

**PARAMETERS AFFECTING THE BLAST PERFORMANCE OF
HIGH STRENGTH FIBRE REINFORCED CONCRETE BEAMS**

by

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Abstract

A limited number of studies have been conducted in the literature in order to investigate the behaviour of high-strength fibre-reinforced concrete (HSFRC) structural components subjected to blast loads. This study summarizes the results of a research program investigating the potential of using steel fibres to improve the blast performance of high-strength reinforced concrete beams. As part of the experimental investigation twenty beams were tested, including nine beams tested under static four-point bending, and eleven beams tested under dynamic blast loads using a shock-tube. Parameters considered in the study include the effect of concrete strength, steel fibres, fibre content, fibre type, longitudinal reinforcement ratio, and presence of shear reinforcement. All beams in the study have identical dimensions, with a cross-section of 125 x 250 mm and length of 2440 mm. To manufacture the specimens, two beams were cast with normal-strength self-consolidate concrete (SCC), with a specified strength of 50 MPa, while the remaining beams were cast with either plain or fibre-reinforced high-strength concrete having a compressive strength which varied between 95-110 MPa. The steel fibre content in the HSFRC beams varied between 0.5 and 1.0%, by volume of concrete. To investigate the effect of reinforcement ratio (ρ), the beams were reinforced with 2-#4 (American size) bars, 2-15M bars or 2-20M bars ($\rho = 1.02\%$, 1.59% , and 2.41% , respectively). The majority of the plain concrete beams had transverse reinforcement which consisted of 6 mm stirrups arranged at a spacing of 100 mm in the shear spans, while most of the HSFRC beams were built without stirrups. The results indicate that all the parameters in this study (reinforcement ratio, presence of stirrups, concrete strength, steel fibres, fibre content and fibre type) affected the static and blast response of the beams, however, the results demonstrate that steel fibres have a more remarkable effect when compared to the other parameters. The provision of fibres is found to improve the blast performance of the HSC beams by increasing shear capacity, reducing maximum and residual mid-span displacements, reducing blast fragments and increasing damage tolerance.

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NOTATIONS

Symbol	Definition
A	Area impacted by the blast pressure
d_b	Steel reinforcement bar diameter
d_f	Fibre diameter
E_c	Modulus of elasticity, concrete
E_s	Modulus of elasticity, steel
f_{ctf}	Fibre reinforced concrete tensile stress
f_{cu}	Unconfined concrete stress
f_{cuf}	Unconfined fibre reinforced concrete stress
f'_c	Compressive concrete strength
f'_{cu}	Unconfined compressive concrete strength
f'_{cuf}	Unconfined compressive fibre reinforced concrete strength
f'_{dc}	Design compressive concrete strength
f_{du}	Design steel ultimate stress
f_{dy}	Design steel yield stress
$f_{fct,eq}$	Equivalent flexural strength
f_s	Steel stress
f_{sh}	Steel strain hardening stress
f_u	Steel ultimate stress
f_y	Steel yield stress
F_m	Mean load
I_r	Reflected impulse
K_{LM}	Load mass transformation factor
L_d	Driver length
l_f	Fibre length
m	Total mass of the system
P_r	Reflected pressure
P_d	Driver pressure
P_y	Load at yield of reinforcing bar
P_{max}	Maximum applied load
R	Resistance of the member
RI_v	Reinforcing index
t_p	Positive phase duration
u	Deflection at mid-height
$\ddot{u}$	Acceleration at mid-height
v_f	Fibre volume ratio
ε_{20cu}	Unconfined concrete strain at 20% of peak stress

ε_{50cu}	Unconfined concrete strain at 50% of peak stress
ε_{cu}	Unconfined concrete strain
ε_{cuf}	Unconfined fibre reinforced concrete strain
ε'_{cu}	Unconfined concrete strain at peak stress
ε'_{cuf}	Unconfined fibre reinforced concrete strain at peak stress
ε_s	Steel strain
ε_{sh}	Steel strain hardening strain
ε_u	Steel ultimate strain
ε_y	Steel yield strain
$\dot{\varepsilon}$	Strain rate
$\dot{\varepsilon}_s$	Static strain rate
δ_{anls}	Analytical displacement
δ_{exp}	Experimental displacement
δ_{max}	Maximum displacement
δ_{res}	Residual displacement
θ_{max}	Maximum support rotation
τ_{bond}	Matrix bond strength
Δ_y	Maximum displacement
Δ_{max}	Displacement at yield of reinforcing bar

ACRONYMS

Acronym	Definition
ACI	American Concrete Institute
ASTM	American Society of Testing and Materials
ASCE	American Society of Civil Engineering
ANFO	Ammonium nitrate/fuel oil
BLS	Blast loading simulator
CSA S850	Canadian Standard for the design and assessment of buildings subjected to blast loads.
CEB	Comité euro-international du béton (Euro-international Concrete Committee)
C4	Plastic explosive known as composition C
DCT	Displacement Cable Transducer
DIF	Dynamic increase factor
DLF	Dynamic Load Factor
HL	Hairline (crack width)
HSFRC	High Strength Fibre Reinforced Concrete
HSC	High strength concrete
HSS	Hollow steel section
LTD	Load transfer device
LVDT	Linear variable displacement transducer
NSC	Normal strength concrete
SCC	Self-consolidating concrete
SDOF	Single-degree-of-freedom
SFRC	Steel fibre reinforced concrete
SHPB	Split Hopkinson Pressure Bar
TNT	Trinitrotoluene
UFC	Unified Facilities Criteria

Chapter 1: Introduction

1.1. General

The mitigation of blast loads on reinforced concrete structures has become an increasingly important design consideration for buildings of strategic importance, government offices, embassies and military installations. Incidents such as the World Trade Center bombing (1993) and the Khobar Towers bombing (1996) demonstrate the devastation that can be caused by malicious explosions and provides motivation for engineers to design and construct safe and blast-resistant structures. Accidental explosions such as the Toronto Sunrise propane explosion (2008) and the Lac-Mégantic rail disaster (2013) also raise questions relating to the blast safety of vulnerable structures in populated regions. The possibility of similar events occurring in the future is an ongoing threat that needs to be addressed through the development of effective methods that can mitigate the effects of blast loads on structures.

Although high-strength concrete (HSC) is now commonly used in many structural applications, the brittleness of this material when compared to normal-strength concrete may make HSC structural members more vulnerable to brittle failure under blast loading. The provision of steel fibres has been shown to mitigate the brittleness of HSC, while also increasing its ductility and toughness. The combined use of HSC and steel fibres may provide an ideal solution to improve the blast resistance of concrete structures.

Existing research on the behaviour of HSC and high-strength fibre-reinforced concrete (HSFRC) at high-strain rate dynamic loading is limited, and there is a need for research data, particularly in the case of the behaviour of large-scale structural components under extreme blast loading.

1.2. Research Objectives

The primary objective of this thesis is to study the potential of using steel fibres to improve the blast performance of high-strength concrete structural components. This objective is achieved by testing a large set of HSC and HSFRC beams under quasi-static and simulated blast loading. The study also investigates the influence of various design variables (steel reinforcement and fibre parameters) on the quasi-static and blast performance of the HSC and HSFRC beams. A secondary objective of the research program is to investigate the suitability of using dynamic inelastic single-degree-of-freedom (SDOF) analysis to predict the blast response of the tested HSC and HSFRC beams.

