did not exhibit strain-hardening characteristics and in the meantime due to the wider spacing (i.e. greater than the limit specified by ACI 318-08 [A2]) of transverse reinforcements, sudden drop in load-midspan deformation after peak resistance was perceived. Comparison of peak load in between test and numerical analysis results was summarized in **Table 4.2** for all beams except DR4.4_0.82_0.19 for low rates due to the lack of test result. It is evident that the FE models could capture the peak load under varying loading rates within an acceptable accuracy. The average value of the ratio of peak load in between test and simulation is 0.98 and the coefficient of variation is less than 10%. It was mentioned in the introduction section of this chapter that the FE model developed by Adhikary *et al.* [A3] did not capture the load vs. midspan deflection response of the beam under high loading rates quite satisfactorily. Thus, comparison of load vs. midspan deflection of RC3_S56 under varying loading rates was also made to check the accuracy of the current FE model, as shown in **Fig. 4.5**.

Fig. 4.5: Comparison of load vs. midspan deflection of RC3_S56 under varying loading rates (Adhikary *et al.* [A3])

Table 4.2: Comparison of peak resistance of beam between test and FE analysis results

Specimens	Loading rate (m/s)	Peak Load (kN)		Test/Simulation
		Test	Simulation	
SR3.3_0.84	4×10^{-4}	76.7	81.0	0.95
	4×10^{-2}	82.9	90.2	0.92
	4×10^{-1}	93.4	99.0	0.94
	2×10^0	103.0	108.3	0.95
DR3.3_0.84_0.11	4×10^{-4}	90.5	95.8	0.94
	4×10^{-2}	96.7	105.1	0.92
	4×10^{-1}	104.6	106.0	0.99
	2×10^0	115.1	119.5	0.96
DR3.3_0.84_0.15	4×10^{-4}	78.1	92.0	0.85
	4×10^{-2}	100.8	103.8	0.97
	4×10^{-1}	106.6	107.8	0.99
	2×10^0	113.3	115.0	0.99
SR4.4_0.82	4×10^{-4}	40.0	40.2	1.00
	4×10^{-2}	40.8	43.8	0.93
	4×10^{-1}	46.8	45.9	1.02
	2×10^0	51.0	52.3	0.98
DR4.4_0.82_0.13	4×10^{-4}	45.1	41.5	1.09
	4×10^{-2}	45.1	44.0	1.03
	4×10^{-1}	47.0	45.8	1.03
	2×10^0	53.1	54.3	0.98
DR4.4_0.82_0.19	4×10^{-4}	46.7	41.5	1.13
	4×10^{-2}	No data	-	-
	4×10^{-1}	47.5	46.1	1.03
	2×10^0	54.0	55.7	0.97
Mean				0.98
COV				0.06

4.4.2 Failure patterns

After comparing the load vs. midspan deflection responses, this section is presented the comparison of crack pattern of beams under varying loading rates. The damage of the beams obtained from numerical simulation is shown by plotting the fringes of effective plastic strain in the right hand side of **Fig. 4.6**. For DR3.3_0.84_0.11, cracking patterns are almost similar to the experimental one for all loading rates. However, for static loading the diagonal crack developed during experiment from the loading point to the support point cannot be predicted by numerical simulation.

Fig. 4.6: Comparison of crack pattern from experiment and fringes of effective plastic strain from simulation of beams under varying loading rates (continued)

Fringe Levels

Fig. 4.6: Comparison of crack pattern from experiment and fringes of effective plastic strain from simulation of beams under varying loading rates

To avoid excessive run time for the case of static loading in explicit method, the event simulation time (i.e. in seconds) is much lesser than the experiment (i.e. in few minutes). That means there was no enough time for the diagonal crack to

extend from loading to support points for the numerical simulation case. Moreover, it is well recognized that for quasi-static problem, implicit methods are usually more effective than explicit methods. To keep the consistency, here explicit methods are employed for all loading cases. Similarly, for DR3.3_0.8_0.15, crack patterns were well captured for all loading rates except static one. For 4.4 series specimens, ductile flexure type failures were arrested effectively by the FE model for all loading rates.

4.5 Parametric Study

4.5.1 Numerical simulation matrix

After verifying the numerical simulation results against the experimental results, this section presents a parametric investigation to elucidate more information about the behavior of beams under varying loading rates. The effects of some key parameters such as longitudinal reinforcement ratio, transverse reinforcement ratio, yield strength of longitudinal reinforcement and concrete grades on the response of beam were studied. The influence of these parameters on DIF of maximum resistance and failure modes was summarized and discussed. **Fig. 4.7** illustrates the general schematic diagram of the beams and **Table 4.3** summarizes the specimen characteristics of the simulation matrix.

Fig. 4.7: General schematic diagram of beam

Table 4.3: Specimen characteristics of simulation matrix

Beam mark	f'_c (MPa)	h (mm)	b (mm)	d (mm)	a (mm)	L (mm)	a/d	ρ_{LT} & ρ_{LB}		ρ_T		R_s/R_b
								Ratio (%)	f_{Ly} (MPa)	Ratio (%)	f_{Ty} (MPa)	
DR2.8_0.89_0.13	40	290	180	250	700	1700	2.8	0.89	520	0.13	310	0.93
DR2.8_0.89_0.26										0.26		1.19
DR2.8_0.89_0.42										0.42		1.52
DR2.8_1.69_0.13								1.69		0.13		0.51
DR2.8_1.69_0.26										0.26		0.65
DR2.8_1.69_0.42										0.42		0.84
DR3.3_0.84_0.19		250	150	210			3.3	0.84		0.19		1.32
DR3.3_0.84_0.38										0.38		1.80
DR3.3_0.84_0.56										0.56		2.20
DR3.3_1.62_0.19								1.62		0.19		0.73
DR3.3_1.62_0.38										0.38		0.99
DR3.3_1.62_0.56										0.56		1.21
DR4.4_0.82_0.19		190	120	160			4.4	0.82		0.19		1.84
DR4.4_0.82_0.47										0.47		2.82
DR4.4_0.82_0.94										0.94		4.46
DR4.4_1.6_0.19								1.6		0.19		0.98
DR4.4_1.6_0.47										0.47		1.51
DR4.4_1.6_0.94										0.94		2.38
Note: All side cover 40 mm for 2.8 and 3.3 series and 30 mm for 4.4 series (from the centre of tensile and compression reinforcements)												

4.5.2 Influence of longitudinal reinforcement ratio on DIF of maximum resistance

Fig. 4.8 illustrates the effects of longitudinal reinforcement ratio on the DIF of maximum resistance of beam under varying loading rates (low: 4×10^{-2} , medium: 4×10^{-1} and high: 2×10^0 m/s). Three types of beams were considered here in terms of their slenderness ratios (a/d -2.8, 3.3 and 4.4).

Fig. 4.8: Influence of longitudinal reinforcement ratio on DIF of maximum resistance of beams (a) a/d -2.8; (b) a/d -3.3; (c) a/d -4.4 (continued)

Fig. 4.8: Influence of longitudinal reinforcement ratio on DIF of maximum resistance of beams (a) a/d -2.8; (b) a/d -3.3; (c) a/d -4.4

From each type three cases were taken into account by varying transverse reinforcement ratios. Then, finally for each distinct case, two amounts of longitudinal reinforcements (approx. 0.3 and 0.6 times of balanced reinforcement ratio) were considered to quantify the effect on the DIF of maximum resistance. The amount of balanced reinforcement ratio is estimated to be 2.7%. Moreover, this

is worthwhile to mention that for all cases f'_c are taken as 40 MPa and f_{Ly} , f_{Ty} are considered to be 520 and 310 MPa, respectively. It was observed that DIF are on the higher side for beam with low reinforcement ratio. When the beam contains low amount of longitudinal reinforcements, the response is mainly influenced by the strain rate effect of longitudinal reinforcing steel which in turn causing an enhancement of the DIF.

4.5.3 Influence of longitudinal reinforcement ratio on failure mode

To describe the effects of longitudinal reinforcement ratio on failure mode of beam under varying loading rates, specimens from a/d -3.3 and 4.4 series were chosen. Left hand side of **Fig. 4.9(a)** displays the failure pattern of 3.3 series beam having longitudinal reinforcements of 0.3 times of balanced reinforcement ratio whereas right hand side shows longitudinal reinforcements of 0.6 times of balanced reinforcement ratio and for all cases transverse reinforcement ratios were kept identical (i.e. 0.56%). Similarly, Left hand side of **Fig. 4.9(b)** presents the failure pattern of 3.3 series beam having longitudinal reinforcements of 0.3 times of balanced reinforcement ratio whereas right hand side shows longitudinal reinforcements of 0.6 times of balanced reinforcement ratio and for all cases transverse reinforcement ratios are kept constant (i.e. 0.47%). This was observed that when the beam contains relatively small amount of longitudinal reinforcements (i.e. 0.3 times of balanced reinforcement ratio), only the formation of flexural cracks were observed for all loading rates; whereas for the case of beam having relatively higher amount of longitudinal reinforcements (i.e. 0.6 times of balanced reinforcement ratio), both flexural-shear and some diagonal cracks were noticed. Specifically, for high loading rates, extent of damage was more severe when the beam contained relatively higher amount of longitudinal reinforcements. Therefore, it can be concluded that change in failure mode may be observed (i.e. mainly in high rates considered here) when the amount of longitudinal reinforcement ratio is increased (i.e. approaching towards balanced-reinforcement ratio) in an under-reinforced beam.

Fig. 4.9: Influence of longitudinal reinforcement ratio on failure mode: (a) a/d - 3.3; (b) a/d - 4.4

4.5.4 Influence of transverse reinforcement ratio on DIF of maximum resistance

The effects of transverse reinforcement ratio on the DIF of maximum resistance of beam under varying loading rates (low: 4×10^{-2} , medium: 4×10^{-1} and high: 2×10^0 m/s) are shown in **Fig. 4.10**.

Fig. 4.10: Influence of transverse reinforcement ratio on DIF of maximum resistance of beams (a) a/d -2.8; (b) a/d -3.3; (c) a/d -4.4

Here, for all cases f'_c were taken as 40 MPa and f_{Ly} , f_{Ty} were considered to be 520 and 310 MPa, respectively. It is evident that DIF is on the lower side when the beam contains high amount of transverse reinforcements.

4.5.5 Influence of transverse reinforcement ratio on failure mode

The effects of transverse reinforcement ratio on failure mode of beam for medium and high loading rates are shown in **Fig. 4.11** and **Fig. 4.12**.

Fig. 4.11: Influence of transverse reinforcement ratio on failure modes of a/d -3.3 series beams under medium and high loading rates

Fig. 4.12: Influence of transverse reinforcement ratio on failure modes of a/d -4.4 series beams under medium and high loading rates

Here only medium and high loading rates were selected due to that the effects of transverse reinforcement on cracking patterns are minimal for the considered beams under static and low loading rates. **Fig. 4.11** shows the failure patterns of 3.3 series beam having three different amounts of transverse reinforcements (i.e. 0.19, 0.38

and 0.56%). From this figure, it was clear that when the beam contained lower amount of transverse reinforcements (i.e. 0.19%), diagonal cracks were formed with flexure cracks; whereas flexure type of failure modes were observed in beam having high amount of transverse reinforcements (i.e. 0.56%). Similar type of phenomena was noticed in the failure patterns of 4.4 series beam, as clearly presented in **Fig. 4.12**.

4.5.6 Influence of yield strength of longitudinal reinforcement on failure mode

Literature review on RC beams under varying loading rates revealed that rate sensitivity of yield strength of longitudinal reinforcements plays a pivotal role in determining the mode of failure. Kulkarni and Shah [K13] reported that the final failure mode shifted from shear failure at static loading to flexure failure at the high rate for three pairs of singly reinforced beams and may be due to relatively small rate sensitivity of longitudinal reinforcement used. The above-mentioned phenomenon runs counter to the brittle transition in the mode of failure of doubly reinforced beam without shear reinforcements under high loading rates documented by Mutsuyoshi and Machida [M11] and may be due to high rate sensitivity of longitudinal reinforcement used by them. Therefore, this section provides some case-studies to highlight the effect of yield strength of longitudinal reinforcement on failure modes. Four types of beam were taken from **Table 4.3** and some case-studies were carried out by changing the yield strength of longitudinal reinforcements, keeping all others parameters identical. The beam designations, considered yield strengths of longitudinal reinforcements and corresponding static shear to bending resistance ratios are shown in **Table 4.4**. It is noted that all considered beams contain longitudinal reinforcement ratio of 0.6 times of balanced reinforcement ratio.

Table 4.4: Beam characteristics for numerical case-studies

Type	Beam mark	f_{Ly} (MPa)	R_s/R_b
(A)	DR3.3_1.62_0.19	420	0.89
	DR3.3_1.62_0.19	520	0.73
(B)	DR3.3_1.62_0.38	420	1.21
	DR3.3_1.62_0.38	520	0.99
(C)	DR4.4_1.6_0.19	420	1.17
	DR4.4_1.6_0.19	520	0.98
(D)	DR4.4_1.6_0.47	420	1.79
	DR4.4_1.6_0.47	520	1.51

Fig. 4.13 shows the influence of yield strength of longitudinal reinforcement on failure modes of a/d -3.3 series beams under varying loading rates. From **Table 4.4**, it is clear that type-A beams are statically shear-critical. There was no evidence of flexure type failure (**Fig. 4.13a**) when the loading rates progressed from static to high; however the severity of shear failure increased for high rates. First beam from type-B (i.e. having yield strength of 420 MPa of longitudinal reinforcements) is statically flexure critical as static shear to bending resistance ratio (i.e. 1.21) is greater than one and from **Fig. 4.13b**, ductile type failure was observed for static and low rates whereas punching shear cracks at midspan was noticed for medium loading rate. With the increment of the loading rate to high, severe failure in terms of both punching and diagonal shear cracks was the ultimate consequences. Due to high rate sensitivity of low yield strength steel, the increment of yield strength in medium and high rates was such a way that the bending resistance surpassed the shear resistance and eventually failure mode changed. The static shear to bending resistance ratio of second beam of type-B (i.e. having yield strength of 520 MPa of longitudinal reinforcements) is almost equal to one, thus any type of failure (i.e. ductile or brittle) could be possible for static loading. Here, ductile type failure was perceived for static and low loading rates whereas for medium and high rates, both punching and diagonal shear cracks were observed (**Fig. 4.13b**). Similarly, the influence of yield strength of longitudinal reinforcement on failure modes of a/d -4.4 series beams under varying loading rates are presented in **Fig. 4.14**. Change in failure mode from ductile at static loading to punching and diagonal shear failure at high rates was seen for the first beam of type-C (static shear to bending resistance

ratio- 1.17). For both beams of type-D (the static shear to bending resistance ratio is greater than 1.5), ductile type of failure is observed for all loading rates. Although the severity in flexure failure increases in large extent, however, there was no change in failure mode.

Fig. 4.13: Influence of yield strength of longitudinal reinforcement on failure modes of a/d -3.3 series beams under varying loading rates

Fig. 4.14: Influence of yield strength of longitudinal reinforcement on failure modes of a/d -4.4 series beams under varying loading rates

4.5.7 Influence of grade of concrete on peak load

Due to the limitation of constitutive model of concrete applied in this FE model, the unconfined compressive strength of concrete used in this parametric study is within 58 MPa. Therefore, to delineate the effect of grade of concrete on peak load under varying loading rates, the unconfined compressive strength of concrete was varied from 30 to 50 MPa keeping all other parameters constant. For all cases, f_{Ly} and f_{Ty} are considered to be 520 and 310 MPa, respectively. Influence of various grade of concrete (e.g. 30, 40 and 50 MPa) on peak load of beams under varying loading rates is shown in **Fig 4.15**. Increasing trend in peak load was observed when the loading rate progresses from static to high. Moreover, beam having high strength concrete (i.e. 50 MPa) yielded higher peak load for all loading rates as compared to relatively low strength concrete (i.e. 30, 40 MPa) beams.

Fig. 4.15: Influence of grade of concrete on peak load of beams under varying loading rates: (a) $a/d-2.8$; (b) $a/d-3.3$; (c) $a/d-4.4$ (continued)

Fig. 4.15: Influence of grade of concrete on peak load of beams under varying loading rates: (a) a/d -2.8; (b) a/d -3.3; (c) a/d -4.4

4.6 Summary

FE model was developed in this chapter to evaluate behavior of beams under varying rates of concentrated loading at midspan. The comparisons of experimental and analytical results indicated that this model is able to predict both the load vs. midspan deflection characteristics and cracking profile with reasonable accuracy. After verifying the numerical simulation results with the experimental outcomes, parametric investigation was carried out to elucidate the influence of some key parameters on DIF of maximum resistance and failure modes.

This was observed that DIF were on the higher side for beam with low reinforcement ratio. When the beam contains low amount of longitudinal reinforcements, the response was mainly influenced by the strain rate effect of longitudinal reinforcing steel which in turn causing an enhancement of the DIF. This was noticed that when the beam contains relatively small amount of longitudinal reinforcements (i.e. 0.3 times of balanced reinforcement ratio), only the formation of flexural cracks were observed for all loading rates; whereas for the case of beam having relatively higher amount of longitudinal reinforcements (i.e. 0.6 times of balanced reinforcement ratio), both flexural-shear and some diagonal cracks were noticed. Specifically, for high loading rates, extent of damage was more severe for the case of beam containing relatively higher amount of

longitudinal reinforcements. Therefore, it can be concluded that change in failure mode may be observed (i.e. mainly in high rates considered here) when the amount of longitudinal reinforcement ratio was increased (i.e. approaching towards balanced-reinforcement ratio) in an under-reinforced beam.

Beam containing higher amount of transverse reinforcements, DIF would be on the lower side. Furthermore, for medium and high loading rates, the beam containing lower amount of transverse reinforcements, diagonal cracks were formed with flexure cracks; whereas flexure type of failure was observed in beam having high amount of transverse reinforcements. Hence, the amount of transverse reinforcements had significant influence in preventing the formation of diagonal cracks under medium and high loading rates. For beams having various concrete grades (30, 40 and 50 MPa), increasing trend in peak load was observed when the loading rate progressed from static to high. Moreover, beam having high strength concrete (i.e. 50 MPa) yielded higher peak load for all loading rates as compared to relatively low strength concrete (i.e. 30, 40 MPa) beams.

The yield strength of longitudinal reinforcements of beams is the key parameter, affecting the change in failure mode from flexure at static loading to shear at high rates. Punching and diagonal shear cracks were observed in medium and high rates whereas only flexure cracks were perceived in static and low rates in beams containing longitudinal reinforcements of yield strength 420 MPa and the static shear to bending resistance ratio less than 1.5. Due to high rate sensitivity of low yield strength steel (i.e. 420 MPa yield strength steel was more sensitive than 520 MPa yield strength steel), the increment of yield strength of longitudinal reinforcements in medium and high rates was such a way that the bending resistance surpassed the shear resistance and eventually failure mode changed.

CHAPTER 5

EXPERIMENTAL ASSESSMENT OF RC BEAMS SUBJECTED TO DROP-WEIGHT IMPACT LOADING

5.1 Introduction

With the rapid development of the infrastructural project around the world, there is a chance that RC structures may be experienced from various types of impact loading during their service life due to several events. Moreover, concrete structures have been widely used over the years by engineers in the design and construction of protective structures to resist impact loads. Typical examples comprise in a diversified fields ranging from transportation structures (i.e., bridge piers, guard rails, traffic signal posts and electric poles etc.) 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 and ice impact or subjected by tornado or tsunami-borne debris impact, columns in multi-story car park or bridge-pier strike by moving vehicle, protective structures subjected to projectiles or aircraft impact and structures sustaining shock and impact loads during explosions or earthquakes. **Fig. 5.1** depicts some of the impact loading scenarios on structures. Impact loading is an extremely severe loading conditions characterized by its application of a force of great intensity within a short duration. Impact loading can be categorized into two basic types: single point impact loading and distributed impact loading. When a structures strike by an impactor or striker at a particular point that is called single point impact; whereas explosions or blasts would bring about distributed impact loading. Banthia [B1] summarized three distinct situations in a typical impact event by considering the size and mass of the impacting body: a very large object struck by a small impacting mass, an impact involving comparable masses, a small object struck by a large impactor. Although, the third case is comparatively rare, the first and second cases are often encountered. Furthermore, in terms of mass and velocity of the impactor, impact can be classified as low and high velocity impact. Low velocity impact can be referred to the impact load caused by a larger impactor with low

velocity whereas low-mass impactor at high velocity can cause a high velocity impact. Gas gun can be used to generate high velocity impact which will cause local failure (i.e. perforation and penetration etc.) of the structures. To simulate the low velocity impact scenario, drop-weight impact test set up would be ideal, which will cause the overall failure (i.e. shear and flexure failure with some local failure at or nearby impacting point) of the structures.

Fig. 5.1: Impact loading scenarios on structures: (a) Yoshida *et al.* [Y4]; (b) Yuan and Harik [Y5]; (c) Dahlberg [D1]; (d) Sugano *et al.* [S9]

The behavior of RC beam under low velocity impact may consist of two response phases as shown in **Fig. 5.2**: the local response due to the stress wave that occurs at the impacting point during a very short period after impact; and the overall response include the free vibration of the whole structural member (Fujikake *et al.* [F4]). Load rate effects and dynamic behavior of the structural components primarily

control the overall response. It is recognized that the structural members behave differently under impact loading as compared static loading. Furthermore, the effect of impact loading can result in catastrophic and sudden structural failure. So the understanding of the performance and vulnerability of RC structures under drop-weight impact loading has become an emerging topic in recent years. Although, numerous researches (Hughes and Beeby [H3]; Banthia [B1]; Kishi *et al.* [K5-K8]; Chen and May [C3]; Fujikake *et al.* [F4]; Bhatti *et al.* [B8]; Saatci and Vecchio [S1]; Tachibana *et al.* [T1]; Kishi and Mikami [K10]) have been conducted on evaluating the impact response of RC beams; however till now little effort has been made towards determining the impact responses of beams by considering various parameters at a time. Therefore, this chapter is devoted on the assessment of the structural response of beam induced by low velocity impact (<10 m/s) at its midspan. The impact loading is produced by dropping a steel impactor on the beams from specific heights.