1.3. Scope

As part of the experimental investigation, twenty beams are tested: nine beam specimens are tested under quasi-static four-point bending, and eleven beam specimens are tested under dynamic blast loads using the University of Ottawa Shock-Tube Facility. The design parameters investigated in the study include:

- The effect of concrete strength, with two concrete mixes, comprised of normal-strength self-consolidating concrete (SCC) and high-strength concrete (HSC);
- The effect of longitudinal reinforcement ratio, with beams constructed with 2-#4, 2-15M, and 2-20M regular strength steel bars;
- The effect of steel fibres, with companion high-strength concrete beams built with and without steel fibres;
- The effect of steel fibre content, with two volumetric ratios of steel fibres;
- The effect of steel fibre type, with two different types of steel fibres having varied tensile strength and aspect ratios;
- The effect of shear reinforcement, with beams built with and without stirrups.

The test program includes six beams reinforced with #4 steel bars, four beams reinforced with 15M steel bars, and ten beams reinforced with 20M steel bars. Two of these beam specimens are constructed with normal-strength concrete; the remaining beam specimens are constructed with high-strength concrete, with or without steel fibres. Dynamic testing was conducted by subjecting the beams to gradually increasing blast pressures until failure. Companion beams were also tested under four-point static loading in order to examine the effect of loading-rate on beam response. Performance criteria were used to investigate the performance of the beams under static and dynamic loads. The performance criteria for the dynamic tests included: failure mode, maximum and residual mid-span displacements, crack control, damage tolerance, and mitigation of secondary fragmentation. For the static tests, the performance criteria included: load resistance, stiffness, ductility, maximum displacement, and failure mode. These criteria are used to determine the effects of the experimental design parameters on beam performance.

Dynamic inelastic single-degree-of-freedom (SDOF) analysis, with the development of analytical resistance curves for HSC and HSFRC beams, is also conducted in order to predict the dynamic response of the beam specimens tested in the research program.

1.4. Thesis Breakdown

This thesis contains nine chapters as shown in Figure 1-1, and is divided as follows:

Chapter 1 - Introduction

- Introduces the significance of the thesis, describes the research objectives, and defines the scope.

Chapter 2 - Literature Review

- Summarizes previous research on the static and blast performance of reinforced concrete beams and structural components constructed with high-strength concrete and fibre-reinforced concrete materials.

Chapter 3 - Experimental Program

- Describes the experimental program, including the test matrix, specimen design details, material parameters, test setup and testing protocols.

Chapter 4- Experimental Results of the Static Test

- Summarizes the detailed results from the quasi-static experimental testing program with the results presented in terms of load-deflection response.

Chapter 5- Discussion of Quasi-Static Experimental Results

- Summarizes various observations in terms of the effects of the concrete strength, steel reinforcement ratio, steel fibres, fibre type/content, and transverse reinforcement on the quasi-static behaviour of the beam specimens. The performance is compared in terms of the effect of the variables on load resistance, initial stiffness, toughness, failure mode, displacement and ductility.

Chapter 6 - Experimental Results of the Dynamic Test

- Presents the detailed results from the blast experimental program. For each beam, the results are presented in terms of pressure and impulse time histories, displacement time-histories, damage, and failure mode.

Chapter 7 - Discussion of Dynamic Experimental Results

- Discusses the effect of test parameters considered in this research including the effects of the concrete strength, steel reinforcement ratio, steel fibres, steel fibre type, steel fibre content, and transverse reinforcement on the blast performance of the beam specimens. The performance is compared in terms of the effect of the variables on maximum and residual displacements, blast resistance, failure mode, damage tolerance and secondary fragmentation.

Chapter 8 – Analysis and Models

- Explains the process of predicting the blast response of each beam specimen using a single-degree-of-freedom approach
- Compares the effect of various modeling parameters on the analytical predictions, including the effect of material stress-strain models and dynamic increase factor combinations for concrete and steel;
- Summarizes the resulting comparisons between the experimental and analytical dynamic response in terms of maximum mid-span displacement.

Chapter 9 – Conclusion

- Presents final remarks on the highlights of the research program and proposes recommendations for further research

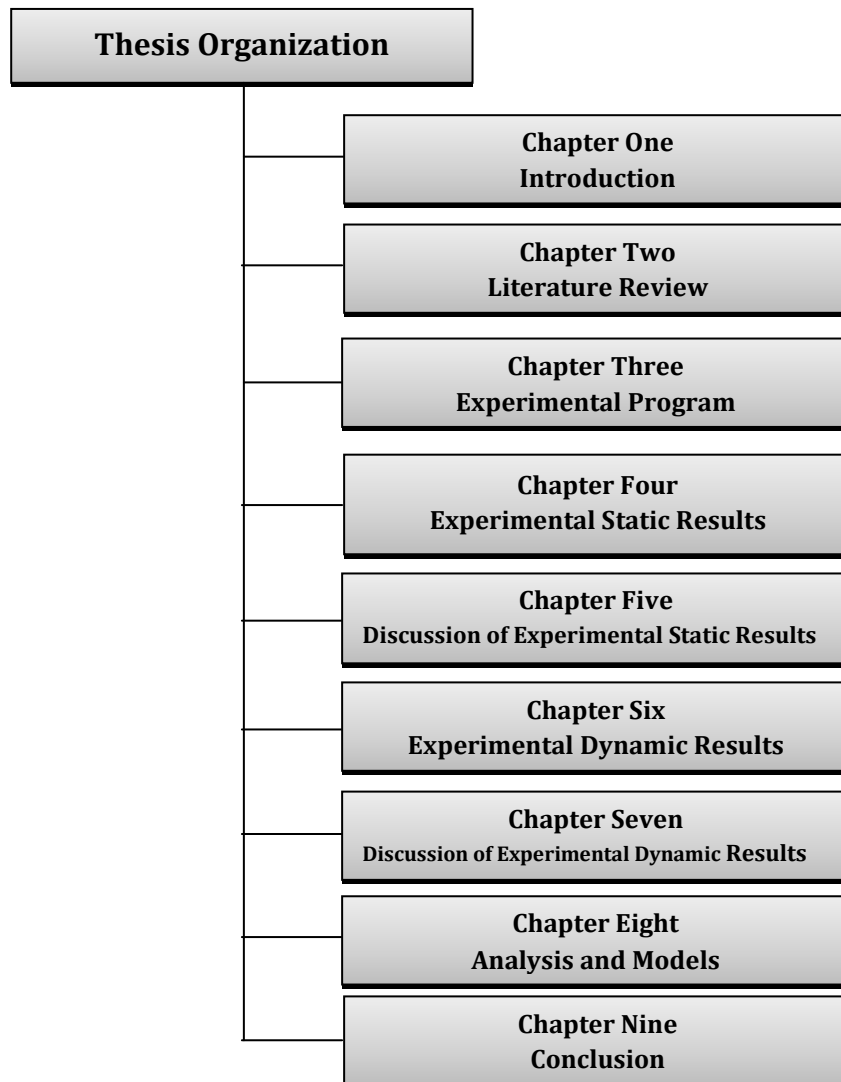


Figure 1-1 Thesis organization

Chapter 2: Literature Review

This chapter provides a literature review related to the research topic which focuses on the blast response of beams with high-strength concrete and steel fibres. After briefly reviewing blast loading on structures and examining the basic properties of high-strength concrete (HSC) and high-strength steel fibre-reinforced concrete (HSFRC), previous research studies related to the flexural behaviour of HSC and HSFRC beams are reviewed. Thereafter, previous research on the blast behaviour of reinforced concrete and fibre-reinforced concrete beams and flexural members is presented, as well as studies on the effects of high strain rates which are characteristic of impact and blast load responses.

2.1 A Brief Review of Blast Loading and Structural Response

2.1.1 Introduction

According to NGO et al. (2007), the definition of an explosion is a large-scale, rapid, and sudden release of energy. Explosions can be categorized into three basic types: physical, nuclear, or chemical explosions. An example of a physical explosion would be the catastrophic failure of a cylinder of compressed gas or the mixing of two liquids at different temperatures, which causes the sudden release of energy. A nuclear explosion involves the redistribution of protons and neutrons within interacting nuclei because of a high-speed nuclear reaction, which causes the rapid release of energy. Chemical explosions are caused by the rapid oxidation of fuel elements (carbon and hydrogen atoms) and are generally considered the most common type of explosions which can affect structures. A common example includes the explosion of trinitrotoluene (TNT) (Burrell, 2012).

2.1.2 Categories of Blast Loads on Structures

The distance between the burst and the structural element and the location of the detonation with respect to the structure are important parameters, which affect the blast loading on structures. The U.S. Department of Defense's Unified Facilities Criteria (UFC 3-340-02, 2008) categorizes explosions into two main types based on the location of the explosive in relation to the structure as shown in Figure 2-1. First, explosions that occur inside structures such as homes or apartments due to the build-up of gas pressure are called confined explosions. Confined explosions take one of three forms: a fully vented explosion, a partially confined explosion, or a fully confined explosion. Unconfined explosions are those that occur outside structures in an open area. An unconfined explosion can be further categorized based on the location of the detonation in relation to the surface of the earth. A free air burst explosion is an explosion that occurs in the air, far away from the earth's surface. An air burst explosion is one that occurs in the air close to the surface of the

earth. The third category of unconfined explosion is a surface burst explosion; it occurs on the surface of the earth and takes the shape of a hemispherical blast wave due to the initial reflection with the earth's surface. When the distance between the detonation and the structural element is small, it results in a high intensity pressure load over the structural element surface; this type of detonation is termed a "close-in explosion." When the distance is larger, the result is a lower-intensity, longer duration shockwave with uniform pressure distribution; this type of detonation is called a "far-field explosion". It is noted that, the University of Ottawa shock-tube is capable of simulating blast waves generated by far-field explosions.

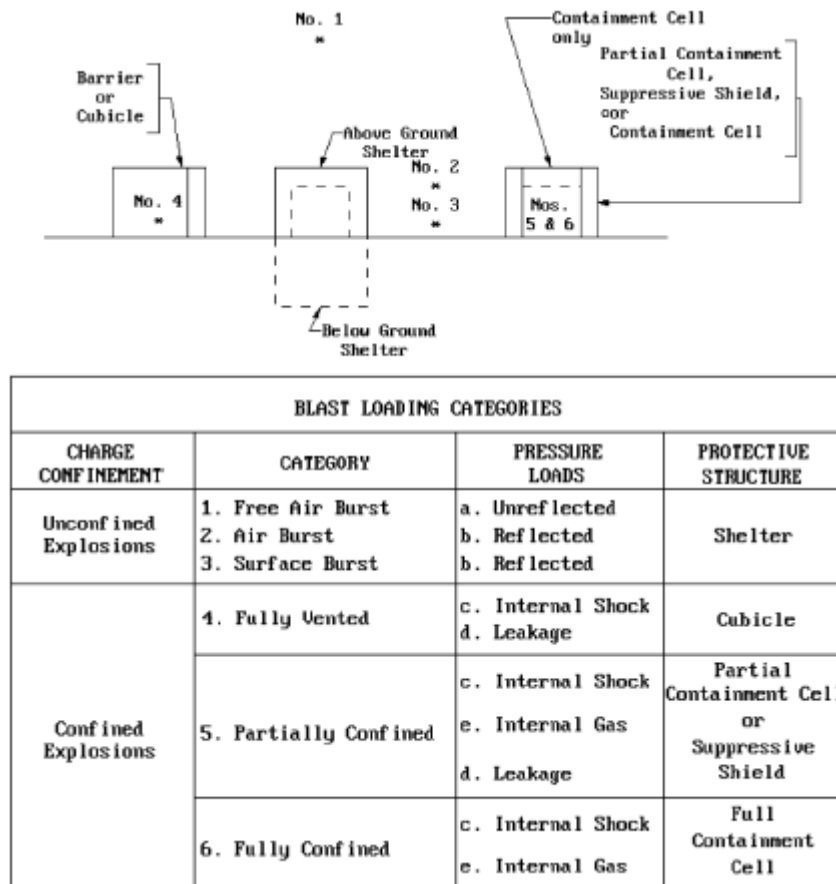


Figure 2-1 Blast loading categories (UFC 3-340-02, 2008)

2.1.3 Blast Waves Parameters

The main parameters of blast loading are defined as the blast overpressure, duration, and impulse. When a structure is subjected to a blast wave that is released from a burst at a certain distance in relation to the structure, it faces two different phases in terms of pressure-time history, as shown in Figure 2-2. The duration of the shock front wave, when the blast wave travels from the center of the detonation towards the structure, is called "travel time (t_A)". The first phase causes a sudden

increase in ambient pressure to ultimate overpressure (P_{so}) when the shock front wave hits the structure; after some time, the pressure decays and returns to the ambient pressure, resulting in the end of the “positive phase duration (t_d)”. The second phase is the duration of negative pressure which creates a partial vacuum where the overpressure returns back to the area of the detonation and is associated with the “negative phase duration (t_d)”. This phase has a longer duration than the first phase, but is lower in intensity than the positive phase.

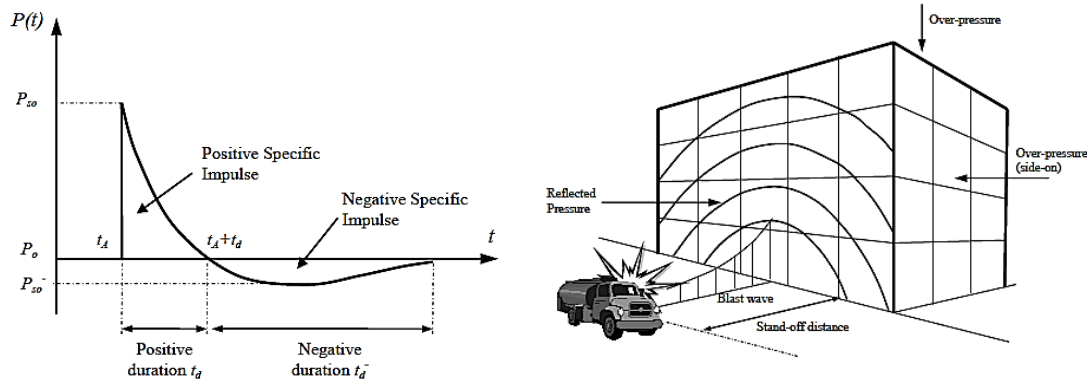


Figure 2-2 Idealized pressure-time history of a shock wave and blast loads on a building
[Adapted from NGO et al (2007)]

2.1.4 Blast Testing Methods

There are two main methods used to subject structural members to blast loading: live high-explosive testing, and the use of blast simulators such as shock-tubes to simulate blast loading. High explosive testing can be achieved by using TNT, C4, or ANFO to generate blast waves. While providing more realistic loading, this method has several drawbacks which include safety concerns and costs associated with testing and instrumentation. Blast simulators provide some advantages when compared to live explosive testing, including safe and controlled test environments and the ability to conduct parametric studies at relatively low cost. The University of Ottawa shock-tube can simulate blast waves produced by free-air detonations of high explosives (Lloyd et al., 2010). The operational range of the shockwaves generated by the shock-tube is defined in regards to the peak reflected pressure, reflected impulse, and positive phase duration. The driver pressure controls the magnitude of the peak reflected pressure while the driver length controls the positive phase duration. Figure 2-3 illustrates the reflected impulse over the positive phase plotted against driver pressure for various driver lengths, as reported by Lloyd et al. (2010). It can be observed that as the driver length increases, the positive phase duration becomes longer. It can also be seen that the shock-tube can simulate a range of realistic shockwaves with varying pressure-impulse combinations. Various combinations of equivalent masses and standoff

distances for hemispherical explosions of TNT can be determined by selecting the appropriate driver length and driver pressure.