Fig. 5.2: Impact responses of a RC member (Fujikake *et al.* [F4])

5.2 Objective of Experimental Program

Due to the wide prevalence of unintentional, man-made accidents and natural

calamities, demand for impact-resistant structures has increased tremendously. However, there is a lack of systematic research on structural behavior of beams under drop-weight impact loading. Considering the effect of various parameters at a time, drop-weight impact test was performed for various drop heights on beams by varying span length, cross-sections, shear span to effective depth ratios, longitudinal and transverse reinforcement ratios. The objective is to evaluate the impact responses (e.g. maximum impact load and maximum midspan deflections) and the failure modes of beams. Moreover, this experimental program will provide experimental results that can contribute further in the development and validation of analytical and numerical models. In addition, these impact-damaged beams will be used further to evaluate their post impact residual performances (e.g. residual resistance and stiffness) which are summarized in **Chapter 8**.

5.3 Outline of Experiment

The experimental program carried out in two phases: drop-weight impact tests and static tests. Impact program consisted of 30 RC beams which were divided into two groups in terms of their shear span-to-effective depth ratios (a/d -3.3, 3.8 and 5.7). Each group had three types of specimens distinguished by their shear reinforcement ratios. Each type of 3.3 series had four identical specimens whereas 3.8 and 5.7 series had three identical specimens. Six additional specimens identical to 3.8 and 5.7 series were cast to determine their experimental static behavior. However, the static behavior of 3.3 series specimens was determined from FE analysis due to the unavailability of experimental data. **Table 5.1** presents the overview of the entire test program including the considered drop-heights. Mass ratios ($\alpha = m_b/m_i$) i.e. ratios of the mass of the beams (m_b) to the impactor-mass (m_i) for a/d -3.3, 3.8 and 5.7 specimens were 0.42, 0.49 and 0.26 respectively. The specimen denoted by SR5.7_1.6 corresponds to ‘singly-reinforced’ beam (SR) followed by the shear span-to effective depth ratio and percentage of longitudinal tensile reinforcement ratios. Again, DR5.7_1.6_0.20 refers to ‘doubly-reinforced’ beam (DR) followed by the shear span-to effective depth ratio, percentage of longitudinal tensile and compressive reinforcement ratio and percentage of shear reinforcement ratio. It is

noted that all test specimens have been designed and cast at NTU, Singapore however; the drop-weight impact testing was conducted at NDA, Japan. Before commencing the impact loading tests, quasi-static tests were performed in a 3-point bending test set up by hydraulic machine, as shown in **Fig. 5.3**. For impact loading, a drop-weight impact loading machine was used, as shown in **Fig. 5.4**.

Table 5.1: Test program

Beam designation	Static test	Drop height (m)	Impact velocity (m/s)	Impact energy (kJ)
DR3.3_2.4	*	0.15, 0.30, 0.60, 1.20	1.72, 2.43, 3.43, 4.85	0.44, 0.88, 1.77, 3.53
DR3.3_2.4_0.12		0.30, 0.60, 0.90, 1.20	2.43, 3.43, 4.20, 4.85	0.88, 1.77, 2.65, 3.53
DR3.3_2.4_0.56		0.60, 0.90, 1.2, 1.60	3.43, 4.20, 4.85, 5.60	1.77, 2.65, 3.53, 4.71
SR3.8_0.8	√	0.30, 0.60, 0.90	2.43, 3.43, 4.20	0.88, 1.77, 2.65
DR3.8_0.8_0.11		0.60, 0.90, 1.20	3.43, 4.20, 4.85	1.77, 2.65, 3.53
DR3.8_0.8_0.15		0.60, 0.90, 1.20	3.43, 4.20, 4.85	1.77, 2.65, 3.53
SR5.7_1.6	√	0.30, 0.45, 0.60	2.43, 2.97, 3.43	0.88, 1.32, 1.77
DR5.7_1.6_0.15		0.30, 0.45, 0.60	2.43, 2.97, 3.43	0.88, 1.32, 1.77
DR5.7_1.6_0.20		0.30, 0.45, 0.60	2.43, 2.97, 3.43	0.88, 1.32, 1.77
Note: √= Tested; *= FEM determined

Fig. 5.3: Static loading test set up

A drop-weight with a mass of 300 kg was dropped freely onto the top surface of the

beam at midspan from different heights. For 3.3 series four and for 3.8 and 5.7 series specimens three different drop heights were considered. The striking head of 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 the uplift of beam during impact loading.

Fig. 5.4: Drop-weight impact loading test set up

All beams had rectangular cross section 120 to 160 mm in width, 170 to 250 mm in depth. Beams of 3.3 series were supported over the clear span of 1400 mm whereas specimens from 3.8 and 5.7 series were supported over the clear span of 1600 mm. The longitudinal tensile reinforcement ratios are in the range of 0.8 to 2.4%. For DR beam section equal amount of reinforcements were provided in compression side. When the longitudinal tensile reinforcement ratios were calculated in terms of balanced reinforcement ratios, these values would be around $0.3p_b$, $0.6p_b$ and $0.9p_b$ for 3.3, 3.8 and 5.7 series respectively; where p_b refers to balanced reinforcement ratio in percentage. The layout of the longitudinal reinforcements, spacing of shear reinforcements and the measuring points (accelerometers for measuring accelerations and strain gauges for measuring strain of steel reinforcing bars) are shown in **Fig. 5.5**. Here, ‘SA’ refers to the strain gauges attached at the midspan of longitudinal tensile reinforcements whereas ‘SS’ refers to the strain gauges attached to transverse reinforcements and A1, A2, A5 and A6 refers to the

accelerometers.

Unit: mm • Strain gauge for rebars

☒ Accelerometer

2@40=80

DR3.3_2.4

DR3.3_2.4_0.12

DR3.3_2.4_0.56

2@50=100

SR3.8_0.8

Fig. 5.5: Dimensions of RC beams, layout of reinforcements, location of strain gauges and accelerometers (continued)

Fig. 5.5: Dimensions of RC beams, layout of reinforcements, location of strain gauges and accelerometers

Longitudinal reinforcements consisted of deformed steel bars of T22 (diameter-13mm), T13 (diameter-13mm) and T13 (diameter-13mm) for a/d -3.3, 3.8 and 5.7 series specimen respectively while R6 (diameter-6mm) plain round bars were used as transverse reinforcements for all specimens. Target concrete compressive strength at 28 days was 40 MPa with maximum aggregate size of 10 mm for all specimens. The yield strengths of T22, T13 and R6 were 520, 520 and 310 MPa respectively. All specimens were cast at the same time using ready mix concrete of same batch; thence, all specimens had an identical concrete strength. The compressive strength at 28 days obtained from three concrete cylinders was 39.0, 37.6 and 38.8 MPa respectively. The specimen details, theoretical static capacities, static expected and observed failure modes are listed in **Table 5.2**, where the static flexural and shear resistances are calculated according to ACI 318-08 [A2].

Table 5.2: Details of specimens and static design parameters

Designation	Shear span to effective depth ratio	Longitudinal reinforcement ratio (%)	Transverse reinforcement ratio (%)	Bending resistance (kN)	Shear resistance (kN)	Shear to bending resistance ratio
DR3.3_2.4	3.3	2.4	0.00	203.70	66.20	0.33
DR3.3_2.4_0.12			0.12		87.10	0.43
DR3.3_2.4_0.56			0.56		170.40	0.84
SR3.8_0.8	3.8	0.8	0.00	70.45	67.83	1.04
DR3.8_0.8_0.11			0.11	67.83	93.32	1.38
DR3.8_0.8_0.15			0.15		102.30	1.51
SR5.7_1.6	5.7	1.6	0.00	42.40	35.32	0.83
DR5.7_1.6_0.15			0.15		50.52	1.19
DR5.7_1.6_0.20			0.20		56.40	1.33

To construct the specimens, steel formwork was used. Before the placement of reinforcing cages, oiling was carried out on the inside surfaces of formwork so that specimens can be removed easily and the surface of the specimens would not damage. Before the casting, concrete spacer blocks were placed in the underside as well as the side faces of transverse reinforcements to achieve clear concrete cover. Lifting hooks were mounted on both sides of the specimens to facilitate lifting process during testing. **Fig. 5.6** shows the photos of the casting of beams and the

preparation of concrete cylinders (150mm x 300 mm). After casting, damp hessian fabrics were employed to cure the specimens for two weeks.

Fig. 5.6: (a) Casting of beams; (b) preparation of concrete cylinders

All specimens were well instrumented to capture the loads, displacements, accelerations and the strains of steel reinforcements. The instrumentation for this test program included a load cell, strain gauges and accelerometers. Strain gauges with 3 mm gauge length were installed in the mid-span of the longitudinal tensile reinforcements and in the mid-point of the two legs of the shear reinforcements to measure the local strain. These are general purpose foil strain gauges (KFG types) with resistance of 120Ω and linear expansion coefficient of $11 \times 10^{-6}/^{\circ}\text{C}$. Soldering to the terminal of the gauges was not required due to the pre-attachment of three parallel vinyl-coated lead wires having length of 10 m. Before the installation of strain gauges at the specific locations, the reinforcement surfaces were grinded lightly and cleaned with alkaline and acidic chemicals. **Fig. 5.7(a) and (b)** shows the photos of the glued strain gauges in longitudinal tensile and transverse reinforcements. Although in **Fig. 5.7**, the strain gauge is shown in top face of the tensile reinforcements however, the reinforcement cages were inverted during the placement into the moulds; then the gauges would be in the bottom face of the tensile reinforcements. The impact load developed between the impactor and beams was measured by a dynamic load cell (frequency response 5 kHz), which was rigidly attached to the drop-weight. Four accelerometers (capacity of 1000 times gravity and resonance frequency greater than 70 kHz) were mounted on the

specimens for each test (except for static loading) to measure the accelerations. The midspan deflection of RC beams was measured by laser-type variable displacement transducers (LVDTs) which have a measuring range of 80 mm and sampling rate of 50 kHz. Data from the sensors were collected by a computer-based data acquisition system which has a sampling rate of 100 kHz. For getting insights about the cracking patterns and failure modes of beams, digital photography and high-speed video recording systems were employed.

(a)

(b)

Fig. 5.7: (a) Strain gauging in tensile longitudinal reinforcements; (b) Strain gauging in transverse reinforcements

5.4 Experimental Results and Discussions

Following the outline of experimental sections which consist of specimen design,

material properties, test-setup and instrumentation; experimental results and major observations are comprehensively presented and discussed in this section.

5.4.1 Static test results

Load vs. midspan deflection responses obtained from static tests are presented in **Fig. 5.8** and simultaneously failure patterns are depicted in **Fig 5.9**. SR beam of 3.8 series failed in shear-critical mode with the formation of diagonal shear crack. Sharp fall of the post-peak branch in the load vs. midspan deflection curves of SR3.8_0.8 is clearly seen whereas, DR beams from same series exhibited ductile failure response. For specimens of 5.7 series, ductile failures were observed. From the picture of the crack pattern of SR5.7_1.6 diagonal crack was perceived, however this crack was formed at the ending time of the testing process or at a large deflection.

Fig. 5.8: Static load vs. midspan responses of beams (a) 3.8 series; (b) 5.7 series

SR3.8_0.8

DR3.8_0.8_0.11

DR3.8_0.8_0.15

(a)

SR5.7_1.6

DR5.7_1.6_0.15

DR5.7_1.6_0.20

(b)

Fig. 5.9: Failure pattern of beams under static loading (a) 3.8 series; (b) 5.7 series

5.4.2 Impact test results

This section summarizes the test results obtained from the drop-weight impact tests. Time histories of impact load, midspan deflection, impact response characteristics (i.e. maximum impact load, maximum midspan deflection and time to reach maximum midspan deflection etc.) and crack patterns for each specimen at each drop height are summarized.

5.4.2.1 Time histories of impact load

Measuring the impact load in a drop-weight impact experiment poses some challenges. Here, the impact load history was measured using a load-cell attached to the impactor. **Fig. 5.10** shows the time histories of impact load with time duration of 10 ms from the beginning of impact. The impact load history comprises two initial successive peak parts within short time interval followed by steady part. The first peak was due to the initial shock of impact which was generated by the inertia of beam. Few cases the second peak surpassed the first one; this one could be due to the presence and the arrangement of load-cell which may itself affect the contact and the measurements (Bischoff [B8]). Furthermore, due to the direct contact in between the impactor and beam, local failure (i.e. cracking and spalling of concrete at or around the impact zone) played a significant role in controlling the characteristics of impact load history. Here, maximum value corresponding to first peak has taken as peak impact load.

Fig. 5.10: Time histories of impact load (a) 3.3 series; (b) 3.8 series; (c) 5.7 series
(continued)

Fig. 5.10: Time histories of impact load (a) 3.3 series; (b) 3.8 series; (c) 5.7 series

5.4.2.2 Time histories of midspan deflection

Fig. 5.11 shows the time histories of midspan deflection of beam. Almost similar type of shapes can be seen from these figures. The initial response exhibited a half-sine wave followed by steady response. Maximum and residual midspan deflection of the beam in the same series increased with the increment of the drop heights.

Fig. 5.11: Time histories of midspan deflection (a) 3.3 series; (b) 3.8 series; (c) 5.7 series (continued)

Fig. 5.11: Time histories of midspan deflection (a) 3.3 series; (b) 3.8 series; (c) 5.7 series

5.4.2.3 Impact response characteristics

Fig. 5.12 shows variation of maximum impact load under various drop heights of three series specimens. Similarly, the variation of maximum midspan deflection and time to reach maximum deflection at each drop heights are presented in **Fig. 5.13** and **5.14** respectively. With the enhancement of the drop heights, incremental

response was observed for each beam.

Fig. 5.12: Maximum impact load under various drop heights of (a) 3.3 series; (b) 3.8 series; (c) 5.7 series specimens

Fig. 5.13: Maximum midspan deflection under various drop heights of (a) 3.3 series; (b) 3.8 series; (c) 5.7 series specimens (continued)

Fig. 5.13: Maximum midspan deflection under various drop heights of (a) 3.3 series; (b) 3.8 series; (c) 5.7 series specimens

Fig. 5.14: Time to reach maximum midspan deflection under various drop heights of (a) 3.3 series; (b) 3.8 series; (c) 5.7 series specimens

5.4.2.4 Crack patterns

Besides, data collected by the sensors, the cracks developed after each test was marked and the crack widths were measured. The widths of the diagonal crack were measured at mid-height whereas for the case of flexural cracks, measurement was taken at the bottom surface of beam at midspan. **Fig. 5.15** shows the sketched cracks profiles of beams at each drop height. This was observed that the crack patterns and the failure modes varied between the specimens depending on their static bending and shears resistance. Detailed description of cracks profiles of beam at each drop height is summarized below:

(1-a) DR3.3_2.4 (Drop height - 0.15 m)

No damage was observed in this beam at a drop height of 0.15 m.

DR3.3_2.4 (Drop height - 0.30 m)

At 0.30 m drop height, a 0.75 mm diagonal crack was observed in the right hand side whereas another hairline diagonal crack appeared in other side. Local failure did not happen at or around the impact region.

DR3.3_2.4 (Drop height - 0.60 m)

With the increment of the drop height to 0.60 m, two diagonal cracks formed in the right hand side having width of 0.65 and 1.2 mm, respectively. In the other side, hairline diagonal crack was perceived. Massive crushing and spalling of concrete occurred at the impact region.

DR3.3_2.4 (Drop height - 1.2 m)

At 1.2 m drop height, the specimen suffered extensive damage with significant spalling of concrete at top, bottom and side surface of the beam in one side, exposing the longitudinal tensile and compressive bars. In the other side, spalling of concrete occurred at top and side surface of the specimen. The damage was so catastrophe that the beam was disintegrated into two components.

(1-b) DR3.3_2.4_0.12 (Drop height - 0.30 m)

Two hairline diagonal cracks were observed in this beam at a drop height of 0.30 m.

DR3.3_2.4_0.12 (Drop height - 0.60 m)

At 0.60 m drop height, the diagonal cracks in one side widened, reaching up to 1 mm. Moreover, diagonal cracks forming a shear-plug (crack width 1.3 mm in one side) under the impact point were observed.

DR3.3_2.4_0.12 (Drop height - 0.90 m)

With the further increment of the drop height to 0.90 m, the diagonal cracks in one side widened significantly, reaching up to 6.5 mm and in the other side, diagonal crack formed with 1.2 mm width. Like previous case, shear-plug developed at the impact point. Furthermore, local failure in terms of crushing and spalling of concrete at the impact region occurred.

DR3.3_2.4_0.12 (Drop height - 1.2 m)

At 1.2 m drop height, diagonal cracks in both sides broadened extensively, attaining up to 12.5 and 4.5 mm, respectively. The local damage was quite extensive with crushing and spalling of concrete. Moreover, few horizontal cracks propagated in the longitudinal direction of beam from the impact point.

(1-c) DR3.3_2.4_0.56 (Drop height - 0.60 m)

Hairline flexure-shear cracks were observed in this beam at a drop height of 0.60 m without any local damage.

DR3.3_2.4_0.56 (Drop height - 0.90 m)

At 0.90 m drop height, the number of flexure-shear cracks increased and flexural cracks at midspan developed. Still, at this drop height, no damage in the impact region occurred.

DR3.3_2.4_0.56 (Drop height - 1.2 m)

With the further increment of the drop height to 1.2 m, the flexure-shear cracks

became much flatter as compared to previous two cases. Moreover, some local damage has been occurred in the impact region. Some splitting cracks were identified in the side cover of the beam.

DR3.3_2.4_0.56 (Drop height - 1.6 m)

At 1.6 m drop height, the severity of damage increased significantly mainly at or around impact region, with crushing and spalling of concrete and number of flexure-shear cracks also increased.

(2-a) SR3.8_0.8 (Drop height - 0.30 m)

Typical flexure failure was observed in this beam for a drop height of 0.30 m. Flexural cracks developed from the bottom surface and propagated towards top surface of the beam. Flexural cracks (crack width 0.5 mm) underneath the impact point formed at the bottom surface and propagated towards top surface and finally crushed the compression concrete.

SR3.8_0.8 (Drop height - 0.60 m)

Like previous case, almost similar type of crack pattern has been perceived at a drop height of 0.60 m. However, flexural cracks underneath the impact point widened further, reaching up to 1.4 mm.

SR3.8_0.8 (Drop height - 0.90 m)

With the further increment of the drop height to 0.90 m, no too much difference in the crack pattern has been observed. However, flexural crack underneath the impact point became wider, reaching up to 1.7 mm. Moreover, local failure occurred at the impact region which combined with the flexural crack underneath the impact point disintegrated beam into two parts.

(2-b) DR3.8_0.8_0.11 (Drop height - 0.60 m)

Typical flexure failure was observed in this beam for a drop height of 0.60 m. Flexural cracks (width 5 mm) underneath the impact point developed from the bottom surface and propagated towards top surface. Moreover, some hairline flexure crack was perceived near surrounded region of midspan.

DR3.8_0.8_0.11 (Drop height - 0.90 m)

At 0.9 m drop height, flexural cracks underneath the impact point became widened, attaining up to 9 mm width and successively some crushing at the impact region was noticed.

DR3.8_0.8_0.11 (Drop height – 1.2 m)

With the further increment of the drop height to 1.2 m, both flexure crack (width around 15 mm) and shear plug (crack width around 11 mm at bottom side of beam) were observed. Moreover, massive local damage occurred at or around the impact region.

(2-c) DR3.8_0.8_0.15 (Drop height - 0.60 m)

Typical flexure failure was observed in this beam for a drop height of 0.60 m. Flexural cracks (width 1.1 mm) underneath the impact point developed from the bottom surface and propagated towards top surface. Moreover, few hairline cracks were perceived in the surrounding region.

DR3.8_0.8_0.15 (Drop height - 0.90 m)

Flexural cracks underneath the impact point became widened at 0.9 m drop height. The crack almost attained 9 mm in width at bottom surface of beam. At the same time some cracking and crushing of concrete at impact region were noticed.

DR3.8_0.8_0.15 (Drop height - 1.2 m)

With the further increment of the drop height to 1.2 m, flexural cracks underneath the impact point became widened significantly, reaching up to 30 mm in width. Moreover, there was significant damage occurred at the impact region.

(3-a) SR5.7_1.6 (Drop height - 0.30 m)

Typical flexure failure was observed in this beam for a drop height of 0.30 m. Flexural cracks (crack width 1.4 mm) underneath the impact point formed at the bottom surface and propagated towards top surface and connected to the crushed and spalled concrete at the impact point.

SR5.7_1.6 (Drop height - 0.45 m)

Like previous case, almost similar type of crack pattern has been perceived at a drop height of 0.45 m. However, flexural crack underneath the impact point widened more; reaching up to 6 mm. Crushing and spalling of concrete at the impact region were also noticed. These crushed and spalled concrete combined with the flexure crack crumbled the beam into two parts.

SR5.7_1.6 (Drop height - 0.60 m)

With the further increment of the drop height to 0.60 m, more flexural cracks developed and the flexure cracks beneath the impact point became more broaden, reaching up to 7 mm. This crack propagated vertically upwards and disintegrated the beam into two parts.

(3-b) DR5.7_1.6_0.15 (Drop height - 0.30 m)

Typical flexure failure was observed in this beam for a drop height of 0.30 m. Flexural cracks (crack width 1.1 mm) underneath the impact point formed at the bottom surface and propagated towards top surface.

DR5.7_1.6_0.15 (Drop height - 0.45 m)

Flexure cracks (width 5 mm) at the midspan and crushing of concrete at the impact region were perceived at drop height of 0.45 m.

DR5.7_1.6_0.15 (Drop height - 0.60 m)

With the increment of the drop height to 0.60 m, both flexure crack (width 4 mm) and shear-plug were observed. Significant crushing of concrete at the impact region has also been occurred.

(3-c) DR5.7_1.6_0.20 (Drop height - 0.30 m)

Typical flexure failure was observed in this beam for a drop height of 0.30 m. Flexural cracks (crack width 0.5 mm) underneath the impact point formed at the bottom surface and propagated towards top surface.

Fig. 5.15: Crack pattern of beams (a) 3.3 series; (b) 3.8 series; (c) 5.7 series

DR5.7_1.6_0.20 (Drop height - 0.45 m)

Flexural cracks underneath the impact point widened more as compared to previous case, reaching up to 6 mm and some hairline flexure cracks were observed in the surrounded region. Moreover, some crushing and spalling of concrete at impact region were noticed.

DR5.7_1.6_0.20 (Drop height - 0.60 m)

Extensive local damage (chunk of concrete spalled off from the both side of the impact point) has been occurred at a drop height of 0.60 m. Two major flexure cracks underneath the impact point were seen with width of 4.5 and 0.85 mm, respectively.