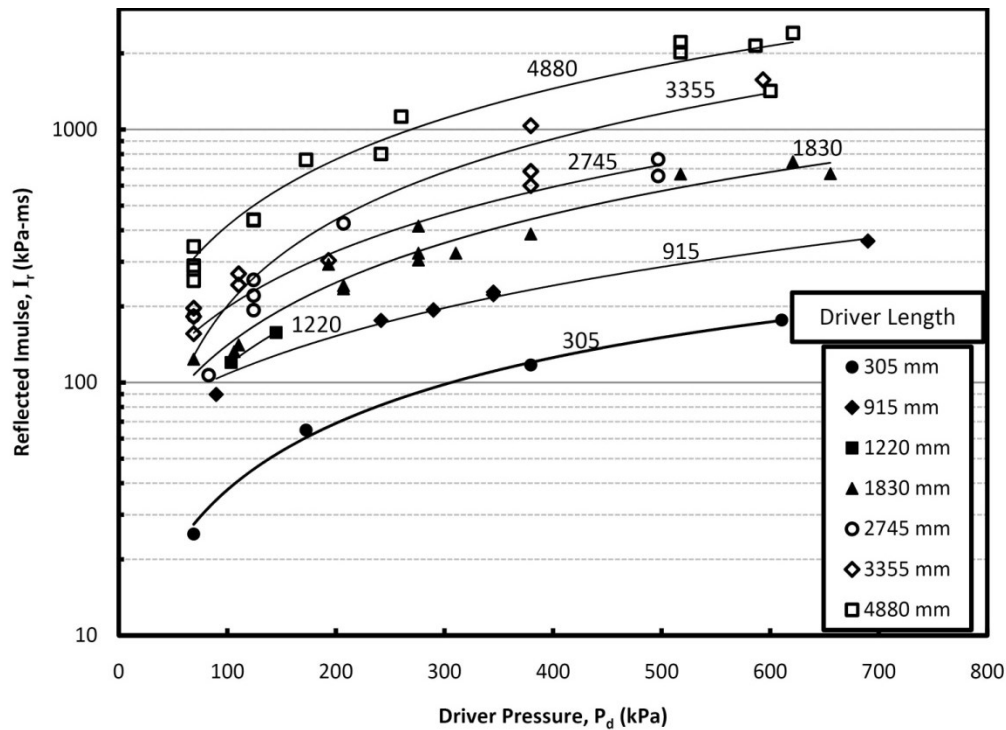


Figure 2-3 Reflected impulses over the positive phase as a function of driver pressure for various driver length

[Adapted from Lloyd et al. (2011)]

2.1.5 Dynamic Effects on Material Properties

Structures are subjected to different types of dynamic loads, which include loads generated by wind, earthquakes, impact from missiles or projectiles, machine vibration, and blasts. These loads, which occur over varying time durations, result in the application of loading at varying strain rates which invariably affect material and structural response. As can be seen in Figure 2-4, the strain rate is higher for a blast load when compared to the other types of dynamic loading. The following sections review the dynamic effects of high strain rates on the properties of concrete and steel when subjected to compressive or tensile loading, with a review of relevant models which can be used to determine the dynamic increase factor (DIF) for blast analysis of reinforced concrete structures.

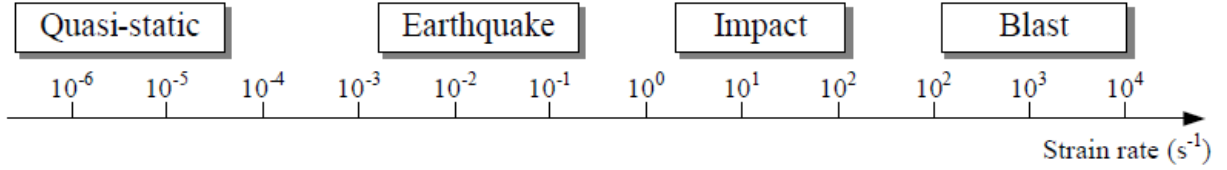


Figure 2-4 Strain rates associated with different types of loading
[Adapted from Burrell (2010)]

2.1.5.1 Introduction to Strain Rate Effects

Several researchers have found that strain rate is considerably increased when a structural member is subjected to impact or blast loads. The material properties of concrete and steel are affected by this sudden increase in strain rate. The strain rate ($\dot{\epsilon}$) of concrete and steel can be determined using equations [2-1] and [2-2]. The effect of strain rate on the dynamic properties of materials can be evaluated using varying methods which include drop weight tests, pendulum tests, firing of projectiles, and detonating explosives. The results of these dynamic tests are then compared to the results of static tests in order to obtain the dynamic increase factor (DIF), which is defined as the ratio of dynamic material strength to static material strength. Based on these tests, equations have been proposed by researchers for evaluating the DIF of concrete and steel in compression and tension, with guidelines provided in documents such as the U.S. Department of Defense's Unified Facilities Criteria (UFC 03-340-02, 2008) and the Comité euro-international du béton (CEB) Model Code (1990).

$$\dot{\epsilon} = \frac{\epsilon_0}{t_E} \quad [2-1]$$

Where:

ϵ_0 = the peak strain of concrete

t_E = the time required to yield reinforcement

$$\dot{\epsilon} = \frac{f_{dy}}{E_s t_E} \quad [2-2]$$

Where:

f_{dy} = the dynamic yield strength of steel

2.1.5.2 Dynamic Increase Factor for UFC-3-340-02 (2008)

For the design of blast-resistant reinforced concrete elements, UFC-3-340-02 (2008) provides dynamic increase factors for far and close-in design ranges. The DIF values proposed by UFC-3-340-02 (2008) do not take into consideration the strain rate and are applied on the concrete and steel stress-strain curves as shown in Figure 2-5. For this thesis, the DIF for bending at the far range will be used. These values can be found in Table 2-1.