5.4.2.5 Strain rates

Effect of strain rate on material behavior (concrete and reinforcing steel) has been well documented in the literature (CEB model code [C5], Malvar and Ross [M5], Malvar [M2]). Constitutive material properties for structural materials are dependent of loading rates. Impact loads introduce considerably higher strain rate as compared to static loading, resulting in different behavior in concrete and steel. Hence, it is worthwhile to determine the maximum strain rate and their variation in the beam for various drop heights considered in this test program. This is mentioned earlier that strain gauges were mounted in all longitudinal tensile reinforcements and some transverse reinforcements; however, no initiative was taken to measure the strain in concrete. Strain rates are calculated by differentiating the strain-history data provided by the strain gauges. One specimen for a particular drop height was taken from each series (i.e. 3.3, 3.8 and 5.7) to present their strain rate-history data for longitudinal tensile reinforcements and transverse reinforcements. Here, 'SA' refers to the strain rates in longitudinal tensile reinforcements whereas 'SS' refers to the strain rates in transverse reinforcements. Maximum strain rates in longitudinal tensile reinforcements were found to be around 2.5, 6 and 5.9 s⁻¹ for DR3.3_2.4_0.56, DR3.8_0.8_0.15 and DR5.7_1.6_0.15, respectively for the corresponding drop heights mentioned in **Fig. 5.16**. Similarly, maximum strain rates in transverse reinforcements were found to

be around 3.4, 3 and 1.75 s^{-1} for DR3.3_2.4_0.56, DR3.8_0.8_0.15 and DR5.7_1.6_0.15, respectively for the corresponding drop heights mentioned in **Fig. 5.17**.

Fig. 5.16: Variation of strain rates in longitudinal tensile reinforcements

Fig. 5.17: Variation of strain rates in transverse reinforcements

Furthermore, **Fig. 5.18** presents the range of maximum strain rates of longitudinal tensile reinforcements for beams under various drop-heights. This will help the researchers to gain some idea regarding the development of maximum strain rates and correspondingly strain-rate dependent material properties can be applied in the analytical and numerical model. In general, maximum strain rates of longitudinal tensile reinforcements varied in between 1 to 7 s^{-1} for the corresponding considered drop heights of 0.15 to 1.2 m (i.e. impact velocity varies from 1.72 to 4.85 m/s).

Fig. 5.18: Range of maximum strain rates of longitudinal tensile reinforcements for beams of various series under various drop heights

5.5 Analysis of Experimental Database

This is mentioned in the literature review chapter that an equation was proposed by Tachibana *et al.* [T1] to estimate the maximum midspan deflection of beam based on impact energy and static flexural resistance. Similarly, empirical design formulae following the performance based design concept were suggested by Kishi and Mikami [K10], which were the equations involving the static flexural resistance, maximum and /or residual deflection and input impact energy. These equations can be used as design formulae following the performance-based design concept of RC beams. As limited database was employed by the previous researchers, thus, efforts have been devoted to find the relationship among static resistance (both flexure and shear), maximum midspan deflection and input impact

energy by using a databank documented by several researchers from the literature. A database of RC beams (including the data from the current test program too) subjected to drop weight impact loading was assembled from the literature. In total, 174 beams under drop weight impact loading at their mid span have been collected. The beam details, material properties and static shear to bending resistance ratio were tabulated in Appendix-B. Out of 174 specimens, 53 of them are statically shear failure type and remaining 121 are statically flexural failure type beams. All impact responses were sorted out based on their first impact. Furthermore, mass of the impactor, impact velocity and impact responses are succinctly presented in Appendix-C. Geometry of the test specimens, amount of their longitudinal and shear reinforcement, compressive strength of concrete and yield strength of longitudinal reinforcement spans a wide range. All beams have rectangular cross section 100 to 300 mm wide, 150 to 560 mm height and 1 to 4 m clear span length. Longitudinal reinforcement ratio varies from 0.84 to 2.75%, whereas the shear reinforcement ratio ranges from 0.11% to 0.75%. Concrete strength of the specimens mostly falls within 24 to 50 MPa. Diameter of longitudinal reinforcement among the test specimens in the database varies from 13 to 35 mm and diameter of transverse reinforcements for 90% of beam is 6 mm. The yield strength of longitudinal reinforcement varies from 345 MPa to 520 MPa. Moreover, various types, shape of the impactors and the interface between the beam and impactors are used by various researchers, as shown in **Table 5.3**. **Fig. 5.19** shows the relationship between maximum midspan deflections δ_{max} versus input impact energy over static flexural resistance E/R_b . About 90 out of 121 specimens of experimental data from statically flexural failure type beam are plotted in **Fig. 5.19** as there is insufficient experimental data given by previous researchers for the remaining 31 specimens. The static bending resistance of the considered beams varies from 11.4 to 237.5 kN and the input impact energy varies from 0.1 to 19.2 kJ due to the variation of mass of impactor (98.7 to 600 kg) and impact velocity (1 to 8 m/s). Although variations exist in beam geometry, types of impactor and the interface, it was observed that values of maximum midspan deflection fits well with the linear line. However, this was noticed that the proposed equation can be applied up to the value of 150 of E/R_b . More data is required in order to extend the

applicability of the equation beyond the value of 150 of E/R_b .

Table 5.3: Type of impactor and the interface between the beam and impactor

Investigators	Type of Impactor	Impact Interface
Kishi <i>et al.</i> [K5]	N/P	N/P
Kishi <i>et al.</i> [K7]	Spherical with a radius of curvature of 1407 mm	Direct Contact
Bhatti <i>et al.</i> [B8]	Spherical steel weight with a radius of curvature of 1407 mm	Direct Contact
Fujikake <i>et al.</i> [F4]	Drop hammer with a hemispherical tip with radius of 90 mm	Direct Contact
Chen and May [C3]	Stainless steel impactor with a 90 mm diameter and hemispherical profile of 125 mm radius and second one made of Mild steel with a 100 mm diameter and a flat contact surface	12 mm plywood and direct Contact
Saatci and Vecchio [S1]	305 mm square hollow structural steel weight	50 mm thick 305 mm square steel plate
Tachibana <i>et al.</i> [T1]	Steel weight with a curve contact surface with a length of 565 mm, a radius of 75 mm	Direct Contact
Kishi and Mikami [K10]	Steel weight with a spherical bottom of radius in 1407 mm and a 2 mm taper	Direct Contact
Note: N/P-Not Provided

Fig. 5.19: Relationship between maximum midspan deflections vs. input impact energy over static flexural resistance

Similarly, for statically shear critical beams, the relationship between maximum midspan deflections δ_{max} versus input impact energy over static shear resistance E/R_s is shown in **Fig. 5.20**. As mentioned earlier that there is insufficient impact response data for some shear failure type beam from the database, thus the data points shown in **Fig. 5.20** consists of 42 out of 53 specimens. The input impact energy varies from 0.15 to 19.2 kJ due to the variation of mass of impactor (98.7 to 600 kg) and impact velocity (1 to 9.3 m/s). It was observed that the maximum displacement can be accurately predicted by using the proposed equation. Finally this can be concluded that more data would be essential in future in order to extend the applicability of the equation beyond the value of 50 of E/R_s .

Fig. 5.20: Relationship between maximum midspan deflections vs. input impact energy over static shear resistance

5.5.1 Empirical formulas for impact-resistant design

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

For statically flexure failure type beam,

$$\delta_{max} = 0.574 \frac{E}{R_b} \quad (5-1)$$

Rearranging the above equation, it would be

$$R_b = 0.574 \frac{E}{\delta_{max}} \quad (5-2)$$

Similarly, for statically shear failure type beam,

$$\delta_{max} = 0.614 \frac{E}{R_s} \quad (5-3)$$

Rearranging the above equation, it would be

$$R_s = 0.614 \frac{E}{\delta_{max}} \quad (5-4)$$

where, R_b and R_s are in kN; E and δ_{max} are in J and mm, respectively.

Therefore, by specifying the maximum midspan deflection δ_{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 Eq. (5-2) and (5-4). To demonstrate the applicability of the proposed equations, comparison has been made with the current test results. **Figs. 5.21 and 5.22** (for flexure-critical and shear-critical beams) show the comparative plot of current experimental results of maximum deflection with the proposed equations. Thus, these two proposed equations could be used in design with the before-mentioned limitations.

Fig. 5.21: Comparison of current test results with proposed equation for flexure-critical beams

Fig. 5.22: Comparison of current test results with proposed equation for shear-critical beams

The equations proposed for statically flexure failure type beam by Tachibana *et al.* [T1] and Kishi and Mikami [K10] are almost similar to the proposed Eq. (5-2) except for the constant values which were specified as 0.522 and 0.63, respectively. The constant value proposed in Eq. (5-2) is 0.574, which is almost average of the two values mentioned above; therefore Eq. (5-2) gives a less conservative but more economical calculation of required static bending resistance as compared to Kishi and Mikami [K10] and vice versa for Tachibana *et al.* [T1]. Although there are some variations in beam dimensions, test setup, type of impactor and impactor interface among researchers, 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 statically shear failure type beam against impact loading can be evaluated by a simple equation proposed by Kishi *et al.* [K7] as follows:

$$V_{usd} = 0.8 \frac{E_{kd}}{\delta_{rd}} \quad (5-5)$$

where V_{usd} is static shear resistance, E_{kd} is input kinetic energy and δ_{rd} is residual displacement. The constant value proposed by them was 0.8 whereas Eq. (5-4) provides a constant value of 0.614. The discrepancy is due to the reason that Kishi *et al.* [K7] considered limit state of residual deflection, δ_{rd} as an input parameter while in Eq. (5-4) limit state of maximum midspan deflection, δ_{max} is considered. Therefore, it is difficult to directly compare these two equations.

5.6 Summary

Experimental assessment of RC beam under drop-weight impact loading and the analysis of experimental dataset obtained from the literature are presented in this chapter. Following conclusions are obtained:

(1) Maximum impact load, maximum midspan deflection and time to reach maximum deflection were found to increase with the increment of drop-heights or impact velocities. Here, maximum value of impact load corresponding to first peak had taken as peak impact load. The first peak was due to the initial shock of impact which was generated by the inertia of beam. However, for few cases the second peak surpassed the first one; this one could be due to the presence and the arrangement of load-cell which may itself affect the contact and the measurements. Furthermore, due to the direct contact in between the impactor and beam, local failure (i.e. massive cracking and spalling of concrete at or around the impact zone at higher drop heights) played a significant role in affecting the response of impact load history.

(2) The crack pattern and failure modes varied among the specimens depending on their static resistance. Specimens of 3.3 series are statically shear-critical, therefore in DR3.3_2.4 (without transverse reinforcements) hairline diagonal cracks were observed at 0.3 m drop height. With the enhancement of the drop-heights, width of the diagonal cracks and severity of local failure increases. At drop height of 1.2 m, the damage was so severe that the beam was disintegrated into two components. Similarly, for DR3.3_2.4_0.12, hairline diagonal cracks formed at low drop height (i.e. 0.3 m) and with the increment of the drop heights, both widened diagonal

cracks and shear plug were noticed. At 1.2 m drop height, diagonal cracks in both sides broadened extensively and extensive crushing and spalling of concrete occurred at the impact region. Although, DR3.3_2.4_0.56 is statically shear-critical, however, only few flexure-shear cracks were perceived at a drop height of 0.6 m. With the further increment of drop heights, number of flexure-shear cracks increases without any formation of diagonal cracks. This could be due to the narrower spacing (i.e. 70 mm) of the transverse reinforcements in this beam. As much as at 1.6 m drop heights, severity of damage increased significantly mainly at or around impact region, with crushing and spalling of concrete. Specimens of 3.8 series are statically flexure-critical, therefore in SR3.8_0.8 (without compression and transverse reinforcements) flexural cracks beneath the impact point were observed at 0.3 m drop height. With further increment of drop heights, flexural cracks became much wider and propagated vertically and crushed the compression concrete. At a drop height of 0.9 m, local failure occurred at the impact region which combined with the flexural crack underneath the impact point disintegrated beam into two parts. Therefore, it can be concluded that compression reinforcements plays an important role in preventing the disintegration of beam into two parts. Another two types of beams of the same series (DR3.8_0.8_0.11 and DR3.8_0.8_0.15) exhibited typical flexure failure at a drop height of 0.6 m. With the further increment of the drop height to 1.2 m, flexural cracks underneath the impact point became widened significantly. Moreover, there was massive local damage occurred at or near the impact region. Almost similar type of failure was observed for SR and DR beams of 5.7 series.

(3) Strain rates were computed from the strain history obtained from strain gauge. The range of maximum strain rates of longitudinal tensile reinforcements for beams of various series under various drop-heights is provided. This will help researchers to gain some idea regarding the development of maximum strain rates and correspondingly strain-rate dependent material properties can be applied in the analytical and numerical model. In general, maximum strain rates of longitudinal tensile reinforcements vary in between 1 to 7 s⁻¹ for the corresponding considered drop heights of 0.15 to 1.2 m (i.e. impact velocity varies from 1.72 to 4.85 m/s).

(4) Two empirical equations are proposed by analyzing a dataset on RC beams under drop-weight impact loading, documented by several researchers from the literature. By specifying the maximum midspan deflection for each limit state of beam, the required static bending and shear resistance can be determined for designing the beam subjected to input impact energy.

CHAPTER 6

FINITE ELEMENT MODELING OF RC BEAMS UNDER IMPACT LOADING

6.1 Introduction

From the literature review on FE analysis of RC beams under drop-weight impact loading, it was seen that most of the studies (Kishi *et al.* [K6], Bhatti *et al.* [B8], Saatci and Vecchio [S2], May and Sangi [M7]) concentrated their efforts on validating the numerical results and little efforts was expended towards understanding the effect of various parameters on impact responses. Moreover, impact testing of structural members to quantify the effect of several parameters is cumbersome, expensive and requires special laboratory facilities. Thus, to expand the knowledge beyond the range of parameters investigated experimentally and their effects on the structural responses, three-dimensional nonlinear FE model was developed in this chapter. Explicit finite element (FE) program LS-DYNA [H1] was employed in this study because of its proven effectiveness in geometric modeling and analysis capability for impact loading. However, prior to trusting the results of any numerical model, it is of vital interest to validate the numerical results with the test results. Therefore, the proposed FE model is first validated by the test results and then used to conduct parametric studies. Therefore, the first objective of this chapter is to numerically predict the experimental behavior (e.g. time histories of impact and midspan deflections and failure modes etc.) of beams under impact loading for various drop-heights. The second objective is that the model will be employed further in **Chapter 8** to numerically predict the residual resistance of beam after impact damage. Moreover, to carry out the residual resistance simulation in two steps, the resulting deformation and damages of beams from the impact stages will be utilized.

Finite element models of RC structures have generally been based on replacing the composite continuum by an assembly of elements representing the concrete and the steel reinforcement. Three alternative approaches are mainly used for modeling

reinforcement in a three-dimensional finite element model of a concrete structure: the discrete model, the embedded model, and the smeared model (El-Mezaini and Citipitiouglu [E1]). Existing literature suggests the improvement of bond between concrete and steel under impact loading. Thus, in this model, perfect bond is assumed in between the concrete and reinforcement. Hence, concrete and reinforcement mesh share the common nodes. However, to model the bond-slip cases, reinforcing bars are modeled using special elements connected to concrete through fictitious springs representing bond. This model is simple in concept and it has advantage of representing different material properties more precisely. The only shortcoming of using the discrete model is that concrete mesh patterns are restricted by the location of reinforcement.

6.2 Finite Element Model

The nonlinear explicit FE program LS-DYNA [L1] was employed in this numerical simulation study.

6.2.1 Structural configuration

To replicate the same experimental conditions, efforts are devoted here to develop a FE model as close to the physical system as possible. Therefore, a three-dimensional model is the obvious option to adopt. **Fig. 6.1** shows the three-dimensional FE model of beams (i.e. one type from each series). Eight node solid hexahedron elements with a single integration point and viscous hourglass control were used to represent beams (concrete), support rollers, inverted triangular plates and impactor while beam elements (2-node Hughes-Liu beam element formulation with 2×2 Gauss quadrature integration) were used to model steel reinforcing bars. The FE model of the impactor divided into three sections to reproduce the same used in the experiments. First section consists of hemispherical striking head having 90 mm radius whereas solid cylinder and rectangular block were used to model the remaining part. Different mesh sizes have been considered to study mesh sensitivity during the initial stage of numerical study. Here, a mesh size of 15 mm was used to create the solid elements which seemed to be ideal as further decrease

in mesh size almost generated the same results however augmented the computational time. **Table 6.1** presents total number of elements and nodes for each beam model. The beams were supported on two cylinders made of solid elements and to prevent the uplifting of the beam after impact loading two inverted triangular plates were modeled at the top of the beam at support positions. A numerical convergence study has shown that further decrease in mesh size had only a very little effect on the accuracy of the results, while considerably increasing the computational time. The mesh discretization was established in such a way that the reinforcement nodes coincided with the concrete nodes. The steel reinforcing bars were modeled 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 [W2]). 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 few milliseconds. Thus, the assumption of perfect bond in between the reinforcement and concrete was quite reasonable.

Table 6.1: Total number of elements and nodal points for FE model

Beam designation	Solid elements for concrete	Beam elements	Nodal points
DR3.3_2.4	22572	456	41266
DR3.3_2.4_0.12	22968	634	41900
DR3.3_2.4_0.56	24156	1258	46790
SR3.8_0.8	23760	270	42212
DR3.8_0.8_0.11	24288	818	40922
DR3.8_0.8_0.15	24640	902	52657
SR5.7_1.6	12960	270	27631
DR5.7_1.6_0.15	13248	720	28432
DR5.7_1.6_0.20	13440	776	28722

Fig. 6.1: Three-dimensional FE model of RC beams (a) $a/d = 3.3$; (b) $a/d = 3.8$; (c) $a/d = 5.7$ series

6.2.2 Boundary conditions, application of impact velocity and contact algorithm

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. Triangular plates are placed 1 mm above the beam surface instead of giving the direct contact to the beam to avoid any development of restraining moment at the supports. After placing the impactor 1 mm apart from the top surface of beam at mid-span, initial velocity was defined to it to save computational time which was calculated from the free-falling formula. Constraints were defined to the impactor in such a way that it could only move in vertical downward direction. Moreover, the gravitational acceleration of impactor and self weight of beam 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 was used to model the contact between the support cylinder, impactor and beam. The contact non-linearity was stabilized 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 were used to interpenetrate between element and surfaces. The corresponding impact force due to the prescribed initial velocity to impactor was then determined by monitoring the contact forces at the concrete nodes in contact with impactor.

6.2.3 Constitutive model for concrete

MAT CSCM CONCRETE (MAT 159) is commonly known as 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. The salient features of the models are: isotropic constitutive equations, three stress invariant yield surface with translation for pre-peak hardening, a hardening cap that expands and contracts, damage-based softening with erosion and modulus reduction, rate effects for increasing strength in high-strain rate application. Further

details of the concrete model theory could be found elsewhere (Murray [M10]). Moreover, the strain rate effects on concrete strength are duly considered in this model. Default parameter generation option (by providing some key specifications) was used herein instead of the traditional method where all material parameters were needed. Default parameters were provided for the concrete model based on three input specifications: the unconfined compressive strength of concrete, aggregate size and units. This option is valid for unconfined compressive strength from 20 to 58 MPa and aggregate sizes from 8 to 32 millimeters. The unconfined compressive strength affects all aspects of the fit, including stiffness, three-dimensional yield strength, hardening and damage-based softening whereas the aggregate size affects only the softening behavior of damage formulation.

6.2.4 Constitutive model for reinforcing steel

The steel reinforcement bars (longitudinal and transverse reinforcements) within the beam were modeled as a strain sensitive uniaxial elastic-plastic material to account for its strain rate sensitivity as well as stress-strain history dependence. Material model MAT PIECEWISE LINEAR PLASTICITY (MAT 024) was used in this study to model reinforcing bars where the strain rate effects are duly considered. The expressions proposed by Malvar [M2] about strain rate effects on yield and ultimate stresses of steel reinforcements were employed in this study (already discussed in Chapter 4).

6.2.5 Constitutive model for impactor, support rollers and inverted triangular plate

MAT RIGID (MAT 020) was used from LS-DYNA [L1] material library to model the impactor, support rollers and inverted 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. Young's modulus and Poisson's ratio of steel material were used for the rigid material in the numerical simulation.

6.3 Verification of Finite Element Analysis Results

To establish the reliability and accuracy of the FE model, it is essential to validate or verify the FE results with the experimental results. Therefore, this section provides the comparison of FE results with the test results presented in the previous chapter. Time histories of impact load and midspan deflection and crack pattern (i.e. at various stages of impact loading duration) of the beams for various drop heights would be used here for verification purpose. But, before that one conservation of energy should be checked in simulation results. As mentioned earlier, eight node solid hexahedron elements with a single integration point were used to model concrete. Although these elements have the advantage of requiring less computational time, however, hourglass modes (i.e. nonphysical, zero-energy modes of deformation that produce zero strain and no stress) occur in these elements. LS-DYNA [L1] has various algorithms for inhibiting hourglass modes. A probable way to reduce hourglassing is to refine the mesh. However, to entirely eliminate hourglass concern, the efficient way is to switch the element formulation with fully-integrated or selectively reduced integration. To satisfy the conservation of energy during an impact event, the kinetic energy of impactor before impact should be equal to the sum of the current kinetic energy, the internal energy of beam and the energy loss due to friction and hourglass modes. Energy loss due to friction was very small and can be neglected. The variation of various energies in a typical impact simulation is shown in **Fig. 6.2**.

Fig. 6.2: Variation of various energies in a typical impact simulation

Here, hourglass energy was around 3.5% of total energy at peak impact load and around 8.5% of total energy at maximum midspan deflection. Therefore, the low amount of hourglass energy in impact simulation corroborates the acceptance of analysis results.

6.3.1 Time histories of impact load curves

The comparison of time histories of impact load curves between test results and FE predictions is illustrated in **Fig. 6.3**. Moreover, comparison of first peak load in between test and numerical analysis results is summarized in **Table 6.2** for all tested beams. From **Fig. 6.3** and **Table 6.2**, it can be concluded that the FE models can predict the first peak load in impact load history under various drop heights within a reasonable accuracy.

Fig. 6.3: Comparison of time histories of impact load of beams for various drop heights (a) 3.3 series; (b) 3.8 series; (c) 5.7 series (continued)

Fig. 6.3: Comparison of time histories of impact load of beams for various drop heights (a) 3.3 series; (b) 3.8 series; (c) 5.7 series (continued)

(c)

Fig. 6.3: Comparison of time histories of impact load of beams for various drop heights (a) 3.3 series; (b) 3.8 series; (c) 5.7 series

However, the discrepancy in first post peak response characteristics is due to the presence and the arrangement of load-cell which may itself affect the contact and the measurements (Bischoff [B8]). Furthermore, due to the direct contact in between the impactor and beam, local failure (i.e. cracking and spalling of concrete at or around the impact zone) played a significant role in controlling the characteristics of impact load history. Moreover, the local oscillation and stress wave propagation through the impactor could affect the measurements.

Table 6.2: Comparison of maximum impact load of beam between test and FE analysis results

Specimens	Drop heights (m)	Impact velocity (m/s)	Maximum impact load (kN)		Test/FEM
			Test	FEM	
DR3.3_2.4	0.15	1.72	128.4	147.0	0.87
	0.30	2.43	195.3	209.2	0.93
	0.60	3.43	245.0	284.0	0.86
	1.20	4.85	257.0	345.0	0.74
DR3.3_2.4_0.12	0.30	2.43	195.0	208.0	0.94
	0.60	3.43	236.0	284.0	0.83
	0.90	4.20	315.0	315.3	1.00
	1.20	4.85	-	-	-
DR3.3_2.4_0.56	0.60	3.43	289.0	289.5	1.00
	0.90	4.20	336.0	305.0	1.11
	1.20	4.85	-	-	-
	1.60	5.60	-	-	-
SR3.8_0.8	0.30	2.43	201.0	170.0	1.18
	0.60	3.43	247.9	269.0	0.92
	0.90	4.20	291.0	304.0	0.96
DR3.8_0.8_0.11	0.60	3.43	245.9	258.0	0.95
	0.90	4.20	347.0	303.0	1.15
	1.20	4.85	374.0	347.0	1.08
DR3.8_0.8_0.15	0.60	3.43	264.5	271.0	0.98
	0.90	4.20	297.0	343.0	0.87
	1.20	4.85	315.0	360.0	0.88
SR5.7_1.6	0.30	2.43	126.0	117.0	1.08
	0.45	2.97	146.6	143.0	1.03
	0.60	3.43	170.6	170.0	1.00
DR5.7_1.6_0.15	0.30	2.43	127.2	132.0	0.96
	0.45	2.97	145.9	153.0	0.95
	0.60	3.43	160.5	162.0	0.99
DR5.7_1.6_0.20	0.30	2.43	117.0	116.0	1.01
	0.45	2.97	137.2	155.0	0.89
	0.60	3.43	168.9	170.0	0.99
Mean					0.97
COV					0.10

Moreover, two types of RC beam specimens (i.e. S1616 and S1322) were taken from Fujikake *et al.* [F4] to examine the accuracy of numerical model. The comparison of time histories of impact load curves between test results and FE predictions is presented in **Fig. 6.4**. From this figures, it can be opined that the numerical model can reasonably predict the impact load history of beams.

(b)

Fig. 6.4: Comparison of time histories of impact load of beams for various drop heights (a) S1616; (b) S1322

6.3.2 Time histories of midspan deflection curves

The comparison of time histories of midspan deflection curves between test results and FE predictions is shown in **Fig. 6.5**. Furthermore, **Table 6.3** summarized the comparison of maximum midspan deflection of test and numerical results for all tested beams.

Fig. 6.5: Comparison of time histories of midspan deflection of beams for various drop heights (a) 3.3 series; (b) 3.8 series; (c) 5.7 series (continued)

Fig. 6.5: Comparison of time histories of midspan deflection of beams for various drop heights (a) 3.3 series; (b) 3.8 series; (c) 5.7 series

Table 6.3: Comparison of maximum midspan deflection of beam between test and FE analysis results

Specimens	Drop heights (m)	Impact velocity (m/s)	Maximum midspan deflection (mm)		Test/FEM
			Test	FEM	
DR3.3_2.4	0.15	1.72	3.8	3.2	1.19
	0.30	2.43	6.9	5.2	1.33
	0.60	3.43	12.7	8.5	1.49
	1.20	4.85	-	-	-
DR3.3_2.4_0.12	0.30	2.43	6.4	5.2	1.23
	0.60	3.43	11.5	8.4	1.37
	0.90	4.20	-	-	-
	1.20	4.85	-	-	-
DR3.3_2.4_0.56	0.60	3.43	9.0	7.4	1.22
	0.90	4.20	13.4	10.0	1.34
	1.20	4.85	15.7	12.0	1.31
	1.60	5.60	19.2	15.2	1.26
SR3.8_0.8	0.30	2.43	10.8	13.0	0.83
	0.60	3.43	20.2	25.0	0.81
	0.90	4.20	29.5	38.0	0.78
DR3.8_0.8_0.11	0.60	3.43	18.6	16.9	1.10
	0.90	4.20	34.5	25.5	1.35
	1.20	4.85	41.0	33.0	1.24
DR3.8_0.8_0.15	0.60	3.43	19.6	17.4	1.13
	0.90	4.20	28.8	25.0	1.15
	1.20	4.85	39.2	33.6	1.17
SR5.7_1.6	0.30	2.43	20.0	19.9	1.01
	0.45	2.97	28.8	32.7	0.88
	0.60	3.43	37.1	42.7	0.87
DR5.7_1.6_0.15	0.30	2.43	20.0	18.3	1.09
	0.45	2.97	30.0	25.8	1.16
	0.60	3.43	39.1	33.5	1.17
DR5.7_1.6_0.20	0.30	2.43	19.1	18.4	1.04
	0.45	2.97	28.8	26.0	1.11
	0.60	3.43	37.9	33.2	1.14
Mean					1.14
COV					0.16

Similarly, the comparison of time histories of mid-span deflection curves between test results and FE predictions is shown in **Fig. 6.6**. RC beam specimens (i.e. S1616 and S1322) were adopted from the experimental program conducted by Fujikake *et al.* [F4].

Fig. 6.6: Comparison of time histories of mid-span deflection of beams for various drop heights (a) S1616; (b) S1322 (continued)

Fig. 6.6: Comparison of time histories of mid-span deflection of beams for various drop heights (a) S1616; (b) S1322

6.3.3 Crack profiles

This section presents the comparative plot of crack profiles (e.g. side surface) of beams at various stages (at 2.5, 5, 10, 15, 20 and 25 ms) of impact loading. The damage of the beams obtained from numerical simulation is shown by plotting the fringes of effective plastic strain in **Figs. 6.7 to 6.10**. **Fig. 6.7** shows the comparison of crack profiles of DR3.8_0.8_0.11 at a drop height of 0.6 m. In this case, the beam failed in flexure with minor local damage. At a drop height of 1.2 m, massive local damage occurred at or around the impact region with the widening of flexure cracks which is shown through the comparative plot in **Fig. 6.8**. Similar two comparative cases for DR5.7_1.6_0.20 under drop heights of 0.3 and 0.6 m are presented in **Figs. 6.9 and 6.10**, respectively where in the first case flexure cracks (with mild local damage) were observed and for the second extensive local damages occurred. This is evident from these figures that the crack patterns were captured quite effectively at various stages of impact loading.

Fig. 6.7: Comparison of crack profile from experiment and fringes of effective plastic strain from simulation of DR3.8_0.8_0.11 at a drop height of 0.6 m
(continued)

Fig. 6.7: Comparison of crack profile from experiment and fringes of effective plastic strain from simulation of DR3.8_0.8_0.11 at a drop height of 0.6 m

Fig. 6.8: Comparison of crack profile from experiment and fringes of effective plastic strain from simulation of DR3.8_0.8_0.11 at a drop height of 1.2 m
(continued)

at 15 ms

at 20 ms

at 25 ms

Fig. 6.8: Comparison of crack profile from experiment and fringes of effective plastic strain from simulation of DR3.8_0.8_0.11 at a drop height of 1.2 m

Fig. 6.9: Comparison of crack profile from experiment and fringes of effective plastic strain from simulation of DR5.7_1.6_0.20 at a drop height of 0.3 m
(continued)

Fig. 6.9: Comparison of crack profile from experiment and fringes of effective plastic strain from simulation of DR5.7_1.6_0.20 at a drop height of 0.3 m

at 2.5 ms

at 5 ms

at 10 ms

Fig. 6.10: Comparison of crack profile from experiment and fringes of effective plastic strain from simulation of DR5.7_1.6_0.20 at a drop height of 0.6 m
(continued)

Fig. 6.10: Comparison of crack profile from experiment and fringes of effective plastic strain from simulation of DR5.7_1.6_0.20 at a drop height of 0.6 m
(continued)

Fig. 6.10: Comparison of crack profile from experiment and fringes of effective plastic strain from simulation of DR5.7_1.6_0.20 at a drop height of 0.6 m

6.3.4 Impact resistance of test specimens

Impact resistance (i.e. reaction force) of specimens can be obtained from the support load cells. However, in this experimental investigation, due to the attachment of 100 kg mass to the support load cell, the test reaction force would not be the actual response. Hence, the test reaction force was not used for validation of simulation results. **Fig. 6.11** displays the typical variation of force history obtained from top and bottom load-cells during drop-weight impact testing.

Fig. 6.11: Typical variation of force history obtained from top and bottom load-cells during drop-weight impact testing

Moreover, there is a time lag (2.98 ms) in between the two responses which is clearly visible in this figure. This is due to the propagation time required of stress wave to travel from loading point to support region.

Fig. 6.12: Typical variation of impact and reaction load history (from numerical simulation)

Still, efforts are made (after gaining confidence from the verification of others impact responses) to present the variation of maximum reaction force (obtained from numerical simulation) and the ratio of maximum reaction force to calculated static resistance. **Fig. 6.12** displays the typical variation of impact and reaction force history obtained from numerical simulation and the time lag (2.45 ms) in between the two responses is clearly visible.

Table 6.4: Maximum reaction force obtained from numerical simulation

Specimens	Drop heights (m)	Maximum reaction force after impact (kN)	Calculated static resistance (kN)	Maximum reaction force after impact/ Calculated static resistance
DR3.3_2.4	0.15	176.4	66.2	2.66
	0.30	235.1		3.55
	0.60	314.9		4.76
	1.20	377.8		5.70
DR3.3_2.4_0.12	0.30	242.2	87.1	2.78
	0.60	315.9		3.63
	0.90	327.2		3.76
	1.20	358.1		4.11
DR3.3_2.4_0.56	0.60	317.0	170.4	1.86
	0.90	331.2		1.94
	1.20	367.5		2.16
	1.60	420.0		2.46
SR3.8_0.8	0.30	115.4	67.8	1.70
	0.60	150.8		2.22
	0.90	202.7		2.99
DR3.8_0.8_0.11	0.60	165.3	67.8	2.44
	0.90	209.6		3.10
	1.20	214.3		3.16
DR3.8_0.8_0.15	0.60	166.7	67.8	2.46
	0.90	224.9		3.32
	1.20	262.8		3.88
SR5.7_1.6	0.30	133.1	35.3	3.77
	0.45	168.6		4.77
	0.60	193.7		5.48
DR5.7_1.6_0.15	0.30	132.1	42.4	3.12
	0.45	160.4		3.78
	0.60	182.2		4.29
DR5.7_1.6_0.20	0.30	126.7	42.4	2.99
	0.45	168.5		3.97
	0.60	197.0		4.65

Table 6.4 presents the maximum impact resistance of beam for each drop-heights and the corresponding DIF. Here, DIF is defined here as the ratio of maximum reaction force to calculated static resistance of beam. This table may provide some insight about the variation of DIF under various drop-heights. However, more efforts should be spent in retrieving the reaction force-history accurately from experiment so that actual impact resistance of beam could be assessed.

6.4 Parametric Study

Parametric investigation is presented in this section to discuss more information about the behavior of beams under varying drop heights by varying some key parameters such as mass ratio (α) i.e. ratio of the beam-mass (m_b) to impactor-mass (m_i), longitudinal reinforcement ratios (ρ_L), compressive strength of concrete (f'_c) and boundary conditions. The influence of these parameters on impact responses is summarized and discussed. The specimens considered here having clear span length and cross sections are identical to the DR beams (a/d -3.3, 3.8 and 5.7) used in the experiment.

6.4.1 Effect of mass ratio (α)

Table 6.5 shows the variation in mass-ratios (i.e. 0.4 to 1.0) considered for each series specimens. Here the mass of the beams was kept constant; however, different mass ratios were obtained by varying the impactor-mass. To compare, the input kinetic energy is maintained constant whereas the impactor mass and drop-height is allowed to vary (e.g. impact energy: 2.65 kJ; impactor mass: 300, 170 and 125 kg; drop-height: 0.9, 1.59 and 2.16 m). **Fig. 6.13** illustrates the effect of the mass ratios on maximum impact load and maximum midspan deflection of beams under varying impact energies.

Table 6.5: Variations in mass-ratios

a/d	f'_c (MPa)	f_{Ly} (MPa)	f_{Ty} (MPa)	m_b (kg)	m_i (kg)	$\alpha = m_b/m_i$
3.3	40	520	310	126.0	300, 170, 125	0.42, 0.74, 1.00
3.8				147.5	300, 200, 150	0.49, 0.74, 1.00
5.7				78.3	150, 105, 78	0.52, 0.75, 1.00

It was observed that under certain impact energy, maximum impact load increases and maximum midspan deflection decreases with the increment of the mass-ratios. This physically implied that under constant impact energy, impacts caused by high mass with low velocity resulted in smaller maximum impact load but higher

maximum midspan deflection of beam and vice versa.

Fig. 6.13: Effect of mass ratios on maximum impact load and maximum midspan deflection of beams (a) $a/d = 3.3$; (b) $a/d = 3.8$; (c) $a/d = 5.7$

6.4.2 Effect of longitudinal reinforcement ratio (ρ_L)

The influence of longitudinal reinforcement ratio on maximum impact responses under various impact energies is presented in **Fig. 6.14**. The percentage of

longitudinal reinforcement ratios (i.e. 0.8, 1.5 and 2.0%) and these ratios in terms of balanced reinforcement ratios (0.3, 0.56 and 0.75) are depicted in **Table 6.6**. Here, the mass of the impactor was considered to be 300 kg. Maximum impact load increases and maximum midspan deflection of beam decreases with the enhancement of the longitudinal reinforcement ratios. **Fig. 6.15** exhibits the effect of longitudinal reinforcement ratio on failure pattern of beam under impact energy of 3.53 kJ. Severity in failure pattern (including diagonal and punching shear cracks) is observed with increasing amount of longitudinal reinforcements. This could be the reason behind the decrement in maximum midspan deflection when the beam contains higher amount of longitudinal reinforcements.

Table 6.6: Variations in longitudinal reinforcement ratios

a/d	f'_c (MPa)	f_{Ly} (MPa)	f_{Ty} (MPa)	ρ_b (%)	ρ_L (%)	ρ_L/ρ_b
3.8	40	520	310	2.7	0.8	0.30
					1.5	0.56
					2.0	0.75

Fig. 6.14: Effect of longitudinal reinforcement ratios on maximum impact load and maximum midspan deflection of beams

Fig. 6.15: Effect of longitudinal reinforcement ratios on failure pattern of beam under impact energy of 3.53 kJ

6.4.3 Effect of compressive strength of concrete (f'_c)

Beam having concrete compressive strength of 30, 40, 50 MPa was used to determine the effect of f'_c on maximum impact load and maximum midspan deflection under varying impact energies (considered mass of the impactor is 300 kg). Two case studies are depicted in **Fig. 6.16**. With the enhancement of concrete compressive strength from 30 to 50 MPa, an increasing trend in maximum impact load is observed whereas the opposite phenomenon was noticed for the case of maximum midspan deflections.

Fig. 6.16: Effect of compressive strength of concrete on maximum impact load and maximum midspan deflection of beams (a) $a/d=3.3$; (b) $a/d=5.7$

6.4.4 Effect of boundary conditions

To investigate the effect of boundary conditions on impact response of beam, pinned-end and fixed-end conditions were considered. **Fig. 6.17** clearly presents that maximum impact load is in the higher side however maximum midspan deflection is in lower side for fixed-end beam as compared to pinned-end beam. Development and propagation of cracks in beams (pinned-end and fixed-end) at various stage of impact loading are shown in **Fig. 6.18**.

Fig. 6.17: Effect of boundary conditions on maximum impact load and maximum midspan deflection of beams (a) $a/d=3.3$; (b) $a/d=3.8$; (c) $a/d=5.7$

Fig. 6.18: Effect of boundary conditions on failure pattern of beams (left: pinned-end; right: fixed-end) (continued)

DR3.8_0.8_0.11 (Drop height-0.9 m/Impact energy-2.65 kJ)

Fig. 6.18: Effect of boundary conditions on failure pattern of beams (left: pinned-end; right: fixed-end)

6.5 Summary

FE model was developed in this chapter to evaluate behavior of beams under drop-weight impact loading. After verifying the numerical simulation results against the experimental results, parametric investigation was carried out to elucidate the influence of some key parameters on impact responses.

Under constant impact energy, impacts caused by high mass with low velocity result in smaller maximum impact load but higher maximum midspan deflection of beam and vice versa. Maximum impact load increases and maximum midspan deflection of beam decreases with the enhancement of the longitudinal reinforcement ratios. With increasing amount of longitudinal reinforcements severity in failure pattern (including diagonal and punching shear cracks) is observed. Moreover, with the enhancement of concrete compressive strength from 30 to 50 MPa, an increasing trend in maximum impact load was observed whereas the opposite phenomenon was noticed for the case of maximum midspan deflections. Finally, to assess the effect of boundary condition on impact responses, fixed-end beam has been compared with pinned-end beam. Maximum impact load is in the higher side however maximum midspan deflection is in lower side for fixed-end beam as compared to pinned-end beam under considered impact energies.

CHAPTER 7

ANALYTICAL MODEL FOR IMPACT RESPONSE PREDICTION OF RC BEAM

7.1 Introduction

Maximum midspan deflection could be an important performance index to evaluate the damage levels (i.e. from no damage to moderate damage, severe damage and complete collapse) of beam when subjected to impact loadings (Fujikake *et al.* [F4]). Moreover, based on the relationships among maximum mid-span deflection of, static flexural resistance of beam and input impact energy empirical design formula following the performance-based design concept has been proposed by Tachibana *et al.* [T1], Kishi and Mikami [K10]. Therefore, prediction of maximum midspan deflection of beam subjected to impact loading could be an important damage index and design parameter. Numerical modeling and prediction of impact responses have been demonstrated in previous chapter. However, to reduce the modeling efforts and computational time, simplified analytical models are presented in this chapter.

7.2 Energy-balance Model

The balance of energy in the system is considered for analyzing the impact response. The initial kinetic energy of the impactor is used to deform the structure during impact. When the structure reaches its maximum deflection, the velocity of the impactor becomes zero and all the initial kinetic energy has been used to deform the structure, assuming that the structure behaves quasi-statically. The overall deformation of the structures usually involves bending, shear deformation, and for large deflections, membrane stiffening effects. Local deformations in the contact zone also are to be considered. For impacts that induce only small amount of damage, the energy needed to create damage can be neglected. Therefore, the energy-balanced equation can be written as

$$\frac{1}{2}MV^2 = E_b + E_s + E_m + E_c \quad (7-1)$$

where M = mass of the impactor, V = velocity just before impact and the subscripts b , s and m refer to the bending, shear and membrane components of the overall structural deformation, and E_c is the energy stored in the contact region during indentation. Moreover, this is possible to express the load-deflection relation in the form presented below

$$P = K_{bs}W + K_mW^3 \quad (7-2)$$

where P is the impact force, K_{bs} = stiffness including bending and transverse shear deformation effects, K_m = membrane stiffness and W is the deflection at the impact point.

$$E_b + E_s + E_m = \frac{1}{2}K_{bs}W_{max}^2 + \frac{1}{4}K_mW_{max}^4 \quad (7-3)$$

Contact deformation and contact force can be described by the well-known Hertzian Contact Law

$$P = n\alpha^{\frac{3}{2}} \quad (7-4)$$

where n represents the contact stiffness related to the material properties of contacting bodies and is given by $n = \frac{4}{3}R_i^{\frac{1}{2}}\left[\left(1-\nu_i^2\right)/E_i + 1/E_b\right]^{-1}$ (Sun and Yang [S8]) R_i , ν_i and E_i are radius, Poisson's ratio and young modulus of impactor and E_b is the young modulus of beam; α represents the relative motion or indentation of the structure by the impactor

$$E_c = \int_0^{\alpha_{max}} Pd\alpha = \frac{2}{5}n\alpha_{max}^{\frac{5}{2}} \quad (7-5)$$

Using equations (7-2), (7-4) and (7-5), the maximum indentation can be expressed in terms of the maximum displacement of the structure at the impact location:

$$\alpha_{max} = \left(\frac{P}{n}\right)^{\frac{2}{3}} = n^{-\frac{2}{3}}\left(K_{bs}W_{max} + K_mW_{max}^3\right)^{\frac{2}{3}} \quad (7-6)$$

After substituting into equation (7-5), the contact energy becomes

$$E_c = \frac{2}{5} n^{-\frac{2}{3}} (K_{bs} W_{max} + K_m W_{max}^3)^{\frac{5}{3}} \quad (7-7)$$

Using equations (7-3) and (7-7), the energy balance equation becomes

$$\frac{1}{2} MV^2 = \frac{1}{2} K_{bs} W_{max}^2 + \frac{1}{4} K_m W_{max}^4 + \frac{2}{5} n^{-\frac{2}{3}} (K_{bs} W_{max} + K_m W_{max}^3)^{\frac{5}{3}} \quad (7-8)$$

After rearranging equation (7-8),

$$\frac{1}{2} MV^2 = \frac{1}{2} K_{bs} W_{max}^2 \left[1 + \frac{1}{2} \frac{K_m}{K_{bs}} W_{max}^2 + \frac{4}{5} \left(\frac{K_{bs} W_{max}}{n} \right)^{\frac{2}{3}} \left(1 + \frac{K_m}{K_{bs}} W_{max}^2 \right)^{\frac{5}{3}} \right] \quad (7-9)$$

Neglecting the membrane effects ($K_m = 0$), above equation simplifies to

$$\frac{1}{2} MV^2 = \frac{1}{2} K_{bs} W_{max}^2 \left[1 + \frac{4}{5} \left(\frac{K_{bs} W_{max}}{n} \right)^{\frac{2}{3}} \right] \quad (7-10)$$

This equation can be solved numerically for W_{max} , maximum deflection of structures at impact point and maximum impact load can be calculated from the following equation

$$\frac{1}{2} MV^2 = \frac{1}{2} \frac{P_{max}^2}{K_{bs}} + \frac{4}{5} \frac{P_{max}^{\frac{5}{3}}}{n^{\frac{2}{3}}} \quad (7-11)$$

This should be noted that the energy method is not dependent on the mass of the structures. It can only provide the maximum impact responses such as maximum midspan deflection and maximum impact load; however, cannot capture the effect caused by various levels impactor mass and velocity when impact energy remains constant. Furthermore, time-history or time-to-peak response value cannot be evaluated by using this method.