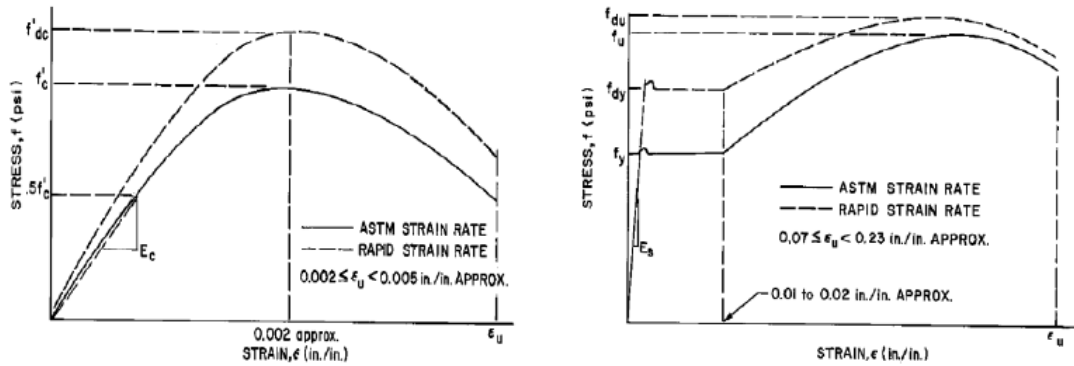


Figure 2-5 Typical stress-strain models for quasi-static and dynamic strain rates (UFC 03-340-02 2008)
[Adapted from UFC 03-340-02 (2008)]

Table 2-1 Dynamic increase factors for design of reinforced concrete elements (UFC-3-340-02, 2008)

Type of stress	Far design range			Close-in design range		
	Reinforcing bars		Concrete	Reinforcing bars		Concrete
	f_{dy}/f_y	f_{du}/f_u	f'_{dc}/f'_c	f_{dy}/f_y	f_{du}/f_u	f'_{dc}/f'_c
Bending	1.17	1.05	1.19	1.23	1.05	1.25
Diagonal tension	1.00	-	1.00	1.10	1.00	1.00
Direct shear	1.10	1.00	1.10	1.10	1.00	1.10
Bond	1.17	1.05	1.00	1.23	1.05	1.00
Compression	1.10	-	1.12	1.13	-	1.16

2.1.5.3 Dynamic Increase Factor for Concrete in Compression

The effects of strain rate on the compressive strength of concrete have been studied by numerous researchers. Figure 2-6 plots the relative increase in compressive strength as a function of strain-rate for some of the data reported in the literature. It can be seen that the ratio of dynamic-to-static strength begins to increase at strain-rates above 10^{-5} s^{-1} . It can also be noticed that the ratio shows a sharper rate of increase at strain-rates beyond $\sim 30 \text{ s}^{-1}$. For this reason, several models in the literature use a bi-linear relationship to model the dynamic increase factor (DIF) of concrete in compression. According to Saatcioglu et al. (2011), some researchers have reported lower DIF for high-strength concrete (Hughes and Watson 1978, Evans 1942, Cowel 1966) while others have found the effect to be negligible or opposite (Watstein 1953, Atchley and Furr 1967, Katsuta 1943, Abrams 1917, Jones and Richart 1936, Oh and Shin 1987). It is noted that the data from the literature shows wide scatter. As shown in Figure 2-6, the CEB model predicts lower DIF for high-strength concrete when compared to normal-strength concrete, while the model proposed by Saatcioglu et al. (2011) (marked by the red

line in Figure 2-6) proposes DIF relationships which are applicable to both normal and high-strength concrete. Equations [2-3] through [2-5] demonstrate the CEB (1990) model and Saatcioglu et al. (2011) model equations.

CEB Model Code 90 (1990)

$$DIF_c = \begin{cases} \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^{1.026\alpha} & \text{for } \dot{\epsilon} \leq 30s^{-1} \\ \gamma \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^{1/3} & \text{for } \dot{\epsilon} > 30s^{-1} \end{cases} \quad [2-3]$$

$$\log \gamma = 6.156\alpha - 2, \quad \alpha = 1/(5 + 9f'_{cu}/10), \quad \dot{\epsilon}_s = 30 \cdot 10^{-6} s^{-1} \quad [2-4]$$

Saatcioglu et al. (2011)

$$DIF = \begin{cases} 0.03\ln(\dot{\epsilon}) + 1.30 & \geq 1.0 \text{ for } \dot{\epsilon} < 30 s^{-1} \\ 0.55\ln(\dot{\epsilon}) - 0.47 & \text{for } \dot{\epsilon} \geq 30 s^{-1} \end{cases} \quad [2-5]$$

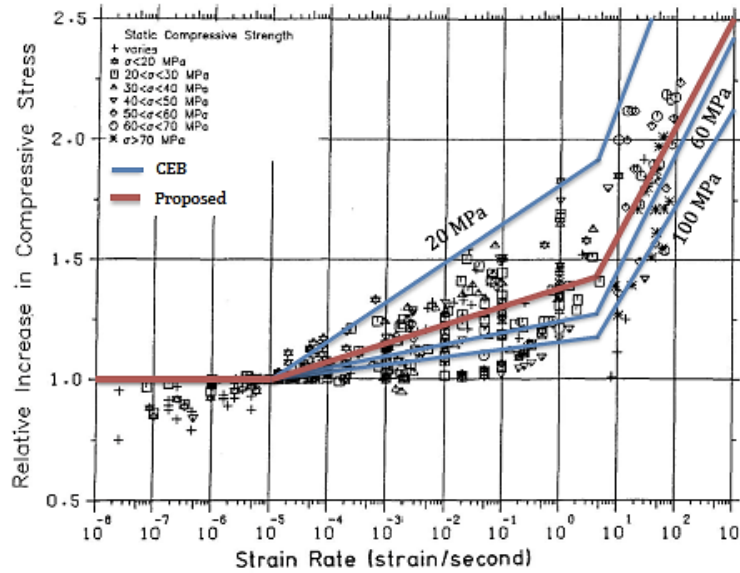


Figure 2-6 Effect of strain rate on concrete compressive
[Adapted from Saatcioglu et al. (2011)]

2.1.5.4 Dynamic Increase Factor for Concrete in Tension

Malvar and Ross (1998) examined the effects of strain rate on the response of concrete in tension and compared experimental data for the DIF of concrete in tension to the CEB model predictions for the DIF of concrete having compressive strengths of 30 MPa and 70 MPa. The comparison showed conflicts with the CEB model predictions, mostly for strain rates beyond $1 s^{-1}$. The data was put in a log-log plot, with the dynamic increase factor (DIF) plotted as a function of strain rate. The

data exhibited no increase for strain rates below 10^{-6} , with a slope change at a strain rate of 1 s^{-1} , while beyond 1 s^{-1} it differed with the CEB model as shown in Figure 2-7. Based on the results of this study, modified equations for DIF of concrete in tension were developed as expressed in equations [2-6] and [2-7]:

$$DIF_t = \begin{cases} \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^\delta & \text{for } \dot{\epsilon} \leq 1 \text{ s}^{-1} \\ \beta \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^{1/3} & \text{for } \dot{\epsilon} > 1 \text{ s}^{-1} \end{cases} \quad [2-6]$$

$$\delta = \frac{1}{\left(1 + \frac{8f'_c}{f'_{co}}\right)}, \quad \log \beta = 6\delta - 2, \quad f'_{co} = 10 \text{ MPa}, \quad \dot{\epsilon}_s = 10^{-6} \text{ s}^{-1} \quad [2-7]$$