7.2.1 Application of energy-balance model

The applicability of the energy-balance model is judged by comparing the predicted

response with the experimental results. **Fig. 7.1** shows the comparison of maximum midspan deflection of beams for various drop heights. For each beam in various series (3.3, 3.8 and 5.7), three drop heights are considered and this was observed that for the case of lower drop height, this model can capture the results quite accurately; however for the cases of intermediate to higher drop heights, the degree of accuracy decreases. Moreover, comparison of all tested data is presented in **Fig. 7.2** and discrepancies are mainly observed for midspan deflection more than 20 mm and obviously these are cases resulted from higher drop heights. This is due to the reason that in the theory it is assumed that the strain energy stored in the impacted structure equals the kinetic energy of the impactor at the instance of time when the maximum impact load occurs.

Fig. 7.1: Comparison of maximum midspan deflection of beams for various drop heights (a) 3.3 series; (b) 3.8 series; (c) 5.7 series (continued)

Fig. 7.1: Comparison of maximum midspan deflection of beams for various drop heights (a) 3.3 series; (b) 3.8 series; (c) 5.7 series (continued)

Fig. 7.1: Comparison of maximum midspan deflection of beams for various drop heights (a) 3.3 series; (b) 3.8 series; (c) 5.7 series

This assumption is solely valid for quasi-static impact (i.e. low velocity or low drop heights) events as the amount of energy lost due to large plastic deformations and generation of heat and noise in high-speed impacts (i.e. higher drop heights) would significantly alter the energy balance.

Fig. 7.2: Comparison of maximum midspan deflection of all tested beams

Furthermore to examine the effectiveness of energy method, the analytical results are compared with other researcher's test results. Two types of beams were taken from Fujikake *et al.* [F4] for validation purpose. From the comparative plots shown in **Fig. 7.3** it can be stated that for the case of lower drop height, this model can capture the results reasonably well.

Fig. 7.3: Comparison of maximum midspan deflection of beams for various drop heights (a) S1616; (b) S1322

7.3 Single-Degree-of-Freedom (SDOF) Model

Single-degree-of-freedom system is defined as one in which only one type of motion is possible, or in other words the position of the system at any instant can be defined in terms of a single coordinate. Here, the mass of the beam assumed to be lumped at the midspan. In this study, the impactor mass (300 kg) is much greater than the effective mass of the beam. Therefore, it is assumed that the beam and the impactor will move together after impact. **Table 7.1** shows the actual and effective mass of beams from various series considered in drop-weight impact test program. The natural circular frequencies are computed to be 968.4 and 686.3 rad/s for beams of 3.8 and 5.7 series, respectively. The idealized SDOF system of beam subjected to drop-weight impact loading is shown in **Fig. 7.4**. It is noted that damping is not accounted in this SDOF system. The initial velocity of the SDOF system is

$$V_0 = m_i V / (m_e + m_i) \quad (7-12)$$

where, $V = \sqrt{2gh}$; h is the drop height of the impactor, g is the gravitational acceleration, m_i is mass of the impactor, m_e is effective mass of beam.

Table 7.1: Actual and effective mass of beams

Beam series	Actual mass, m_b (kg)	Effective mass, m_e (kg)	
		Elastic stage, $0.5m_b$	Plastic stage, $0.33m_b$
3.8 series	147.5	73.8	48.7
5.7 series	78.4	38.2	25.9

Fig. 7.4: Idealized SDOF model

7.3.1 Elastic-plastic analysis

It is well known that elastic analysis of SDOF system (i.e. the resistance function has been a straight line with slope k and without any upper limit) cannot predict the midspan deflection history of beam under impact loading. Therefore, elastic-plastic SDOF system is adopted to reasonably predict the impact response. The assumed bilinear resistance functions of beam with and without strain rate effect are illustrated in **Fig.7.5**.

Fig. 7.5: Idealized resistance function of beam with and without strain rate

The equations of motion are as follows:

The elastic response equation can be written as follows:

$$(m_e + m_i)\ddot{u} + ku = 0; \quad 0 < u < u_e \quad (7-13)$$

The plastic response equation can be written as follows

$$(m_e + m_i)\ddot{u} + R_m = 0; \quad u_e < u < u_m \quad (7-14)$$

$$(m_e + m_i)\ddot{u} + R_m - k(u_m - u) = 0; \quad (u_m - 2u_e) < u < u_m \quad (7-15)$$

where, R_m is the resistance force at yielding of the beam, u_e is the elastic limit displacement and u_m is the maximum deflection. In **Fig. 7.5**, the subscripts ‘st’ and ‘sr’ of R and u are related to static and strain rate effect cases, respectively.

Using the assumed shape functions for the elastic ($\phi_i = \sin(i\pi x/L)$) and the plastic stage ($\phi_i = (2x/L)$), the equivalent beam mass (m_e) is as follows

$$\begin{aligned} \text{Elastic stage, } m_e &= 2 \int_0^{\frac{L}{2}} (m_b/L) \phi_i^2 dx \\ &= 2 \int_0^{\frac{L}{2}} (m_b/L) (\sin(i\pi x/L))^2 dx = \frac{1}{2} m_b \end{aligned} \quad (7-16)$$

$$\text{Plastic stage, } m_e = 2 \int_0^{\frac{L}{2}} (m_b/L) \phi_i^2 dx$$

$$= 2 \int_0^{\frac{L}{2}} (m_b / L) (2x / L)^2 dx = \frac{1}{3} m_b \quad (7-17)$$

To obtain the strain rate dependent resistance function (as shown in **Fig. 7.5**), strain rate dependent constitutive properties of concrete and reinforcing steel need to be incorporated in the sectional analysis. This is generally known that strain rates are in the range of 1 to 10 for low velocity impact. The dynamic increase factor ($DIF_{c/t}$ and DIF_s) and strain rate relationships of concrete and reinforcing steel are well documented in the literature.

The most comprehensive model for strain rate enhancement of concrete both in tension and compression is presented by the CEB model code [C5]. Strain rate effect on compression and tension is typically reported as dynamic increase factor ($DIF_{c/t}$) - i.e. ratio of dynamic to static strength. In compression CEB model equations come out to be properly fit with the available data. The DIF_c for compressive strength is given by:

$$DIF_c = \begin{cases} (\dot{\epsilon} / \dot{\epsilon}_s)^{1.026\alpha_s} & \dot{\epsilon} \leq 30s^{-1} \\ \gamma_s (\dot{\epsilon} / \dot{\epsilon}_s)^{1/3} & \dot{\epsilon} > 30s^{-1} \end{cases} \quad (7-18)$$

where $\dot{\epsilon}$ is the strain rate in the range of 30×10^{-6} to $300 s^{-1}$; $\dot{\epsilon}_s = 3 \times 10^{-6} s^{-1}$ (static strain rate); $\log \gamma_s = 6.156\alpha_s - 2$; $\alpha_s = 1 / (5 + 9 f_{cs} / f_{co})$; $f_{co} = 10$ MPa; f_{cs} is the static compressive strength of concrete.

Moreover, the DIF_c for ultimate strain in compression is as follows:

$$DIF_c = (\dot{\epsilon} / \dot{\epsilon}_s)^{0.02} \quad (7-19)$$

where $\dot{\epsilon}$ is the strain rate and $\dot{\epsilon}_s = 3 \times 10^{-6} s^{-1}$.

Finally, the DIF_t for tension is furnished as:

$$DIF_t = \begin{cases} (\dot{\epsilon} / \dot{\epsilon}_s)^{1.016\delta_s} & \dot{\epsilon} \leq 30s^{-1} \\ \beta_s (\dot{\epsilon} / \dot{\epsilon}_s)^{1/3} & \dot{\epsilon} > 30s^{-1} \end{cases} \quad (7-20)$$

where $\dot{\epsilon}$ is the strain rate in the range of 3×10^{-6} to 300 s^{-1} ; $\dot{\epsilon}_s = 3 \times 10^{-6} \text{ s}^{-1}$ (static strain rate); $\log \beta_s = 7.11\delta - 2.33$; $\delta_s = 1/(10 + 6f_{cs}/f_{co})$; $f_{co} = 10 \text{ MPa}$; f_{cs} is the static compressive strength of concrete.

However, it was found by Malver and Ross [M5] that the available data in literature and their additional new data on concrete in tension differed somewhat from the CEB recommendations, mostly for strain rates beyond 1 s^{-1} . Thus, they modified the DIF_t equations and the change in slope occurs at strain rate of 1 s^{-1} instead of 30 s^{-1} . The proposed formulations then becomes

$$DIF_t = \begin{cases} (\dot{\epsilon}/\dot{\epsilon}_s)^\delta & \dot{\epsilon} \leq 1 \text{ s}^{-1} \\ \beta(\dot{\epsilon}/\dot{\epsilon}_s)^{1/3} & \dot{\epsilon} > 1 \text{ s}^{-1} \end{cases} \quad (7-21)$$

where $\dot{\epsilon}$ is the strain rate in the range of 10^{-6} to 160 s^{-1} ; $\dot{\epsilon}_s = 10^{-6} \text{ s}^{-1}$ (static strain rate); $\log \beta = 6\delta - 2$; $\delta = 1/(1 + 8f_{cs}/f_{co})$; $f_{co} = 10 \text{ MPa}$; f_{cs} is the static compressive strength of concrete.

Several studies have been documented on the effect of strain rate on reinforcing bars, structural steel and steel wires (Keenan and Feldman [K2]; Wakabayashi et al [W1]; Soroushian and Choi [S8]). A detailed review of the available work has been conducted by Fu *et al.* [F3] and Malvar [M2]. According to Wakabayashi *et al.* [W1], yield stress of steel bar increases with increasing strain rate but the behavior in the strain hardening region is not affected largely by a strain rate. Soroushian and Choi [S8] concluded that the yield strength of steel is more strain-rate sensitive than the ultimate strength. The modulus of elasticity is independent of rate of straining. According to them, the most important factor influencing the strain rate effects is the static yield strength. The mechanical properties of steel with lower yield strength are more strain rate-sensitive than high strength steel. Malvar [M2] narrated that the DIF_s of yield and ultimate stress is inversely related to the yield stress itself. A formulation was proposed to find out the DIF_s as a function of strain rate and yield stress by fitting the available data in literature. This formulation is valid for yield stresses in between 290 and 710 MPa and for strain rates in between 10^{-4} s^{-1} and 10 s^{-1} . The formulation which gives the DIF_s for both yield and ultimate

stress is as follows:

$$DIF_s = (\dot{\varepsilon}/10^{-4})^\alpha \quad (7-22)$$

where for yield stress, $\alpha = \alpha_{fy}$; $\alpha_{fy} = 0.074 - 0.04 f_y / 414$; for ultimate stress, $\alpha = \alpha_{fu}$; $\alpha_{fu} = 0.019 - 0.009 f_y / 414$; $\dot{\varepsilon}$ is the strain rate in s^{-1} and f_y is static yield strength of reinforcement in MPa.

Strain-rate dependent material properties are succinctly presented above. Here, in tri-linear moment-curvature analysis, modified material properties are employed to obtain strain-rate dependent resistance function. The formulas proposed by CEB model code [C5] are used for compressive strength and ultimate compressive strain of concrete. For the DIF_t of tensile strength of concrete, the formula suggested by Malver and Ross [M5] is employed here. Furthermore, for yield strength of reinforcing bars, the equation narrated by Malvar [M2] has been used. The modified material properties are shown in **Table 7.2** for three considered strain rates (e.g., 1, 5 and $10 s^{-1}$). The resistance functions for static and three strain rates (e.g., 1, 5 and $10 s^{-1}$) are shown in **Fig. 7.6** for 3.8 and 5.7 series, respectively. Here, in the elasto-plastic SDOF analysis, resistance function for strain rate $1 s^{-1}$ will be used to evaluate the midspan deflection history as almost similar responses were obtained for these three strain-rate based resistance functions. Successively, comparison of the midspan deflection history will be made with the results by using static resistance function to assess the effect of strain rates on impact response.

Table 7.2: Modified material properties

Strain rate (s^{-1})	Compressive strength of concrete (MPa)	Ultimate compressive strain of concrete	Tensile strength of concrete (MPa)	Yield strength of longitudinal steel (MPa)
Static	40.0	0.00350	3.16	520.0
1	54.8	0.00430	4.80	644.8
5	57.2	0.00445	8.18	670.8
10	58.0	0.00452	10.30	681.2

Fig. 7.6: Resistance function of beam for static and various strain rate cases: (a) 3.8 series; (b) 5.7 series

Hence, to solve the dynamic responses, central difference method (i.e. explicit method) is adopted, which is suitable for impact problems. **Fig. 7.7** shows the comparison of the midspan displacement vs. time histories between experimental and elastic-plastic SDOF analysis results with and without strain rates. This is observed that the SDOF model can capture up to maximum midspan displacement reasonably well which occurs almost within 30ms. It is observed that the assumed strain rate (i.e. 1 s^{-1}) dependent SDOF model underestimate the maximum midspan displacement for some cases which could be unsafe in design. On the other hand, prediction by SDOF without strain rate always in the safe side. Therefore, strain rate dependent SDOF model must be used with cautious in design.

Fig. 7.7: Comparison of midspan displacement vs. time histories between experimental and elastic-plastic SDOF analysis results: (a) 3.8 series; (b) 5.7 series (continued)

Fig. 7.7: Comparison of midspan displacement vs. time histories between experimental and elastic-plastic SDOF analysis results: (a) 3.8 series; (b) 5.7 series

Moreover, to examine the usefulness of SDOF model, the analytical results are compared with other researcher's test results. Two types of beams were taken from Fujikake *et al.* [F4] for validation purpose. From the comparative plots shown in **Fig. 7.8** it can be demonstrated that the model can capture the test displacement-history quite satisfactorily.

Fig. 7.8: Comparison of midspan displacement vs. time histories between experimental and elastic-plastic SDOF analysis results: (a) S1616; (b) S1322
(continued)

Fig. 7.8: Comparison of midspan displacement vs. time histories between experimental and elastic-plastic SDOF analysis results: (a) S1616; (b) S1322

7.4 Summary

Simplified analytical models such as energy-balance and SDOF are employed in this chapter to capture the maximum impact responses of beam. For the case of lower drop heights, energy-balance model can capture the results quite accurately; however for the cases of intermediate to higher drop heights, the degree of accuracy decreases. The reason behind that in the theory it is assumed that the strain energy stored in the impacted structure equals the kinetic energy of the impactor at the instance of time when the maximum impact load occurs. This assumption is solely valid for quasi-static impact (i.e. low velocity or low drop heights) events as the amount of energy lost due to large plastic deformations and generation of heat and noise in high-speed impacts (i.e. higher drop heights) would significantly alter the energy balance.

Furthermore, elastic-plastic analysis of SDOF system of the beam is proposed to predict the midspan deflection-time history. Bilinear resistance functions of beam with and without strain rate effect are derived from tri-linear moment-curvature analysis. For a given drop-height and beam configuration, this is observed that the SDOF model can capture up to maximum midspan displacement with reasonable accuracy, which occurs almost within 30 ms. This is noticed that the assumed strain rate (i.e. 1 s^{-1}) dependent SDOF model underestimate the maximum midspan displacement for some cases which could be unsafe in design. On the other hand, prediction by SDOF without strain rate is always in the safe side. Therefore, strain rate dependent SDOF model must be used with caution in design.

CHAPTER 8

PERFORMANCE OF IMPACT-DAMAGED RC BEAMS

8.1 Introduction

RC structures have been the main structural type of choice in new infrastructural projects across the world. Due to the wide distribution and prevalence of such structures, it is almost inevitable that RC structures may be exposed to various types of impact loading during their service life. Progressive collapse could be triggered in these systems due to some impact loading thrusts in the structural elements. Therefore, it is of utmost importance to evaluate the residual resistance of structural components of an impact-damaged structure. A detailed understanding of the residual properties of the structural components would be helpful in determining the integrity and stability of impact-damaged structures during search and rescue operation. Moreover, to ensure that damaged structures will not fail catastrophically during their service life and maintain structural efficiency, it is essential to study the residual properties of the structures after impact. This can be further extended to evaluating proper strengthening or rehabilitation scheme. Assessment of the behavior of RC beam under drop-weight impact (Hughes and Beeby [H3]; Banthia [B1]; Kishi *et al.* [K5-K8]; Chen and May [C3]; Fujikake *et al.* [F4]; Bhatti *et al.* [B8]; Saatci and Vecchio [S1]; Tachibana *et al.* [T1]; Kishi and Mikami [K10]) loading has been well documented in literature and research on residual strength of blast-damaged RC columns (Bao and Li [B3], Wu *et al.* [W3]) has been prolific. However, evaluation of the residual response of impact-damaged beams is a lacking area. Thus, quasi-static response of the undamaged and impact-damaged specimens has been investigated. First, the beams were subjected to free-falling impact loadings from various drop-heights. Subsequently the pre-damaged specimens were tested quasi-statically to determine the load-midspan deflection responses. Finally, one beam from each group without any prior damage was tested under monotonic static loading to evaluate their original load-midspan deflection behavior. Comparative load vs. mid-span deflection profiles of undamaged and impact-damaged beams were plotted and residual resistance index (RRI) and

residual stiffness index (RSI) was computed. RRI is defined as the ratio of the ultimate resistance of undamaged and impact-damaged specimens; similarly RSI is the normalization of the secant stiffness of undamaged and impact-damaged beams. Here, the secant stiffness was computed by joining the origin to the peak load in the load vs. mid-span deformation curves brittle failure-type beams; whereas the yield resistance was taken as reference point to calculate the secant stiffness for specimens having ductility.

To expand the knowledge beyond the range of parameters investigated experimentally, a methodology to numerically simulate the drop-weight impact tests and post impact quasi-static residual resistance tests of damaged beams was developed. To simulate the impact response of beams, explicit finite element (FE) code LS-DYNA [L1] was used herein by considering the same boundary conditions and the exact shape of the impactor which were used during experiment. Afterward, restart analysis was performed by replacing the impactor with the plate to perform quasi-static simulation. The resulting deformation and damages of beam from the impact stages have been utilized to carry out residual resistance simulation. Results of the explicit FE simulations were compared against impact and post impact quasi-static residual resistance tests results. Simulation results seem to match the experimental results with reasonable accuracy. Hence, this numerical procedure could be valuable in providing some insight about the behavior of impact-damaged specimens before performing the actual tests in laboratory. Moreover, newly designed structural elements could be analyzed prior to their actual construction in site and existing impact-damaged structural elements could be retrofitted effectively by calculating their residual resistance following the above-mentioned numerical scheme.

8.2 Outline of Experiment

As mentioned earlier in **Chapter 5**, drop weight impact test program consisted of 30 RC beams which were divided into two groups in terms of their shear span-to-effective depth ratios (a/d -3.3, 3.8 and 5.7). Each group had three types of specimens distinguished by their shear reinforcement ratios. After finishing the

impact test of each beam, the damaged specimens were placed in the 3-point bending test set up at NDA, Japan to give quasi-static loading for capturing their residual resistance vs. midspan deflections curves. Details of the post impact residual resistance test set-up are shown in **Fig. 8.1**.

Fig. 8.1: Post impact residual resistance test set up

8.3 Summary of Experimental Results

In this section, load vs. midspan deflection curves of undamaged and impact-damaged beams are presented first, followed by the variation of residual resistance index (RRI) and residual stiffness index (RSI) in terms of various impact energies.

8.3.1 Load vs. midspan deflection curves

Quasi-static load vs. mid-span characteristics of undamaged and impact damaged specimens (damaged by impactor from different drop-heights) are shown in Fig. 8.2, 8.3 and 8.4. **Fig. 8.2** shows the curves of RC beams having shear span to effective depth ratio 3.3. It was observed that residual resistance of DR3.3_2.4 (without transverse reinforcements) almost reached the same resistance as undamaged specimens with same stiffness when the impactor struck the specimen from a drop-height of 0.15 m. However, for 0.3 and 0.6 m drop-height, both resistance and stiffness degradation was observed notably. Similarly, for DR3.3_2.4_0.12, drop-heights ranging from 0.6 to 1.2 m caused significant strength and stiffness reduction. For the same series of specimen with higher

amount of transverse reinforcements (i.e. DR3.3_2.4_0.56), resistance and stiffness degradation was less than that of above-mentioned two beams. It is noted that the static response of undamaged beams of this series was not evaluated experimentally rather the static response was determined by FE simulation. For 3.8 and 5.7 series beams, specimens with greater transverse reinforcements experienced less strength degradation compared to the other specimens.

Fig. 8.2: Quasi-static load vs. midspan deflection response of undamaged and impact-damaged beams of 3.3 series (continued)

Fig. 8.2: Quasi-static load vs. midspan deflection response of undamaged and impact-damaged beams of 3.3 series

Fig. 8.3: Quasi-static load vs. midspan deflection response of undamaged and impact-damaged beams of 3.8 series (continued)

Fig. 8.3: Quasi-static load vs. midspan deflection response of undamaged and impact-damaged beams of 3.8 series

Fig. 8.4: Quasi-static load vs. midspan deflection response of undamaged and impact-damaged beams of 5.7 series (continued)

Fig. 8.4: Quasi-static load vs. midspan deflection response of undamaged and impact-damaged beams of 5.7 series

8.3.2 Residual resistance index (RRI)

RRI is defined as the ratio of the peak resistance of impact-damaged to undamaged specimens ($RRI = R_{damaged}/R_{undamaged}$). Peak resistance can be defined as the maximum load in load vs. midspan deflection curves of undamaged and impact-damaged beams. **Fig. 8.5** shows the variation of RRI under different impact energies. For DR3.3_2.4 (without transverse reinforcements), when the input impact energies was increased from 0.44 to 1.77 kJ, the reduction in ultimate residual resistance (URR) was 4 and 46% respectively. URR is the maximum resistance of impact-damaged beams. When transverse reinforcement was around 0.12%, it can absorb more impact energy and the reduction in URR was lesser as compared to above-mentioned specimens. With a further increment of transverse reinforcement to 0.56%, URR was reduced 5 to 24% even when the input impact energy increased from 1.77 to 4.71 kJ. Moreover, the URR of singly reinforced beam for a/d -3.8 series specimens did not only reduce at all but also greater than the ultimate resistance of undamaged specimens. This is due to the change in the nature of failure mode where the undamaged specimen failed in shear whereas damaged one failed in flexure (i.e. strain hardening phenomenon was significant in impact-damaged beams) under the impact energies (i.e. 0.88 to 2.65 kJ) considered during testing. For doubly reinforced beams of the same specimen series, under low

impact energies (i.e. 1.77 and 2.65 kJ) their RRIs were almost similar; however for higher input energy (i.e. 3.53 kJ), specimens having higher amount of transverse reinforcements performed better. For a/d -5.7 series specimens, a systematic trend of degradation in RRI was observed; and with increasing impact energies, better performance was observed in specimens with more transverse reinforcements. In general, with the increasing amount of impact energy, decreasing trend in RRI is being observed. This trend is more pronounced in specimens with less transverse reinforcement. Few cases were observed where the URR of damaged specimens was more than the undamaged one; this may be due to the strain hardening effect of the steel reinforcing bars.

Fig. 8.5: Residual Resistance Index (RRI) of RC beams (a) 3.3 series; (b) 3.8 series and (c) 5.7 series under various impact energies

RRI could be used effectively to define the extents of damage of RC beam after impact loading. The damage index D is defined as:

$$D = (1 - RRI) \quad (9-1)$$

The degrees of damage are defined as follows:

$D = (0 - 0.2)$ Low damage (LD); $D = (0.2 - 0.5)$ Medium damage (MD);

$D = (0.5 - 0.8)$ High damage (HD); $D = (0.8 - 1)$ Severe damage (SD)

Shi *et al.* (2008) proposed almost similar type of damage index for RC columns damaged to blast loads and also mentioned that this definition is solely subjective. Thus, here the damage term for $D = (0.8 - 1)$ has been changed from ‘collapse’ to ‘severe damage’ as some researchers believe the shear failure mode as collapse stage. Hence, the degree of damage of the beam for particular impact energy can be calculated from Eq. (9-1). When $RRI > 1$, it could be concluded that the beam did not acquire any substantial damage or the developed hairline cracks might be closed as soon as the impact load died off.

8.3.3 Residual stiffness index (RSI)

RSI is defined as the ratio of the secant stiffness of impact-damaged to undamaged specimens ($RSI = K_{damaged} / K_{undamaged}$). Here, the secant stiffness was computed by joining the origin to the peak load in the load vs. mid-span deformation curves brittle failure-type beams (i.e. sharp fall in load vs. deformation curves after peak load); whereas the yield resistance was taken as reference point to calculate the same for specimens having ductility. The variation of residual stiffness index (RSI) of beams under different impact energies is shown in Fig. 8.6. For DR3.3_2.4 (without transverse reinforcements), when the input impact energies increased from 0.44 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 and 3.53 kJ respectively for specimens having 0.12% transverse reinforcements. The rate of reduction in stiffness for DR3.3_2.4_0.56 was much less drastic with the increasing impact energies (i.e. 1.77 to 4.71 kJ) as compared to the specimens of the same series. Thus, it is evident that transverse reinforcements inhibited significant degradation in stiffness. Around 20% stiffness reduction was perceived of SR3.8_0.8 for all input energies (0.88 to 2.65kJ) considered during

experiment. However, for the case of 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, consistent stiffness-degradation was seen for increasing drop-heights. For the same impact energy, stiffness-degradation was less in DR beams as compared to SR of 5.7 series beams.

Fig. 8.6: Residual Stiffness Index (RSI) of RC beams (a) 3.3 series; (b) 3.8 series and (c) 5.7 series under various impact energies

8.3.4 Failure pattern

Assessment of crack pattern or failure mode is of paramount importance in both impact and post impact quasi-static loading stages, as it would assist in determining the integrity and stability of impact-damaged structures. **Fig. 8.7** shows the crack profile of beams after post impact residual resistance test. For DR3.3_2.4, impact energies were varied from 0.59 to 4.7 kJ (i.e. drop-height varies from 0.15 to 1.2 m). No visible cracks were observed at impact energy of 0.59 kJ (i.e. 0.15 m);

however, diagonal shear cracks were perceived in post impact quasi-static test. With further increment of impact energies, diagonal cracks were observed and successively in residual test these cracks widened with much damaged in the impact regions. At 1.2 m (i.e. impact energy of 4.7 kJ) drop height, the beam was so severely damaged that post impact residual test could not be performed. For DR3.3_2.4_0.12, hair-line diagonal cracks were seen under low velocity impact whereas much wider cracks were discerned for high impact velocity. In residual tests, further development of diagonal cracks and widening of the existing diagonal cracks with concrete spalling were noticed. Impact energies were varied from 2.35 kJ to 6.28 kJ (i.e. 0.6 to 1.6 m) for drop-weight impact test of DR3.3_2.4_0.56. This beam showcases the significance of higher amount of transverse reinforcements by not only absorbing the higher amount of impact energies (i.e. approx. 34% higher than specimen having 0.12% shear reinforcements) but also performing better (i.e. by resisting catastrophic failure mode) in residual resistance test. However, some spalling in side cover concrete nearby impact region was observed under impact energy of 6.28 kJ (i.e. 1.6 m). Flexure failure was observed in all beams of a/d -3.8 series for all impact energies (i.e. 1.18-4.7 kJ) considered during impact tests. Ductile nature (i.e. no change in failure mode) persisted during residual test of the damaged beams however some additional flexural cracks developed or the existing flexural cracks became wider. Thus, beam having a/d of 3.8 and lower amount of longitudinal reinforcements (although less amount of shear reinforcements) could withstand the impact and post impact quasi-static load quite satisfactorily. Similarly, all beams of a/d -5.7 series failed in flexure under impact (i.e. considered impact energies of 1.18-2.35 kJ) and residual tests.

Fig. 8.7: Crack pattern of impact-damaged RC beams (a) 3.3 series; (b) 3.8 series and (c) 5.7 series under various drop-heights

8.4 Finite Element Framework

Detailed description of the simulation of beam subjected to drop-weight impact loading can be found in **Chapter 6**. After finishing the impact loading simulation, the impactor and two triangular plates were removed from the model. Then, 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 given to it. **Fig. 8.8** shows the FE model in impact loading and post-impact residual resistance stage. Here, in the restart analysis environments, resulting deformation and damages of beam from the impact simulation have been utilized to carry out residual resistance simulation.

Fig. 8.8: FE models of beam (a) impact loading stage; (b) post impact residual resistance stage

Similar 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. MAT CSCM CONCRETE (MAT 159) is usually recognized as Continuous Surface Cap Model, was used to model concrete and material model MAT PIECEWISE LINEAR PLASTICITY (MAT 024) is used to model reinforcing bars. Furthermore, to model the impactor, support rollers and triangular plates MAT_RIGID (MAT 020) was used from LS-DYNA [L1] material library.

8.5 Verification of Finite Element Analysis Results

The numerical simulation results of RC beams subjected to drop-weight impact loading are validated with the experimental outcomes. Successively, the numerical results of impact-damaged beam are validated to the test results. Verification of impact loading history under impact and residual resistance vs. mid-span deflection profile of impact damaged specimens and cracks patterns is presented in the following two sections.

8.5.1 Impact loading history and residual resistance vs. midspan deflection

Two types of tested beams are considered for validation of numerical results. Comparison of impact load history and post impact residual resistance vs. midspan deflection of DR3.8_0.8_0.11 under 0.6 m drop height is presented in **Figs. 8.9 (a) and (b)** respectively. Similarly, above-mentioned two responses are validated for DR5.7_1.6_0.20 under 0.45 m drop height through **Figs. 8.9 (c) and (d)**. It is observed that the results from the three-dimensional nonlinear FE model match the test results within an acceptable accuracy.

Fig. 8.9: Validation of impact load history and post impact residual resistance vs. midspan deflection (continued)

Fig. 8.9: Validation of impact load history and post impact residual resistance vs. midspan deflection

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

Assessment of crack patterns or failure mode of beam under drop-weight impact and post impact quasi-static loading bears utmost importance. The damage of the beams obtained from numerical simulation is shown by plotting the fringes of effective plastic strain. These effective plastic strain contours reveal the strain localization where failure propagates.

Fig. 8.10: Comparison of cracking pattern from experiment and fringes of effective plastic strain from simulation of beams after impact and residual test (continued)

(a) DR3.8_0.80_0.11

(b) DR5.7_1.6_0.20

Fringe Levels

Fig. 8.10: Comparison of cracking pattern from experiment and fringes of effective plastic strain from simulation of beams after impact and residual test

Fig. 8.10 shows the comparison of damage plot in between experimental and numerical simulation results. Comparative damage plots of two types of beams mentioned above are shown. From these comparisons, it can be enunciated that the damage plot of numerical simulation results can capture the experimental crack profiles (after impact and residual tests) quite satisfactorily.

8.6 Numerical Simulation Case-studies

After verification of the FE model against the experimental results, this section presents a parametric investigation to elucidate about the residual resistance of beams after impact damage. Some key parameters such as mass ratio (α) i.e. ratio of the mass of the beams to the impactor-mass (α : 0.26, 0.42 and 0.49), longitudinal reinforcement ratios (ρ_L : 0.8-2.4%), transverse reinforcement ratios (ρ_T : 0.12-0.67%), compressive strength of concrete (f'_c : 30, 40 and 50 MPa) are taken into account to study their influence on residual performance under various impact energies (E : 0.88-4.70 kJ). The parameter variation in various case studies is shown in **Table 8.1**. The beams used for numerical case studies have the same geometric properties (i.e. span length and cross-sections etc.) as experimental specimens.

Table 8.1: Parameter variations in case-studies

a/d	α	ρ_L (%)	ρ_T (%)	Impact Energy, E (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 and 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	
5.7	0.26	0.90, 1.60	0.15, 0.20, 0.31, 0.47, 0.67	1.17, 1.77, 2.35	

8.6.1 Effect of mass ratio (α)

Fig. 8.11 illustrates the effect of the mass ratios (α : 0.26, 0.42 and 0.49) on the RRI of beams under varying impact energies. It is noted that here the mass of the impactor has been kept constant (300 kg); however, different mass ratios were obtained due to the variation in span length and cross-sections of beams. Here, the

mass of beam of 3.3, 3.8 and 5.7 series are 126, 147.5 and 78.3 kg, respectively. Considering same amount of longitudinal reinforcement ratio and two amounts of transverse reinforcement ratio, two cases are presented in **Figs. 8.11 (a) and (b)**. It is evident that residual resistance will be in the higher side for the case of higher mass ratio under same impact energy.

Fig. 8.11: Effect of mass ratio (α) on RRI under various impact energies

8.6.2 Effect of longitudinal reinforcement ratio (ρ_L)

Fig. 8.12 shows the variation of RRI under various impact energies. To understand the influence of longitudinal reinforcement ratio on RRI, two cases are considered here. Under low impact energies, the effect of ρ_L seems to be insignificant. However, for high impact energies, specimen having low amount of ρ_L performs

better in terms of achieving residual resistance. Beams of comparatively lower amounts of longitudinal reinforcements exhibited only overall flexure failure, whereas flexure as well as local damage at impacting point occurred in beams with relatively higher amount of longitudinal reinforcements. This local damage initiates the further development of diagonal cracks during residual tests which successively reduce the residual resistance; while in the case of overall flexure-failure type beam, much ductile response was observed due to the strain hardening of the longitudinal tensile reinforcements.

Fig. 8.12: Effect of longitudinal reinforcement ratio (ρ_L) on RRI under various impact energies

8.6.3 Effect of transverse reinforcement ratio (ρ_T)

Effect of transverse reinforcement on RRI under varying amount of impact energies is presented in **Fig. 8.13**. It is evident that specimens having relatively higher amount of transverse reinforcements possess higher residual resistance. Under increasing height of the drop-weights, the difference in RRI between beam having low and high amount of shear reinforcements would be significant. Higher amount of transverse reinforcement contained beam resist the excessive impact damage and can produce much higher residual resistance by confinement to the core concrete and supplementing lateral restraint capacity against the buckling of the longitudinal reinforcements.

Fig. 8.13: Effect of transverse reinforcement ratio (ρ_T) on RRI under various impact energies

8.6.4 Effect of compressive strength of concrete (f'_c)

Beam of concrete compressive strength of 30, 40, 50 MPa was used to determine the effect of f'_c on RRI under varying impact energies. Three case studies are depicted in **Fig. 8.14**. Under low impact energy, the effect of f'_c on RRI quite minimal; however for increasing the impact energies, specimens having high concrete compressive strength perform better (i.e. yielding higher residual resistance) as compared to beam with low concrete compressive strength. It is noted that beam having high strength concrete is out of scope of this research.

Fig. 8.14: Effect of compressive strength of concrete (f'_c) on RRI under various impact energies (continued)

Fig. 8.14: Effect of compressive strength of concrete (f'_c) on RRI under various impact energies

8.7 Summary

In this chapter, experimental and numerical studies of impact-damaged of RC beams are demonstrated comprehensively and following conclusions can be drawn:

- (1) Two indices, residual resistance index (RRI) and residual stiffness index (RSI) were defined and discussed thoroughly to show their variation under various impact energies by analyzing the experimental data. RRI and RSI could be used effectively to delineate the extents of damage (i.e. resistance and stiffness degradation) of the beams after impact loading.
- (2) A methodology to numerically simulate the drop-weight impact tests and post impact quasi-static residual resistance tests of damaged beams was developed. The FE model has been successfully validated with the test results.
- (3) Upon successful verification of the FE model against the experimental results, a parametric study has been carried out to quantify the effect of various parameters on RRI. RRI will be on the higher side in the case of higher mass ratio under same impact energy. Under high impact energies, specimen having low amount of longitudinal reinforcement performs better in terms of achieving residual resistance. Specimens having relatively higher amount of transverse reinforcements

possess higher residual resistance after impact damage. Moreover, under increasing amount of impact energies, specimens having high concrete compressive strength perform better (i.e. yielding higher residual resistance) as compared to beam with low concrete compressive strength.

CHAPTER 9

CONCLUSIONS AND FUTURE WORK

9.1 Conclusions

The dynamic behavior of RC beams under varying rates of concentrated and drop-weight impact loading was investigated experimentally and numerically. Conclusions drawn from this research program are presented in the following section.

RC beam under varying rates of concentrated loading

Experimental investigation:

Conclusions drawn from the experimental investigation on RC beam under varying rates of concentrated loading are as follows:

(1) With the enhancement of loading rates, the ultimate load resistance, cracking stiffness and energy absorption of RC beams were found to increase correspondingly. When the loading rate progressed from low to high, an increasing trend in DIF was observed. In addition, specimens having smaller a/d ratio produced higher DIF as compared to the specimens having higher a/d ratio for low, medium and high loading rates. Peak strain rate was amplified by one order of magnitude (approximately, 10 times) as the loading rates progressed from low to high.

(2) For 3.3 series specimens, diagonal tension and/or shear tension type failure was observed in SR beams whereas diagonal tension and/or shear compression were the dominant failure mode for DR specimens. In all loading rates of the above-mentioned series, diagonal shear cracks originated from the mid-height of beam and propagated towards loading and support points. However, for DR specimens, diagonal shear cracks under medium and high loading rates propagated with much steeper angles towards the bottom surface of the beam as compared to static and

low loading rates. On the other hand, for 4.4 series specimens, ductile flexure type failure was observed for all loading rates.

(3) After observing the wide scatter of results, it can be concluded that there is a need of consensus among researchers in high-rate testing of RC beams. Test set up, frequency response of measurement system, beam geometry, reinforcing detailing and material properties should be kept similar to facilitate better analysis and effective comparisons.

Numerical investigation:

FE model was developed to evaluate the behavior of beams under varying rates of concentrated loading at midspan. Numerical parametric study yielded following conclusions:

(1) It was observed that DIF were on the higher side for beam with low reinforcement ratio. When the beam contained low amount of longitudinal reinforcements, the response was mainly influenced by the strain rate effect of longitudinal reinforcing steel which in turn causing an enhancement of the DIF. Specifically, for high loading rates, extent of damage was more severe for beams containing relatively higher amount of longitudinal reinforcements. Therefore, it can be concluded that change in failure mode may be observed (i.e. mainly in high rates considered here) when the amount of longitudinal reinforcement ratio is increased (i.e. approaching towards balanced-reinforcement ratio) in an under-reinforced beam.

(2) For beam containing more transverse reinforcements, DIF would be on the lower side. Furthermore, for medium and high loading rates, the beam containing lower amount of transverse reinforcements, diagonal cracks were formed with flexure cracks; whereas flexure type of failure was observed in beam having high amount of transverse reinforcements. Hence, the amount of transverse reinforcements had a significant effect in preventing the formation of diagonal cracks under medium and high loading rates. For beams having various concrete grades (30, 40 and 50 MPa), increasing trend in peak load was observed when the

loading rate progressed from static to high. Moreover, beam of high strength concrete (i.e. 50 MPa) yielded higher peak load for all loading rates as compared to relatively low strength concrete (i.e. 30, 40 MPa) beams.

(3) The yield strength of longitudinal reinforcements of beams is the key parameter, affecting the change in failure mode from flexure at static loading to shear at high rates. Due to the high rate sensitivity of low yield strength steel, the increment of yield strength of longitudinal reinforcements in medium and high rates was such a way that the bending resistance surpassed the shear resistance and eventually failure mode changed.

RC beam under drop-weight impact loading

Experimental assessment:

From the experimental assessment of RC beam under drop-weight impact loading, following conclusions can be made:

(1) Maximum impact load, maximum midspan deflection and time to reach maximum deflection were found to increase with the increment of drop-heights or impact velocities. Maximum strain rates of longitudinal tensile reinforcements vary between 1 and 7 s⁻¹ for the corresponding considered drop heights of 0.15 to 1.2 m (i.e. impact velocity varies from 1.72 to 4.85 m/s).

(2) The crack pattern and failure modes varied among the specimens depending on their static resistance. For statically shear-critical beams, (without transverse reinforcements) hairline diagonal cracks were observed at lowest drop height considered. With the enhancement of the drop-heights, width of the diagonal cracks and severity of local failure increases. At a maximum drop height considered, the damage was so catastrophic that the beam disintegrated into two components with extensive crushing and spalling of concrete at the impact region. For statically flexure-critical beams, flexural cracks beneath the impact point were observed at lowest drop height considered. With further increase in drop heights, flexural cracks became much wider and propagated vertically and crushed the compression concrete. At a maximum drop height considered, massive local failure occurred at

the impact region.

(3) Two empirical equations are proposed by analyzing a pool of data on impact loading documented by several researchers from the literature for both statically flexure and shear failure type beams. By specifying the maximum midspan deflection for each limit state of beam, the required static bending and shear resistance can be determined for designing the beam subjected to input impact energy.

Numerical investigation:

After verifying the numerical simulation results against the experimental results, parametric investigation was carried out to elucidate the influence of some key parameters on impact responses. Under constant impact energy, impacts caused by high mass with low velocity result in smaller maximum impact load but higher maximum midspan deflection of beam and vice versa. Maximum impact load increases and maximum midspan deflection of beam decreases with the enhancement of the longitudinal reinforcement ratios. With increasing amount of longitudinal reinforcements severity in failure pattern (including diagonal and punching shear cracks) is observed. Moreover, with the enhancement of concrete compressive strength from 30 to 50 MPa, an increasing trend in maximum impact load was observed whereas the opposite phenomenon was noticed for the case of maximum midspan deflections. Maximum impact load is in the higher side and maximum midspan deflection is in lower side for fixed-end beam as compared to pinned-end beam.

Analytical investigation:

Simplified analytical models such as energy-balance and SDOF are employed to capture the maximum impact responses of beam. For the case of lower drop heights, energy-balance model can capture the results quite accurately; however for the cases of intermediate to higher drop heights, the degree of accuracy decreases. Furthermore, elastic-plastic analysis of SDOF model is proposed to predict the midspan deflection-time history of beam under impact loading.

Residual performance of impact-damaged RC beam

(1) Two indices, residual resistance index (RRI) and residual stiffness index (RSI) were defined and discussed thoroughly to show their variation under various impact energies by analyzing the experimental data. RRI and RSI could be used effectively to delineate the extents of damage (i.e. resistance and stiffness degradation) of the beams after impact loading.

(2) A methodology to numerically simulate the drop-weight impact tests and post impact quasi-static residual resistance tests of damaged beams was developed. Upon successful verification of the FE model against the experimental results, a parametric study has been carried out to quantify the effect of various parameters on RRI. RRI will be on the higher side for the case of higher mass ratio under same impact energy. Under high impact energies, specimen having low amount of longitudinal reinforcement performs better in terms of achieving residual resistance. Specimens having relatively higher amount of transverse reinforcements possess higher residual resistance after impact damage. Moreover, under increasing amount of impact energies, specimens having high concrete compressive strength perform better (i.e. yielding higher residual resistance) as compared to beam with low concrete compressive strength.

In summary, this study has explored various engineering methodology to assess the performance of RC beams under varying rates of concentrated and impact loadings. Useful information has been provided in these two loading environments through experimental tests, FE analyses and analytical approaches. Substantial amount of test data are provided in this thesis that can be used further for the development of new analytical methods. Some points are drawn regarding the testing of beams under varying loading rates which would be useful during the planning of future test program. Furthermore, this study could be employed for the development of future model code and guidelines regarding low velocity impact effect on the performance assessment of RC structural components.

9.2 Future work

In the light of experience gained from this research program, the following recommendations can be made for future study:

(1) An investigation should be carried out on the structural behavior of RC beam under varying rates of concentrated loading, in which the following variables need to be considered:

- Concrete compressive strength and yield strength of reinforcement bars;
- Size and geometry of beam;
- Boundary conditions;
- Asymmetric loading;

(2) Influence of loading history (mainly the loading rates corresponds to impact regime) on structural behavior should be explored.

(3) Efforts can be driven towards determining the impact resistance of RC beam through a drop-weight test program, covering a wider range of parameters.

(4) The effects of various impact parameters, such as mass and velocity of impactor, impactor shape and impact interface should be investigated. Moreover, the influence of structural mass and geometry should be evaluated by testing RC beams with varying span lengths.

(5) Behavior of other structural components such as RC columns, slabs under drop-weight impact loading should be examined in details. Moreover, impact response of prestressed structural components can be investigated.