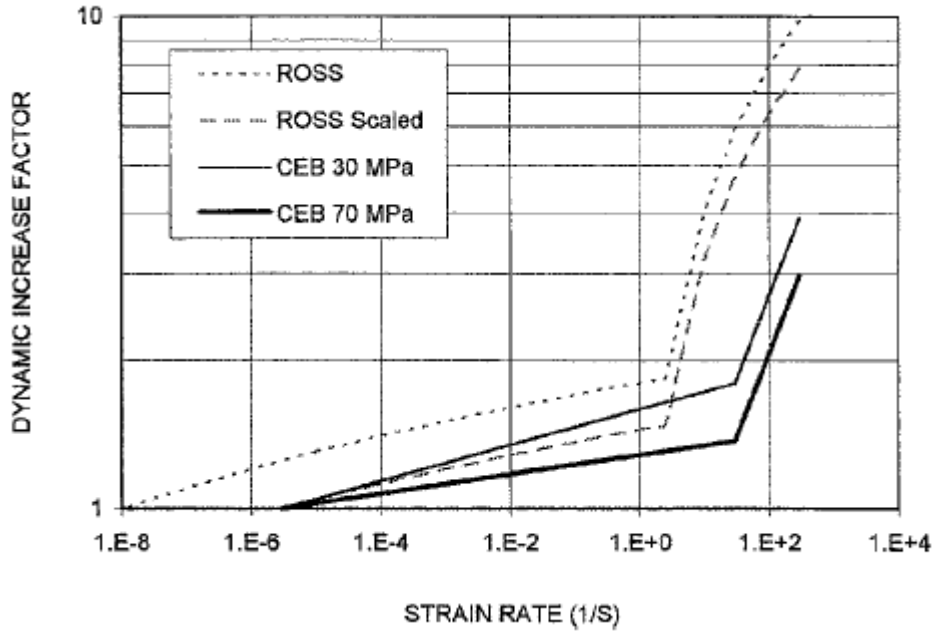


Figure 2-7 Ross scaled model versus CEB model for the DIF of concrete in tension
[Adapted from Malvar and Ross (1998)]

2.1.5.5 Dynamic Increase Factor for Steel Fibre-Reinforced Concrete (SFRC) and High-Strength Fibre-Reinforced Concrete (HSFRC)

Some researchers have found that the CEB model is not adequate for estimating the dynamic increase factor of high-strength steel fibre reinforced concrete (HSFRC). Riisgaard (2007) tested a set of high-strength fibre-reinforced concrete cylinders with compressive concrete strengths of 100 MPa and 160 MPa under dynamic loading. The experimental data was compared to the CEB model predictions for DIF. The study found the CEB model to be adequate for 100 MPa

concrete, whereas the model overestimated the dynamic strength of the 160 MPa cylinders. Based on the results, a new model called the “RCM model” was proposed to predict the DIF of high-strength concrete. In a subsequent study Zhang & Mindness (2011) tested a series of normal, high, and very high-strength fibre-reinforced concrete specimens with compressive strengths of 50 MPa, 90 MPa, and 110 MPa. The specimens were tested at moderate strain-rates using an instrumented drop weight impact machine, while Riisgaard (2007) tested his specimens using a Split Hopkinson Pressure Bar (SHPB). The experimental results showed a good match with the RCM model for strain rates ranging in between 10^{-5} sec^{-1} and 10 sec^{-1} , as shown in Figure 2-8. The following equations and Table 2-2 illustrate the use of the RCM model.

$$DIF_c = \begin{cases} (\phi \dot{\epsilon})^\alpha & \text{for } \dot{\epsilon} < \dot{\epsilon}_{BLT} \\ \beta (\dot{\epsilon})^{1/3} & \text{for } \dot{\epsilon} \geq \dot{\epsilon}_{BLT} \end{cases} \quad [2-8]$$

$$\alpha = \frac{\ln(\beta \cdot \dot{\epsilon}_{BLT}^{1/3})}{\ln(\phi \cdot \dot{\epsilon}_{BLT})}, \quad \beta = \frac{5}{9} \cdot \exp\left(\frac{-f'_{cu}}{230}\right), \quad \phi = 10^5, \quad \dot{\epsilon}_{BLT} = 25 \cdot \exp\left(\frac{f'_{cu}}{130}\right) \quad [2-9]$$

$\dot{\epsilon}_{BLT}$ = based on the compressive strength of the concrete as shown in Table 2-2

Table 2-2 Strain rate comparison of CEB model and RCM model
[Adapted from Zhang & Mindness (2011)]

Strength (MPa)	RCM strain rate, i.e., $\dot{\epsilon}_{BLT}$, at turning point (1/sec)	CEB strain rate at turning point (1/sec)
50	37	30
90	50	30
100	54	30
110	58	30
160	86	30

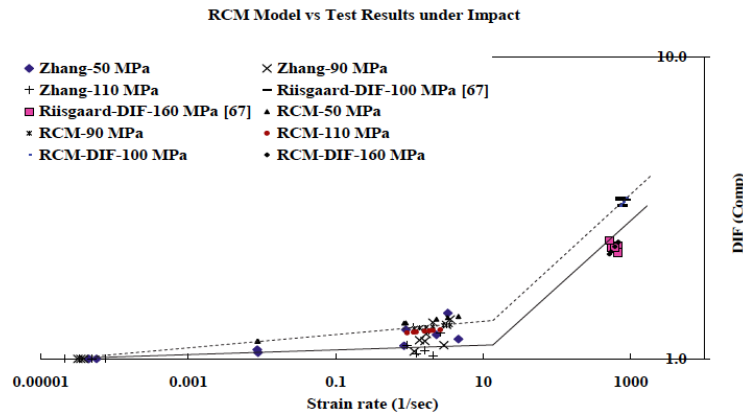


Figure 2-8 Plot of all of the experimental results of the DIF (Comp) compared with DIF derived from the RCM model

[Adapted from Zhang & Mindness (2011)]

2.1.5.6 Dynamic Increase Factor for Steel Reinforcement

High strain rates on steel properties were investigated based on a literature review by Malvar (1998). The data from the literature included steel data from tests conducted on ASTM A615, A15, A432, A431, and A706 reinforcement with yield strengths varying between 290 and 710 MPa. The study showed that the dynamic increase factor (DIF) was lower for steel bars with higher grades, and conversely, was higher for lower grade reinforcement. In addition, the dynamic yield and ultimate stresses were found to be dependent on the strain-rate and static yield stress. It was also determined that the increase in ultimate stress was relatively lower in comparison to the increase in yield stress. Based on this study, relationships for calculating the DIF of steel reinforcement at yield and ultimate stress were proposed as shown below:

$$DIF = \left(\frac{\dot{\epsilon}}{10^{-4}}\right)^{\alpha} \quad [2-10]$$

$$\alpha = \begin{cases} 0.074 - 0.040 \frac{f_y}{414} & \text{for yield stress} \\ 0.019 - 0.009 \frac{f_y}{414} & \text{for ultimate stress} \end{cases} \quad [2-11]$$

Based on a review of data in the literature, Saatcioglu et al. (2011) proposed relationships for predicting the DIF of steel at yield and ultimate stress as shown in equation [2-12]. It is noted that this model is only applicable for Grade 400 reinforcing bars only and is expressed as follows:

$$\begin{aligned} \text{at yield} &\rightarrow DIF_y = 0.034 \ln(\dot{\epsilon}) + 1.30 \geq 1.0 \\ \text{at ultimate} &\rightarrow DIF_u = 0.0101 \ln(\dot{\epsilon}) + 1.10 \geq 1.0 \end{aligned} \quad [2-12]$$

2.1.6 Blast Performance Criteria

The blast performance of reinforced concrete structural members can be assessed based on ductility ratio, $\mu = u_{max}/u_{yield}$, where u_{max} is the maximum displacement and u_{yield} is the yield displacement (ASCE, 1999). More commonly, support rotation or deflection ratio (u_{max}/L) is suggested as an alternative option to assess the blast performance of reinforced concrete components. As shown in Table 2-3, the UFC 03-340-02 (2008) relates the level of damage to a structural member with its support rotation. If the support rotation is lower than 2° , the level of damage to the structure is considered moderate, whereas support rotations higher than 2° but not more than 10° correspond with heavy to hazardous levels of damage. When the support rotation exceeds 10° the structural component is predicted to experience blowout failure. The Canadian CSA S850 (2012) Blast standard proposes similar response limits for various performance levels as shown in Table 2-4. The deformation limits are defined for four levels of protection (performance limits),

corresponding to: B1 - operational (high), B2 - immediate occupancy (medium), B3 - life safety (low), and B4 - collapse prevention (very low).

Table 2-3 Performance levels and associated deformation limits (UFC-3-340-02, 2008)

Component Level Damage	Description of Component Damage	Building Level of Protection	Limit for Reinforced Concrete Element in Flexure
Superficial Damage	Component has no visible permanent damage	High	$\mu \leq 1$
Moderate Damage	Component has some permanent deflection. It is generally repairable, if necessary, although replacement may be more economical and aesthetic	Medium	$\mu > 1$ $\theta \leq 2^\circ$
Heavy Damage	Component has not failed, but it has significant permanent deflections causing it to be irreparable	Low	$2^\circ < \theta \leq 5^\circ$
Hazardous Failure	Component has failed, and debris velocities range from insignificant to very significant	Very low	$5^\circ < \theta \leq 10^\circ$
Blowout	Component is overwhelmed by the blast load causing debris with significant velocities	Below antiterrorism standards	$\theta > 10^\circ$

Table 2-4 Performance levels and associated deformation limits (CSA S850, 2013)

Element Type	B1		B2		B3		B4	
	μ	θ	μ	θ	μ	θ	μ	θ
Single-Reinforcement Slab or Beam	1	-	-	2°	-	5°	-	10°
Double-Reinforcement Slab or Beam	1	-	-	2°	-	5°	-	10°
Double-Reinforcement Slab or Beam with shear reinforcement	1	-	-	4°	-	6°	-	10°
With tension membrane	1	-	-	6°	-	12°	-	20°

2.2 Literature Review on High-Strength Concrete (HSC) and High-Strength Fibre- Reinforced Concrete (HSFRC) Stress-Strain Relationships

2.2.1 High-Strength Concrete (HSC)

High-strength concrete (HSC) is a material with components that include Portland cement, supplementary cementitious materials (fly ash, silica fume, or slag), water, fine and coarse aggregates, and super-plasticizing admixtures (Kosmatka et al., 2002). Between the years 1960 and 1990, HSC started to be used

as premix concrete in Chicago with a compressive strength of 40 MPa. By 1989, concretes with strengths exceeding 100 MPa could be produced, allowing for the construction of taller high-rise buildings (Caldarone, 2009). The primary advantage of HSC is that it allows designers to reduce structural member size when compared to conventional concrete. However, the increase in compressive strength in HSC results in an increase in brittleness when compared to normal-strength concrete (NSC). Figure 2-9 compares the compressive stress-strain behaviour of HSC and NSC. It can be seen that HSC shows increased peak stress, peak strain and stiffness (modulus of elasticity) when compared to NSC. The comparison also shows that HSC has steeper ascending and descending branches when compared to normal strength concrete, which makes it more brittle (Wight and MacGregor, 2009). HSC releases large amounts of energy during failure and this causes the unstable descending branch after the peak load (Mier et al. 1997).

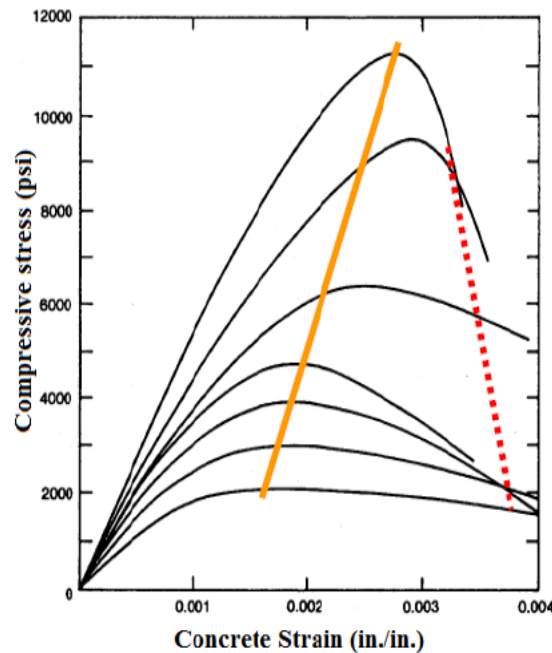


Figure 2-9 Various concrete stress-strain relationships
[Adapted from Wight and MacGregor (2009), reproduced from Whittaker (2012)]

2.2.2 High-Strength Steel Fibre-Reinforced Concrete (HSFRC)

Steel fibre-reinforced concrete (SFRC) is a material that is composed of Portland cement concrete and a distribution of randomly oriented steel fibres. Steel fibres are defined as short, discrete lengths of steel, with various cross-sections and anchorage properties (Kosmatka et al., 2002). This variation in shape allows the fibres to improve resistance to pullout from the cement matrix, especially for steel fibres having hooked ends. When compared to conventional concrete, SFRC shows improved tensile resistance, post-cracking capacity and toughness (Aoude, 2008).

As previously discussed, high-strength concrete (HSC) is brittle when it reaches its peak load; the addition of steel fibres to HSC improves its compressive behaviour and prevents the sudden failure of the material after peak which, in return, increases its post-peak toughness. A series of steel fibre-reinforced concrete (SFRC) cylinders were tested under compressive loads by Bencardino et al. (2008) in order to develop a model for predicting stress-strain behaviour of SFRC in compression. The results showed that the provision of fibres leads to an enhancement in the post-peak strength, as well as an improvement in the overall toughness when compared to plain concrete. Mansur et al. (1999) studied the effect of steel fibres on the compressive behaviour of high-strength concrete. The concrete in the study had compressive strengths which varied between 70 and 120 MPa, while the volumetric ratio of steel fibres ranged from 0.5 to 1.5%. The results show that although the peak strength is not affected by fibres, the post-peak resistance of HSC improves as fibres are added. Furthermore, the enhancement in post-peak responses increases as the steel fibre content increases, as shown in Figure 2-10. The study also noted that the addition of steel fibres changes the failure response of HSC from brittle to a more ductile behaviour.

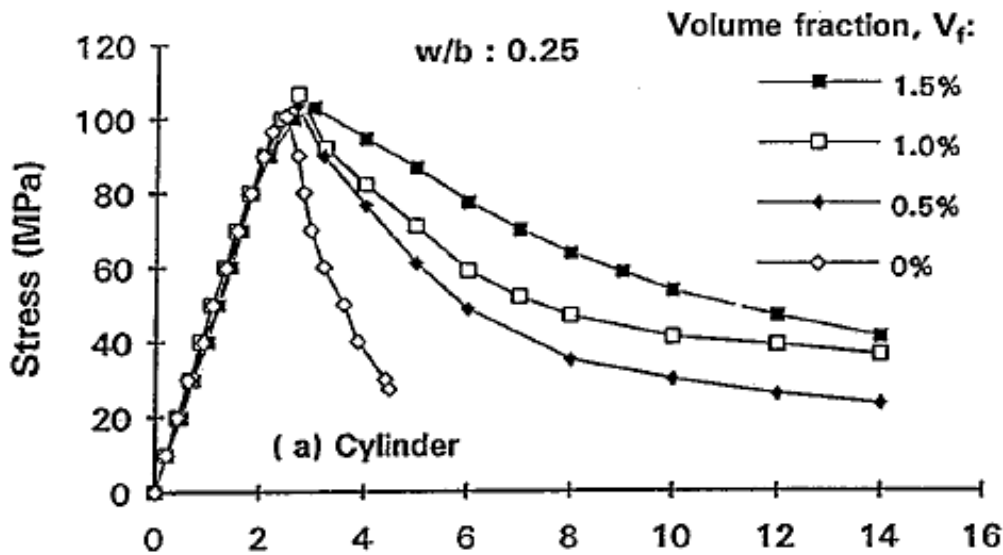


Figure 2-10 Typical stress-strain response of cylindrical HSFRC specimens tested under compression
[Adapted from Mansur et al., (1999)]

2.2.2.1 Stress-Strain Relationship for High-Strength Fibre-Reinforced Concrete (HSFRC) Under Tension

It is well known that concrete shows very weak and brittle behaviour when subjected to tensile stresses. The provision of steel fibres transforms this behaviour and allows concrete to carry stresses after cracking, resulting in improvement in

post-cracking toughness. In addition, the peak tensile resistance can increase when steel fibres are added. Figure 2-11 shows the results from a study conducted by Lee (1993) where dog-bone shaped plain concrete and SFRC specimens were tested in direct tension; it can be seen that the addition of fibres and increase in fibre content results in an improvement in peak tensile resistance, post-cracking capacity and ductility.