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APPENDIX-A: Beam Details and Response under Varying Rates of Concentrated Loading

Reference	Beam mark	f'_c (MPa)	Beam dimensions				Shear span to eff. depth ratio	Longitudinal reinforcement		Shear reinforcement		Loading rate (m/s)	Maximum resistance (kN)	Failure modes			
								Tensile (T) & Compressive (C)									
			h (mm)	b (mm)	d (mm)	a (mm)		a/d	Ratio (%)	f_{Ly} (MPa)	Ratio (%)				f_{Ty} (MPa)		
Mutsuyoshi & Machida [M10]	A 1-S	29.0	200	150	160	880	5.5	0.89 ^T	382	-	-	1×10^{-4}	23.6	Flexure			
	A 1-D							-	-	2.63×10^{-1}	30.8						
	A 2-S	32.7						448	2.8	0.89 ^T	432	-	-		3×10^{-5}	51.7	Shear
	A 2-D											-	-		2.33×10^{-1}	72.4	
	A 3-S	31.9						496	3.1	1.05 ^T	381	-	-		3×10^{-5}	50.3	Flexure
	A 3-D											-	-		1.15×10^{-1}	71.5	
	A 4-S	34				752	4.7	1.70 ^T	360	-	-	2×10^{-5}	45.4	Flexure			
	A 4-D									-	-	2.18×10^{-1}	59.8				
	A 5-S	35.1				784	4.9		393	-	-	5×10^{-5}	49.0	Shear			
	A 5-D									-	-	2.3×10^{-1}	61.6				
	A 6-S	34				150	100	120	684	5.7	2.11 ^T	456	-	-	2×10^{-5}	23.6	Flexure
	A 6-D										-		-	7.63×10^{-1}	36.5	Shear	
	A 7-S	31.6							564	4.7	1.78 ^T	398	-	-	3×10^{-5}	30.4	Flexure
	A 7-D												-	-	1.67×10^{-1}	38.2	
	B 1-S	26.8	480	4.0	1.19 ^T				398	-	-	2×10^{-5}	25.5	Flexure			
	B 1-D									-	-	2.67×10^{-1}	31.3	Shear			
	B 2-S	28.6	588	4.9	1.78 ^T				393	-	-	3×10^{-5}	27.9	Flexure			
	B 2-D									-	-	5.67×10^{-1}	33.7				
	B 3-S	23.2	456	3.8	2.11 ^T				506	-	-	1.43×10^{-1}	49.2	Shear			
	B 3-D									-	-	2.77×10^{-1}	47				
	B 4-S	28.6	456	3.8	2.11 ^T	506	-	-	1.43×10^{-1}	49.2	Shear						
	B 4-D						-	-	2.77×10^{-1}	47							
	B 5-S	28.0	456	3.8	2.11 ^T	506	-	-	1.43×10^{-1}	49.2	Shear						
	B 5-D						-	-	2.77×10^{-1}	47							

Reference	Beam mark	f'_c (MPa)	Beam dimensions				Shear span to eff. depth ratio	Longitudinal reinforcement		Shear reinforcement		Loading rate (m/s)	Maximum resistance (kN)	Failure modes
								Tensile (T) & Compressive (C)						
			h (mm)	b (mm)	d (mm)	a (mm)		a/d	Ratio (%)	f_{Ly} (MPa)	Ratio (%)			
	C 1-S	23.7	150	100	120	684	5.7	4.78 ^T	320	-	-	2×10^{-5}	33.4	Shear
	C 2-D									-	-	1.89×10^{-1}	50.3	
	C 3-D									-	-	4.08×10^{-1}	52.6	
Kulkarni and Shah [K12]	B4JL25-S	41.5	178	102	152	836	5.5	1.37 ^T	518	-	-	7.1×10^{-6}	41.7	Flexure
	B4JL25-H	41.5				836	5.5					3.8×10^{-1}	46.4	
	B3OC25-S	46.2				760	5					7.1×10^{-6}	40.5	
	B3OC25-H	46.2				760	5					3.8×10^{-1}	46.7	
	B4JL20-S	41.9				760	5					7.1×10^{-6}	39.1	Shear
	B4JL20-H	41.9				760	5					3.8×10^{-1}	44.9	Flexure
	B3SE03-S	45				684	4.5					7.1×10^{-6}	45.9	Shear
	B3SE03-H	45				684	4.5					3.8×10^{-1}	51.6	Flexure
	B3DE03-S	43				684	4.5					7.1×10^{-6}	45.3	
	B3DE03-H	43				684	4.5					3.8×10^{-1}	50.9	
	B3NO15-S	43				608	4					7.1×10^{-6}	43.9	
	B3NO15-H	43				608	4					3.8×10^{-1}	56.8	Flexure
	B3NO30-S	45				532	3.5					7.1×10^{-6}	49.9	Shear
	B3NO30-H	45				532	3.5					3.8×10^{-1}	55.8	Shear
Fujikake <i>et al.</i> [F4]	S1616	42	250	150	210	700	3.33	1.27 ^{T&C}	426	0.5	295	5×10^{-4}	115	Flexure
	S1616							1.27 ^{T&C}	426			2×10^0	135	
	S2222							2.41 ^{T&C}	418			5×10^{-4}	200	
	S2222							2.41 ^{T&C}	418			2×10^0	243	
	S1322							2.4&0.84	397			5×10^{-4}	180	
	S1322							2.4&0.84	397			2×10^0	246	

Reference	Beam mark	f'_c (MPa)	Beam dimensions				Shear span to eff. depth ratio	Longitudinal reinforcement		Shear reinforcement		Loading rate (m/s)	Maximum resistance (kN)	Failure modes
								Tensile (T) & Compressive (C)						
			h (mm)	b (mm)	d (mm)	a (mm)		a/d	Ratio (%)	f_{Ly} (MPa)	Ratio (%)			
Adhikary <i>et al.</i> [A3]	RC3_S0_S1	40	250	150	210	700	3.33	2.41 ^{T&C}	371	0	342	4×10^{-4}	93.2	Shear
	RC3_S0_S2											4×10^{-4}	81.3	
	RC3_S0_L1											4×10^{-2}	105.8	
	RC3_S0_L2											4×10^{-2}	102.5	
	RC3_S0_M1											4×10^{-1}	138	
	RC3_S0_M2											4×10^{-1}	126.1	
	RC3_S0_H1											2×10^0	140.6	
	RC3_S0_H2											2×10^0	147.1	
	RC3_S12_S1									0.12		4×10^{-4}	120	
	RC3_S12_S2											4×10^{-4}	120.5	
	RC3_S12_L1											4×10^{-2}	148.8	
	RC3_S12_L2											4×10^{-2}	139.5	
	RC3_S12_M1											4×10^{-1}	156.2	
	RC3_S12_M2											4×10^{-1}	164.7	
	RC3_S12_H1											2×10^0	168.7	
	RC3_S12_H2											2×10^0	180.9	
	RC3_S56_S1									0.56		4×10^{-4}	200.4	Flexure
	RC3_S56_S2											4×10^{-4}	202	
	RC3_S56_L1											4×10^{-2}	211.7	
	RC3_S56_L2											4×10^{-2}	217.9	
	RC3_S56_M1											4×10^{-1}	222.6	
	RC3_S56_M2											4×10^{-1}	223.1	
	RC3_S56_H1											2×10^0	235.6	
	RC3_S56_H2											2×10^0	226.5	

Reference	Beam mark	f'_c (MPa)	Beam dimensions				Shear span to eff. depth ratio	Longitudinal reinforcement		Shear reinforcement		Loading rate (m/s)	Maximum resistance (kN)	Failure modes
								Tensile (T) & Compressive (C)						
			h (mm)	b (mm)	d (mm)	a (mm)		a/d	Ratio (%)	f_{Ly} (MPa)	Ratio (%)			
Adhikary <i>et al.</i> [A4]	RC1.9_S0_S1	40	250	150	210	700	1.9	2.41 ^{T&C}	371	0	342	4×10^{-4}	138.4	Shear
	RC1.9_S0_S2											4×10^{-4}	170.1	
	RC1.9_S0_L1											4×10^{-2}	215.6	
	RC1.9_S0_L2											4×10^{-2}	246.5	
	RC1.9_S0_M1											4×10^{-1}	290.5	
	RC1.9_S0_M2											4×10^{-1}	-	
	RC1.9_S0_H1											2×10^0	353.6	
	RC1.9_S0_H2											2×10^0	-	
	RC1.9_S42_S1									0.42		4×10^{-4}	271.9	
	RC1.9_S42_S2											4×10^{-4}	-	
	RC1.9_S42_L1											4×10^{-2}	-	
	RC1.9_S42_L2											4×10^{-2}	300.6	
	RC1.9_S42_M1											4×10^{-1}	361.3	
	RC1.9_S42_M2											4×10^{-1}	-	
	RC1.9_S42_H1											2×10^0	417	
	RC1.9_S42_H2											2×10^0	-	
	RC1.9_S84_S1									0.84		4×10^{-4}	329.8	
	RC1.9_S84_S2											4×10^{-4}	333.8	
	RC1.9_S84_L1											4×10^{-2}	-	
	RC1.9_S84_L2											4×10^{-2}	378.2	
	RC1.9_S84_M1											4×10^{-1}	419.4	
	RC1.9_S84_M2											4×10^{-1}	422.1	
	RC1.9_S84_H1											2×10^0	447.9	
	RC1.9_S84_H2											2×10^0	445.5	

APPENDIX-B: Beam Geometry and Reinforcement Details for Drop-weight Impact Loading

Beam Mark	Beam characteristics						a/d	L/h	Longitudinal reinforcement						Shear reinforcement			R_s/R_b
	f'_c (MPa)	h (mm)	b (mm)	d (mm)	a (mm)	L (mm)			Top			Bottom			Dia. (mm)	Ratio (%)	f_{Ty} (MPa)	
									Dia. (mm)	Ratio (%)	f_{Ly} (MPa)	Dia. (mm)	Ratio (%)	f_{Ly} (MPa)				
Kishi <i>et al.</i> [K4]																		
A19	26.5	240	160	190	1000	2500	5.26	10.42	19	1.88	373	19	1.88	373	6	0.396	373	3.1
A22	26.5	240	160	190	1000	2500	5.26	10.42	22	2.55	373	22	2.55	373	6	0.396	373	1.9
B10	26.5	220	200	170	1000	2500	5.88	11.36	10	0.42	373	10	0.42	373	6	0.317	373	5.85
B13	26.5	220	200	170	1000	2500	5.88	11.36	13	0.75	373	13	0.75	373	6	0.317	373	3.96
B19	26.5	220	200	170	1000	2500	5.88	11.36	19	1.69	373	19	1.69	373	6	0.317	373	2.13
C10	26.5	160	160	120	1000	2500	8.33	15.63	10	0.74	373	10	0.74	373	6	0.396	373	6.04
C13	26.5	160	160	120	1000	2500	8.33	15.63	13	1.32	373	13	1.32	373	6	0.396	373	4.36
C19	26.5	160	160	120	1000	2500	8.33	15.63	19	2.98	373	19	2.98	373	6	0.396	373	2.3
Kishi <i>et al.</i> [K6]																		
A24	33	250	150	210	500	1400	2.4	5.6	N/A	N/A	N/A	19	1.8	393	N/A	N/A	N/A	0.43
	33	250	150	210	500	1400	2.4	5.6	N/A	N/A	N/A	19	1.8	393	N/A	N/A	N/A	0.43
	33	250	150	210	500	1400	2.4	5.6	N/A	N/A	N/A	19	1.8	393	N/A	N/A	N/A	0.43
	33	250	150	210	500	1400	2.4	5.6	N/A	N/A	N/A	19	1.8	393	N/A	N/A	N/A	0.43
	33	250	150	210	500	1400	2.4	5.6	N/A	N/A	N/A	19	1.8	393	N/A	N/A	N/A	0.43
A36	33	250	150	210	750	1900	3.6	7.6	N/A	N/A	N/A	19	1.8	393	N/A	N/A	N/A	0.64
	33	250	150	210	750	1900	3.6	7.6	N/A	N/A	N/A	19	1.8	393	N/A	N/A	N/A	0.64
	33	250	150	210	750	1900	3.6	7.6	N/A	N/A	N/A	19	1.8	393	N/A	N/A	N/A	0.64
	33	250	150	210	750	1900	3.6	7.6	N/A	N/A	N/A	19	1.8	393	N/A	N/A	N/A	0.64
A48	33	250	150	210	1000	2400	4.8	9.6	N/A	N/A	N/A	19	1.8	393	N/A	N/A	N/A	0.86
	33	250	150	210	1000	2400	4.8	9.6	N/A	N/A	N/A	19	1.8	393	N/A	N/A	N/A	0.86
	33	250	150	210	1000	2400	4.8	9.6	N/A	N/A	N/A	19	1.8	393	N/A	N/A	N/A	0.86

Beam Mark	Beam characteristics						a/d	L/h	Longitudinal reinforcement						Shear reinforcement			R_s/R_b
	f'_c	h	b	d	a	L			Top			Bottom			Dia. (mm)	Ratio (%)	f_{Ty} (MPa)	
	(MPa)	(mm)	(mm)	(mm)	(mm)	(mm)			Dia. (mm)	Ratio (%)	f_{Ly} (MPa)	Dia. (mm)	Ratio (%)	f_{Ly} (MPa)				
Kishi <i>et al.</i> [K6]																		
A48	33	250	150	210	1000	2400	4.8	9.6	N/A	N/A	N/A	19	1.8	393	N/A	N/A	N/A	0.86
B24	33	250	150	210	500	1400	2.4	5.6	N/A	N/A	N/A	13	0.84	393	N/A	N/A	N/A	0.68
	33	250	150	210	500	1400	2.4	5.6	N/A	N/A	N/A	13	0.84	393	N/A	N/A	N/A	0.68
	33	250	150	210	500	1400	2.4	5.6	N/A	N/A	N/A	13	0.84	393	N/A	N/A	N/A	0.68
	33	250	150	210	500	1400	2.4	5.6	N/A	N/A	N/A	13	0.84	393	N/A	N/A	N/A	0.68
	33	250	150	210	500	1400	2.4	5.6	N/A	N/A	N/A	13	0.84	393	N/A	N/A	N/A	0.68
B36	33	250	150	210	750	1900	3.6	7.6	N/A	N/A	N/A	13	0.84	393	N/A	N/A	N/A	1.03
	33	250	150	210	750	1900	3.6	7.6	N/A	N/A	N/A	13	0.84	393	N/A	N/A	N/A	1.03
	33	250	150	210	750	1900	3.6	7.6	N/A	N/A	N/A	13	0.84	393	N/A	N/A	N/A	1.03
	33	250	150	210	750	1900	3.6	7.6	N/A	N/A	N/A	13	0.84	393	N/A	N/A	N/A	1.03
B48	33	250	150	210	1000	2400	4.8	9.6	N/A	N/A	N/A	13	0.84	393	N/A	N/A	N/A	1.37
	33	250	150	210	1000	2400	4.8	9.6	N/A	N/A	N/A	13	0.84	393	N/A	N/A	N/A	1.37
	33	250	150	210	1000	2400	4.8	9.6	N/A	N/A	N/A	13	0.84	393	N/A	N/A	N/A	1.37
	33	250	150	210	1000	2400	4.8	9.6	N/A	N/A	N/A	13	0.84	393	N/A	N/A	N/A	1.37
	33	250	150	210	1000	2400	4.8	9.6	N/A	N/A	N/A	13	0.84	393	N/A	N/A	N/A	1.37
Bhatti and Kishi [B6]																		
A37	41.2	400	200	350	1000	2400	2.86	6	35	2.75	373	35	2.75	373	6	0.21	373	0.57
A46	41.2	400	200	350	1000	2400	2.86	6	35	2.75	373	35	2.75	373	6	0.21	373	0.57
A56	41.2	400	200	350	1000	2400	2.86	6	35	2.75	373	35	2.75	373	6	0.21	373	0.57
A65	41.2	400	200	350	1000	2400	2.86	6	35	2.75	373	35	2.75	373	6	0.21	373	0.57
A74	41.2	400	200	350	1000	2400	2.86	6	35	2.75	373	35	2.75	373	6	0.21	373	0.57

Beam Mark	Beam characteristics						a/d	L/h	Longitudinal reinforcement						Shear reinforcement			R_s/R_b
	f'_c (MPa)	h (mm)	b (mm)	d (mm)	a (mm)	L (mm)			Top			Bottom			Dia. (mm)	Ratio (%)	f_{Ty} (MPa)	
									Dia. (mm)	Ratio (%)	f_{Ly} (MPa)	Dia. (mm)	Ratio (%)	f_{Ly} (MPa)				
Bhatti and Kishi [B6]																		
A84	41.2	400	200	350	1000	2400	2.86	6	35	2.75	373	35	2.75	373	6	0.21	373	0.57
B37	41.2	400	200	350	1000	2400	2.86	6	35	2.75	373	35	2.75	373	6	0.42	373	0.76
B46	41.2	400	200	350	1000	2400	2.86	6	35	2.75	373	35	2.75	373	6	0.42	373	0.76
B65	41.2	400	200	350	1000	2400	2.86	6	35	2.75	373	35	2.75	373	6	0.42	373	0.76
B74	41.2	400	200	350	1000	2400	2.86	6	35	2.75	373	35	2.75	373	6	0.42	373	0.76
B84	41.2	400	200	350	1000	2400	2.86	6	35	2.75	373	35	2.75	373	6	0.42	373	0.76
B93	41.2	400	200	350	1000	2400	2.86	6	35	2.75	373	35	2.75	373	6	0.42	373	0.76
Fujikake <i>et al.</i> [F4]																		
S1616	42	250	150	210	700	1700	3.3	6.8	16	1.28	426	16	1.28	426	10	1.4	295	2.55
	42	250	150	210	700	1700	3.3	6.8	16	1.28	426	16	1.28	426	10	1.4	295	2.55
	42	250	150	210	700	1700	3.3	6.8	16	1.28	426	16	1.28	426	10	1.4	295	2.55
	42	250	150	210	700	1700	3.3	6.8	16	1.28	426	16	1.28	426	10	1.4	295	2.55
S1322	42	250	150	210	700	1700	3.3	6.8	13	0.84	397	22	2.41	418	10	1.4	295	1.51
	42	250	150	210	700	1700	3.3	6.8	13	0.84	397	22	2.41	418	10	1.4	295	1.51
	42	250	150	210	700	1700	3.3	6.8	13	0.84	397	22	2.41	418	10	1.4	295	1.51
	42	250	150	210	700	1700	3.3	6.8	13	0.84	397	22	2.41	418	10	1.4	295	1.51
S2222	42	250	150	210	700	1700	3.3	6.8	22	2.41	418	22	2.41	418	10	1.4	295	1.51
	42	250	150	210	700	1700	3.3	6.8	22	2.41	418	22	2.41	418	10	1.4	295	1.51
	42	250	150	210	700	1700	3.3	6.8	22	2.41	418	22	2.41	418	10	1.4	295	1.51
	42	250	150	210	700	1700	3.3	6.8	22	2.41	418	22	2.41	418	10	1.4	295	1.51

Beam mark	Beam characteristics						a/d	L/h	Longitudinal reinforcement						Shear reinforcement			R_s/R_b
	f'_c	h	b	d	a	L			Top			Bottom			Dia. (mm)	Ratio (%)	f_{Ty} (MPa)	
	(MPa)	(mm)	(mm)	(mm)	(mm)	(mm)			Dia. (mm)	Ratio (%)	f_{Ly} (MPa)	Dia. (mm)	Ratio (%)	f_{Ly} (MPa)				
Chen and May [C3]																		
A1	49.2	200	100	175	1350	3000	7.7	15	6	0.32	510	12	1.29	510	6	0.28	250	1.48
	49.2	200	100	175	1350	3000	7.7	15	6	0.32	510	12	1.29	510	6	0.28	250	1.48
	49.2	200	100	175	1350	3000	7.7	15	6	0.32	510	12	1.29	510	6	0.28	250	1.48
	49.2	200	100	175	1350	3000	7.7	15	6	0.32	510	12	1.29	510	6	0.28	250	1.48
	45.8	200	100	175	1350	3000	7.7	15	6	0.32	510	12	1.29	510	6	0.28	250	1.44
	45.8	200	100	175	1350	3000	7.7	15	6	0.32	510	12	1.29	510	6	0.28	250	1.44
A1*	33.6	200	100	175	1350	3000	7.7	15	6	0.32	510	12	1.29	510	6	0.28	250	1.26
A2	45.8	200	100	175	1350	3000	7.7	15	6	0.32	510	12	1.29	510	6	0.28	250	1.44
	42.8	200	100	175	1350	3000	7.7	15	6	0.32	510	12	1.29	510	6	0.28	250	1.38
A3	35.6	200	100	175	750	1800	4.3	9	6	0.32	510	12	1.29	510	6	0.28	250	0.72
B1	33.6	200	100	175	1350	3000	7.7	15	6	0.32	510	12	1.29	510	6	0.28	250	1.26
	33.6	200	100	175	1350	3000	7.7	15	6	0.32	510	12	1.29	510	6	0.28	250	1.26
B1*	45.8	200	100	175	1350	3000	7.7	15	6	0.32	510	12	1.29	510	6	0.28	250	1.44
B2	35.6	200	100	175	750	1800	4.3	9	6	0.32	510	12	1.29	510	6	0.28	250	0.72
	35.6	200	100	175	750	1800	4.3	9	6	0.32	510	12	1.29	510	6	0.28	250	0.72
B3	42.8	200	100	175	1350	3000	7.7	15	6	0.32	510	12	1.29	510	6	0.28	250	1.38
B3*	33.6	200	100	175	1350	3000	7.7	15	6	0.32	510	12	1.29	510	6	0.28	250	1.26
B4	35.6	200	100	175	750	1800	4.3	9	6	0.32	510	12	1.29	510	6	0.28	250	0.72

Beam mark	Beam characteristics						a/d	L/h	Longitudinal reinforcement						Shear reinforcement			R_s/R_b
	f'_c (MPa)	h (mm)	b (mm)	d (mm)	a (mm)	L (mm)			Top			Bottom			Dia. (mm)	Ratio (%)	f_{Ty} (MPa)	
									Dia. (mm)	Ratio (%)	f_{Ly} (MPa)	Dia. (mm)	Ratio (%)	f_{Ly} (MPa)				
Saatci and Vecchio [S1]																		
SS0a	50.1	410	250	357	1500	4880	4.2	11.9	30	1.58	464	30	1.58	464	6	N/A	N/A	0.67
SS0b	50.1	410	250	357	1500	4880	4.2	11.9	30	1.58	464	30	1.58	464	6	N/A	N/A	0.67
SS1a	44.7	410	250	357	1500	4880	4.2	11.9	30	1.58	464	30	1.58	464	6	0.1	605	1.08
SS1b	44.7	410	250	357	1500	4880	4.2	11.9	30	1.58	464	30	1.58	464	6	0.1	605	1.08
SS2a	47	410	250	357	1500	4880	4.2	11.9	30	1.58	464	30	1.58	464	6	0.2	605	1.28
SS2b	47	410	250	357	1500	4880	4.2	11.9	30	1.58	464	30	1.58	464	6	0.2	605	1.28
SS3a	46.7	410	250	357	1500	4880	4.2	11.9	30	1.58	464	30	1.58	464	6	0.3	605	1.89
SS3b	46.7	410	250	357	1500	4880	4.2	11.9	30	1.58	464	30	1.58	464	6	0.3	605	1.89
Tachibana <i>et al.</i> [T1]																		
A1	24	250	150	210	500	1400	2.38	5.6	13	0.843	345	13	0.843	345	6	0.75	295	1.4
A2-1	24	250	150	210	1000	2400	4.76	9.6	13	0.843	345	13	0.843	345	6	0.75	295	2.7
A2-2	24	250	150	210	1000	2400	4.76	9.6	13	0.843	345	13	0.843	345	6	0.75	295	2.7
A2-3	24	250	150	210	1000	2400	4.76	9.6	13	0.843	345	13	0.843	345	6	0.75	295	2.7
A2-4	24	250	150	210	1000	2400	4.76	9.6	13	0.843	345	13	0.843	345	6	0.75	295	2.7
A2-5	24	250	150	210	1000	2400	4.76	9.6	13	0.843	345	13	0.843	345	6	0.75	295	2.7
A2-6	24	250	150	210	1000	2400	4.76	9.6	13	0.843	345	13	0.843	345	6	0.75	295	2.7
A2-7	24	250	150	210	1000	2400	4.76	9.6	13	0.843	345	13	0.843	345	6	0.75	295	2.7
A2-8	24	250	150	210	1000	2400	4.76	9.6	13	0.843	345	13	0.843	345	6	0.75	295	2.7
A2-9	24	250	150	210	1000	2400	4.76	9.6	13	0.843	345	13	0.843	345	6	0.75	295	2.7
A2-10	24	250	150	210	1000	2400	4.76	9.6	13	0.843	345	13	0.843	345	6	0.75	295	2.7
A2-11	24	250	150	210	1000	2400	4.76	9.6	13	0.843	345	13	0.843	345	6	0.75	295	2.7

Beam mark	Beam characteristics						a/d	L/h	Longitudinal reinforcement						Shear reinforcement			R_s/R_b
	f'_c (MPa)	h (mm)	b (mm)	d (mm)	a (mm)	L (mm)			Top			Bottom			Dia. (mm)	Ratio (%)	f_{Ty} (MPa)	
									Dia. (mm)	Ratio (%)	f_{Ly} (MPa)	Dia. (mm)	Ratio (%)	f_{Ly} (MPa)				
Tachibana <i>et al.</i> [T1]																		
A2-12	24	250	150	210	1000	2400	4.76	9.6	13	0.843	345	13	0.843	345	6	0.75	295	2.7
A2-13	24	250	150	210	1000	2400	4.76	9.6	13	0.843	345	13	0.843	345	6	0.75	295	2.7
A2-14	24	250	150	210	1000	2400	4.76	9.6	13	0.843	345	13	0.843	345	6	0.75	295	2.7
A4	24	250	150	210	2000	4400	3.52	17.6	13	0.843	345	13	0.843	345	6	0.75	295	5.5
B	24	150	300	110	1000	2400	9.09	16	13	0.804	345	13	0.804	345	6	0.37	295	2.1
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 [K9]																		
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
G2L-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
G2L-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
G2L-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
G3-1	34.6	250	150	210	1000	2400	4.76	9.6	13	0.8	393	13	0.8	393	6	0.13	295	3.51
G3-2	34.6	250	150	210	1000	2400	4.76	9.6	13	0.8	393	13	0.8	393	6	0.13	295	3.51
G3-3	34.6	250	150	210	1000	2400	4.76	9.6	13	0.8	393	13	0.8	393	6	0.13	295	3.51

Beam mark	Beam characteristics						a/d	L/h	Longitudinal reinforcement						Shear reinforcement			R_s/R_b
	f'_c (MPa)	h (mm)	b (mm)	d (mm)	a (mm)	L (mm)			Top			Bottom			Dia. (mm)	Ratio (%)	f_{Ty} (MPa)	
									Dia. (mm)	Ratio (%)	f_{Ly} (MPa)	Dia. (mm)	Ratio (%)	f_{Ly} (MPa)				
Kishi and Mikami [K9]																		
G4-1	32.3	250	150	210	1000	2400	4.76	9.6	13	0.8	373	13	0.8	373	6	0.13	295	3.52
G4-2	32.3	250	150	210	1000	2400	4.76	9.6	13	0.8	373	13	0.8	373	6	0.13	295	3.52
G5-1	39.2	300	200	260	1500	3400	5.77	11.33	19	1.1	379	19	1.1	379	6	0.07	295	2.85
G5-2	39.2	300	200	260	1500	3400	5.77	11.33	19	1.1	379	19	1.1	379	6	0.07	295	2.85
G6-1	34.7	250	250	210	1000	2400	4.76	9.6	19	1.09	392	19	1.09	392	6	0.13	295	2.19
G7-1	34.7	250	250	210	1500	3400	7.14	13.6	19	1.09	392	19	1.09	392	6	0.09	295	2.78
G7-2	34.7	250	250	210	1500	3400	7.14	13.6	19	1.09	392	19	1.09	392	6	0.09	295	2.78
G8-1	34.7	200	200	160	1000	2400	6.25	12	25	3.17	383	25	3.17	383	6	0.16	295	1.55
G9-1	34.7	200	200	160	1500	3400	9.38	17	25	3.17	383	25	3.17	383	6	0.12	295	2
G9-2	34.7	200	200	160	1500	3400	9.38	17	25	3.17	383	25	3.17	383	6	0.12	295	2
G10-1	23.5	250	200	210	1500	3400	7.14	13.6	19	1.36	404	19	1.36	404	6	0.09	295	5.11
G10-2	23.5	250	200	210	1500	3400	7.14	13.6	19	1.36	404	19	1.36	404	6	0.09	295	5.11
G10-3	23.5	250	200	210	1500	3400	7.14	13.6	19	1.36	404	19	1.36	404	6	0.09	295	5.11
G10-4	23.5	250	200	210	1500	3400	7.14	13.6	19	1.36	404	19	1.36	404	6	0.09	295	5.11
G11-1	23.6	300	200	250	1350	3100	5.4	10.33	22	1.55	401	22	1.55	401	6	0.08	295	1.66
G11-2	23.6	300	200	250	1350	3100	5.4	10.33	22	1.55	401	22	1.55	401	6	0.08	295	1.66
G11-3	23.6	300	200	250	1350	3100	5.4	10.33	22	1.55	401	22	1.55	401	6	0.08	295	1.66
G11-4	23.6	300	200	250	1350	3100	5.4	10.33	22	1.55	401	22	1.55	401	6	0.08	295	1.66
G11-5	23.6	300	200	250	1350	3100	5.4	10.33	22	1.55	401	22	1.55	401	6	0.08	295	1.66
G11-6	23.6	300	200	250	1350	3100	5.4	10.33	22	1.55	401	22	1.55	401	6	0.08	295	1.66

Beam mark	Beam characteristics						a/d	L/h	Longitudinal reinforcement						Shear reinforcement			R_s/R_b
	f'_c (MPa)	h (mm)	b (mm)	d (mm)	a (mm)	L (mm)			Top			Bottom			Dia. (mm)	Ratio (%)	f_{Ty} (MPa)	
									Dia. (mm)	Ratio (%)	f_{Ly} (MPa)	Dia. (mm)	Ratio (%)	f_{Ly} (MPa)				
Kishi and Mikami [K9]																		
G12-1	23.6	300	200	250	1350	3100	5.4	10.33	25	3.04	407	25	3.04	407	6	0.08	295	1.52
G13-1	23.6	400	200	350	1350	3100	3.86	7.75	25	1.45	406	25	1.45	406	6	0.06	295	2.11
G14-1	23.6	350	200	300	1350	3100	4.5	8.86	25	1.69	406	25	1.69	406	6	0.07	295	1.96
G15-1	23.6	400	200	350	1350	3100	3.86	7.75	25	1.11	406	25	1.11	406	6	0.06	295	3.58
G16-1	23.6	370	200	320	1350	3100	4.22	8.38	25	1.58	406	25	1.58	406	6	0.06	295	2.16
Note: N/A- Not Applicable																		

APPENDIX-C: Impact Response Characteristics

Beam mark	Mass of Impactor (kg)	Height of Impactor (m)	Impact Velocity (m/s)	Input kinetic energy (kJ)	Maximum impact force (kN)	Maximum reaction force (kN)	Maximum displacement (mm)	Failure modes
Kishi <i>et al.</i> [K4]								
A19	200	N/P	1.00	0.1	N/P	41.9	N/P	N/P
A22	200	N/P	1.00	0.1	N/P	N/P	N/P	N/P
B10	200	N/P	1.00	0.1	21.9	29.2	0.45	N/P
B13	200	N/P	1.00	0.1	N/P	N/P	N/P	N/P
B19	200	N/P	1.00	0.1	N/P	46.9	N/P	N/P
C10	200	N/P	1.00	0.1	N/P	N/P	N/P	N/P
C13	200	N/P	1.00	0.1	N/P	21.6	N/P	N/P
C19	200	N/P	1.00	0.1	N/P	N/P	N/P	N/P
Kishi <i>et al.</i> [K6]								
A24	300	0.05	1.00	0.15	90.9	93.5	1.1	N/P
	300	0.46	3.00	1.35	144.4	113.4	10.6	N/P
	300	0.82	4.00	2.4	183.5	162.9	15.1	N/P
	300	1.27	5.00	3.75	194.3	161.2	29.6	N/P
	300	1.84	6.00	5.4	197.3	161.4	53.6	N/P
A36	300	0.05	1.00	0.15	62.2	60.3	1.6	Flexure
	300	0.46	3.00	1.35	124.2	121.5	11.3	Diagonal tension (One side)
	300	0.82	4.00	2.4	117.9	125.6	28.8	Diagonal tension (Both side)
	300	1.27	5.00	3.75	142.2	105.1	70	Disintegration
A48	300	0.05	1.00	0.15	40.3	40.3	4	N/P
	300	0.46	3.00	1.35	90.1	90.1	13.7	N/P
	300	0.82	4.00	2.4	125	107.9	18.4	N/P
	300	1.27	5.00	3.75	141.6	104.6	145.8	N/P

Beam mark	Mass of Impactor (kg)	Height of Impactor (m)	Impact Velocity (m/s)	Input kinetic energy (kJ)	Maximum impact force (kN)	Maximum reaction force (kN)	Maximum displacement (mm)	Failure modes
Kishi <i>et al.</i> [K6]								
B24	300	0.05	1.00	0.15	69.3	70.1	2.3	N/P
	300	0.2	2.00	0.6	101.3	95	6.6	N/P
	300	0.46	3.00	1.35	124.3	98.8	15.1	N/P
	300	0.82	4.00	2.4	125.7	94.5	17	N/P
	300	1.27	5.00	3.75	136.4	110.3	53.3	N/P
B36	300	0.05	1.00	0.15	44.9	43.1	2.9	Flexure
	300	0.46	3.00	1.35	87	71.5	17.1	Flexure
	300	0.82	4.00	2.4	104.7	72.2	27.1	Flexure-Compression
	300	1.27	5.00	3.75	128.8	80.1	109.3	Diagonal tension (One side)
B48	300	0.05	1.00	0.15	39.4	31.6	4.5	N/P
	300	0.46	3.00	1.35	94.5	56.7	21.8	N/P
	300	0.82	4.00	2.4	111.7	86	36.5	N/P
	300	1.27	5.00	3.75	133	110.3	52.8	N/P
	300	1.84	6.00	5.4	148.4	114.5	135.1	N/P
Bhatti and Kishi [B6]								
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	384	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

Beam mark	Mass of Impactor (kg)	Height of Impactor (m)	Impact Velocity (m/s)	Input kinetic energy (kJ)	Maximum impact force (kN)	Maximum reaction force (kN)	Maximum displacement (mm)	Failure modes
Bhatti and Kishi [B6]								
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 appear
B46	384	1.08	4.58	4.2	900	816	11.25	More diagonal and flexural cracks appear
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
B93	384	4.41	9.30	17.3	1519	1160	28	More wide diagonal cracks on both sides and more number of cracks and more spalling of concrete at the top surface appears
Fujikake <i>et al.</i> [F4]								
S1616	400	0.15	1.73	0.6	119	N/P	5.9	Flexure
	400	0.3	2.45	1.2	164	N/P	11.01	Flexure
	400	0.6	3.46	2.4	245	N/P	20.11	Flexure
	400	1.2	4.85	4.7	314	N/P	36.17	Flexure
S1322	400	0.3	2.45	1.2	177	N/P	7.79	Flexure
	400	0.6	3.46	2.4	262.6	N/P	11.44	Flexure

Beam mark	Mass of Impactor (kg)	Height of Impactor (m)	Impact Velocity (m/s)	Input kinetic energy (kJ)	Maximum impact force (kN)	Maximum reaction force (kN)	Maximum displacement (mm)	Failure modes
Fujikake <i>et al.</i> [F4]								
S1322	400	1.2	4.85	4.7	303.3	N/P	23.45	Local failure with heavily crushed concrete at impact point
	400	2.4	6.86	9.4	314.8	N/P	27.03	Local failure with heavily crushed concrete at impact point
S2222	400	0.3	2.45	1.2	204.2	N/P	6.99	Flexure
	400	0.6	3.46	2.4	262.7	N/P	11.04	Flexure
	400	1.2	4.85	4.7	309.9	N/P	21.42	Local failure with heavily crushed concrete at impact point
	400	2.4	6.86	9.4	353.2	N/P	31.82	Local failure with heavily crushed concrete at impact point
Chen and May [C3]								
A1	98.7	2.72	7.30	2.63	N/P	N/P	N/P	Flexure failure and no spalling of impact zone
	98.7	2.72	7.30	2.63	223	N/P	N/P	Same as above
	98.7	2.72	7.30	2.63	234	N/P	N/P	Same as above
	98.7	2.72	7.30	2.63	233	N/P	N/P	Same as above
	98.7	2.72	7.30	2.63	N/P	N/P	N/P	Same as above
	98.7	2.72	7.30	2.63	229	N/P	N/P	Same as above
A1*	98.7	2.72	7.30	2.63	128	N/P	N/P	Same as above
A2	98.7	2.72	7.30	2.63	214	N/P	N/P	Same as above
	98.7	2.72	7.30	2.63	230	N/P	N/P	Same as above
A3	98.7	2.72	7.30	2.63	194	N/P	N/P	Flexure failure and no spalling of impact zone

Beam mark	Mass of Impactor (kg)	Height of Impactor (m)	Impact Velocity (m/s)	Input kinetic energy (kJ)	Maximum impact force (kN)	Maximum reaction force (kN)	Maximum displacement (mm)	Failure modes
Chen and May [C3]								
B1	98.7	2.72	7.30	2.63	N/P	N/P	N/P	High yielding or rupture of tension steel , spalling of impact zone
	98.7	2.72	7.30	2.63	161	N/P	N/P	Same as above
B1*	98.7	2.72	7.30	2.63	183	N/P	N/P	Same as above
B2	98.7	2.72	7.30	2.63	169	N/P	N/P	Same as above
	98.7	2.72	7.30	2.63	171	N/P	N/P	High yielding or rupture of tension steel, spalling of impact zone spalling of impact zone also scabbing and spalling
B3	98.7	2.72	7.30	2.63	654	N/P	N/P	Scabbing and spalling
B3*	98.7	2.72	7.30	2.63	215	N/P	N/P	Scabbing and spalling
B4	98.7	2.72	7.30	2.63	241	N/P	N/P	Scabbing and spalling
Saatci and Vecchio [S1]								
SS0a	211	3.26	8.00	6.75	N/P	305	9.3	Several diagonal cracks on both sides but one side slightly wider 0.5 mm in width
SS0b	600	3.26	8.00	19.18	N/P	N/P	N/P	Extensive damage: massive concrete spalling at top and bottom, longitudinal reinforcements were exposed and bent, bond failure at support
SS1a	211	3.26	8.00	6.75	N/P	356	12.1	Diagonal cracks on both sides and shear plug with 0.25 mm wide under impact point
SS1b	600	3.26	8.00	19.18	N/P	625	39.5	Diagonal cracks up to 5 mm wide and 0.9 mm wide vertical cracks at midspan
SS2a	211	3.26	8.00	6.75	N/P	327	10	Several minor cracks and 0.2 mm wide diagonal crack

Beam mark	Mass of Impactor (kg)	Height of Impactor (m)	Impact Velocity (m/s)	Input kinetic energy (kJ)	Maximum impact force (kN)	Maximum reaction force (kN)	Maximum displacement (mm)	Failure modes
Saatci and Vecchio [S1]								
SS2b	600	3.26	8.00	19.18	N/P	592	37.9	Several diagonal cracks, one 1.8 mm wide and vertical cracks (1.3 mm) in midspan
SS3a	211	3.26	8.00	6.75	N/P	431	10.7	One sided 0.15 mm wide diagonal crack with approx. 45 degree inclination
SS3b	600	3.26	8.00	19.18	N/P	682	35.3	Several diagonal cracks (0.4 mm) and vertical cracks (1 mm) in midspan
Tachibana <i>et al.</i> [T1]								
A1	300	N/P	5.00	3.75	434	N/P	24.1	N/P
A2-1	150	N/P	3.46	0.9	320.5	N/P	13.6	N/P
A2-2	300	N/P	2.45	0.9	293.4	N/P	25.4	N/P
A2-3	450	N/P	2.00	0.9	245.3	N/P	37	N/P
A2-4	150	N/P	4.90	1.8	453.4	N/P	16.3	N/P
A2-8	300	N/P	4.24	2.7	513.3	N/P	33.3	N/P
A2-9	450	N/P	3.46	2.7	444.6	N/P	48.4	N/P
A2-10	300	N/P	1.00	0.15	65.4	N/P	4.5	N/P
A2-11	300	N/P	2.00	0.6	253.2	N/P	12.6	N/P
A2-12	300	N/P	3.00	1.35	426.2	N/P	26.9	N/P
A2-13	300	N/P	4.00	2.4	489.3	N/P	41.4	N/P
A2-14	300	N/P	5.00	3.75	466.2	N/P	58.3	N/P
A4	300	N/P	5.00	3.75	451.5	N/P	114.9	N/P
B	300	N/P	5.00	3.75	667.1	N/P	77	N/P
C	300	N/P	5.00	3.75	650.3	N/P	42.4	N/P
D	300	N/P	5.00	3.75	638.7	N/P	94	N/P

Beam mark	Mass of Impactor (kg)	Height of Impactor (m)	Impact Velocity (m/s)	Input kinetic energy (kJ)	Maximum impact force (kN)	Maximum reaction force (kN)	Maximum displacement (mm)	Failure modes
Tachibana <i>et al.</i> [T1]								
E	300	N/P	5.00	3.75	742.2	N/P	29.1	N/P
F	300	N/P	5.00	3.75	663.5	N/P	43.9	N/P
Kishi and Mikami [K9]								
G1-1	300	2.5	7.00	7.35	1495.6	278.5	64.3	N/P
G1-1S	300	2.5	7.00	7.35	1600.6	287.6	58	N/P
G2-1	300	0.82	4.01	2.41	510.2	95.3	28.3	N/P
G2-2	300	1.28	5.01	3.77	779.3	92.9	44	N/P
G2-3	300	1.84	6.01	5.41	853.5	149.6	57	N/P
G2L-1	400	0.82	4.01	3.22	446.6	99	44.2	Flexural Cracks from lower and upper fibre, diagonal shear cracks around midspan area
G3-1	300	0.82	4.01	2.41	1208.5	80.6	36.7	N/P
G3-2	300	1.28	5.01	3.77	1469.5	126.4	52	N/P
G3-3	300	1.84	6.01	5.41	1038.6	169	70.6	N/P
G4-1	300	0.82	4.01	2.41	800.3	122.5	39.7	N/P
G4-2	300	1.28	5.01	3.77	985.8	153.6	56.1	N/P
G5-1	400	1.84	6.01	7.22	1313.2	317.2	63.5	Flexural Cracks from lower and upper fibre, diagonal shear cracks around midspan area
G5-2	400	2.5	7.00	9.81	1557.1	363.3	83.4	Flexural Cracks from lower and upper fibre, diagonal shear cracks around midspan area
G6-1	300	1.28	5.01	3.77	1335.9	268.4	26.4	N/P
G7-1	300	1.28	5.01	3.77	1242.8	181.1	45.8	N/P
G7-2	300	1.84	6.01	5.41	1588.3	192.8	60.9	N/P
G8-1	300	1.84	6.01	5.41	1192.1	277.5	36.5	N/P

Beam mark	Mass of Impactor (kg)	Height of Impactor (m)	Impact Velocity (m/s)	Input kinetic energy (kJ)	Maximum impact force (kN)	Maximum reaction force (kN)	Maximum displacement (mm)	Failure modes
Kishi and Mikami [K9]								
G9-1	300	1.28	5.01	3.77	931.2	208.5	43.2	N/P
G9-2	300	1.84	6.01	5.41	1102.7	248.2	57.9	N/P
G10-1	300	0.82	4.01	2.41	750.7	173.3	33.7	Flexural Cracks from lower and upper fibre, diagonal shear cracks around midspan area
G10-2	300	1.28	5.01	3.77	922.3	183.6	49.5	Flexural Cracks from lower and upper fibre, diagonal shear cracks around midspan area
G10-3	300	1.84	6.01	5.41	1016.9	188.3	67.8	Flexural Cracks from lower and upper fibre, diagonal shear cracks around midspan area
G10-4	300	2.5	7.00	7.35	1042.3	172.7	83.9	Flexural Cracks from lower and upper fibre, diagonal shear cracks around midspan area
G11-1	500	0.5	3.13	2.45	702.8	230.8	20.5	N/P
G11-2	500	0.9	4.20	4.41	971	380	33.2	N/P
G11-3	500	1.3	5.05	6.37	1461.5	372.4	43.1	N/P
G11-4	500	1.7	5.78	8.34	1877.8	379	55.5	N/P
G11-5	500	2.1	6.42	10.3	1764.8	379.3	67.2	N/P
G11-6	500	2.5	7.00	12.26	1906.6	408.1	83.4	N/P
G12-1	500	3	7.67	14.71	1675	397.5	85.4	N/P
G13-1	500	3	7.67	14.71	2150	598.7	60.6	N/P
G14-1	500	3	7.67	14.71	2258.1	539.6	63.7	N/P
G15-1	500	3	7.67	14.71	2062.9	630.4	40.5	N/P
G16-1	500	3	7.67	14.71	2022.6	567.7	52.9	N/P
Note: N/P- Not Provided								