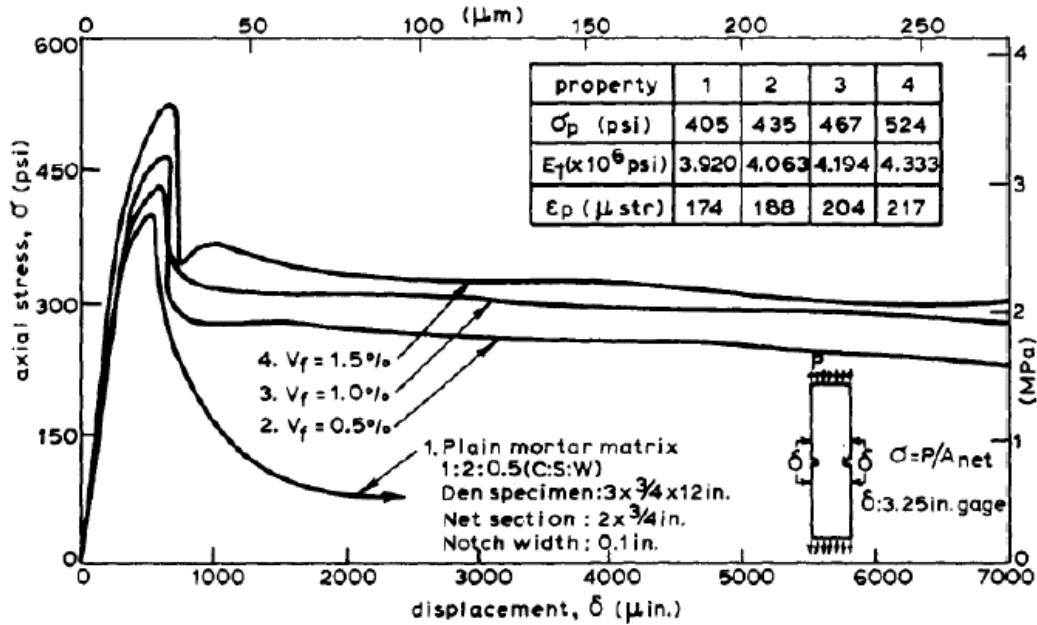


Figure 2-11 Stress-displacement curves for plain and FRC from direct tension tests
[Adapted from Lee (1993)]

2.3 Previous Research on the Behaviour of High-Strength Concrete (HSC) and High-Strength Fibre-Reinforced Concrete (HSFRC) Beams under Flexural Static Loads

2.3.1 High-Strength Concrete (HSC) Beams

This section presents previous research on the behaviour of high-strength reinforced concrete beams when subjected to static flexural loads. Most studies have focused on the effect of the longitudinal reinforcement ratio, the influence of the presence of stirrups, and the effect of concrete compressive strength. A number of these research papers are summarized in the following sections. Table 2-5 summarizes some of the studies which have examined the flexural response of HSC beams.

Table 2-5 Summary of the research studies on HSC beams

Authors	Geometry					Concrete strength	Longitudinal reinforcement		Transverse reinforcement		
	Beams No.	b_w (mm)	h (mm)	d (mm)	a/d	f'_c (MPa)	f_y (MPa)	ρ_{long} (%)	f_y (MPa)	ρ_{trans} (%)	S (mm)
Ahmad et al. (1995)	32	102	204	179	0.6-3.7	33-93	413	2.48	-	0.35	89
Sarkar et al. (1997)	18	150	250	207-220	-	80-120	442-470	1.03-4.04	-	0.14	150
Ashur (2000)	9	200	250	215	6	47-102	530	1.18-2.37	-	0.12	150
Magureanu et al. (2008)	12	125	250	-	-	85-92	355	2.0-3.36	255	0.08	300

2.3.1.1 A Study by Ahmad et al. (1995) on the Effects of Lateral Reinforcement

Ahmad et al. (1995) reported findings from their research on the effects of transverse reinforcement on the shear capacity of high-strength reinforced concrete beams. In this investigation, thirty-two beams were tested under four-point bending. The main variables investigated were the effects of concrete strength, web reinforcement, and shear span-to-depth ratio. The beam cross-section was 102 x 204 mm with a clear cover of 25 mm and a depth of 179 mm. Both normal and high-strength concrete were used with compressive strengths which varied between 33-49 MPa and 75-93 MPa for normal and high-strength concrete, respectively. The ratio of longitudinal reinforcement was kept constant for all beams and was provided by four steel bars having a diameter of 12.7 mm and a yield strength of 413 MPa. Smooth steel bars with a diameter of 6.35 mm were used for stirrups and for the compression reinforcement. The transverse steel bars were installed either in the shear span only or in the flexural span at a spacing of 89 mm, and were denoted as 'type A' for stirrups in the shear span only, and 'type B' for stirrups in the flexural span only. Shear span-to-depth ratio (a/d) varied between 0.6, 1.7, 2.1, and 3.7 in order to observe the effect of a/d ratio on shear capacity. A number of the conclusions and observations of this study are listed below (Ahmad et al., 1995):

- It was observed that the load-carrying capacity increased as the shear span-to-depth ratio decreased, with both NSC and HSC beams showing a reduction in capacity of ~ 60 kN as the a/d ratio increased from 0.6 to 3.7 (see Figure 2-12).
- The variation of concrete strength had a noticeable effect on capacity for beams with an a/d ratio of less than 2.0, where HSC provided an increase in load resistance when compared to NSC. As the a/d ratio increased beyond 2.0, the NSC and HSC beams carried almost the same load (see Figure 2-12).
- The effect of web reinforcement was absent for beams with an a/d ratio of 0.6 for both type A and type B configurations. However, for an a/d ratio of more than 0.6, stirrups affected beam behaviour by changing the beam failure response from a brittle to ductile. For an a/d of 1.7, both type A and type B specimens showed ductility in the NSC beams as well as in the HSC beams (see Figure 2-12).
- Installing transverse reinforcement in shear spans provided better ductility than the provision of stirrups in the constant moment region. Beams with an a/d of 3.7 exhibited large deformations for type A beams, while type B beams

were brittle. At this ratio, HSC was found to decrease the ductility of the beams when compared to NSC (see Figure 2-12).

- The authors compared the shear capacities of the tested beams with predicted shear strengths provided by the equations in the ACI 318-89, BS 8110, and CEB-FIP codes. The results showed good agreement with the CEB-FIP code, while the equations provided by the ACI 318-89 and BS 8110 overestimated the ultimate shear capacity of the beams.

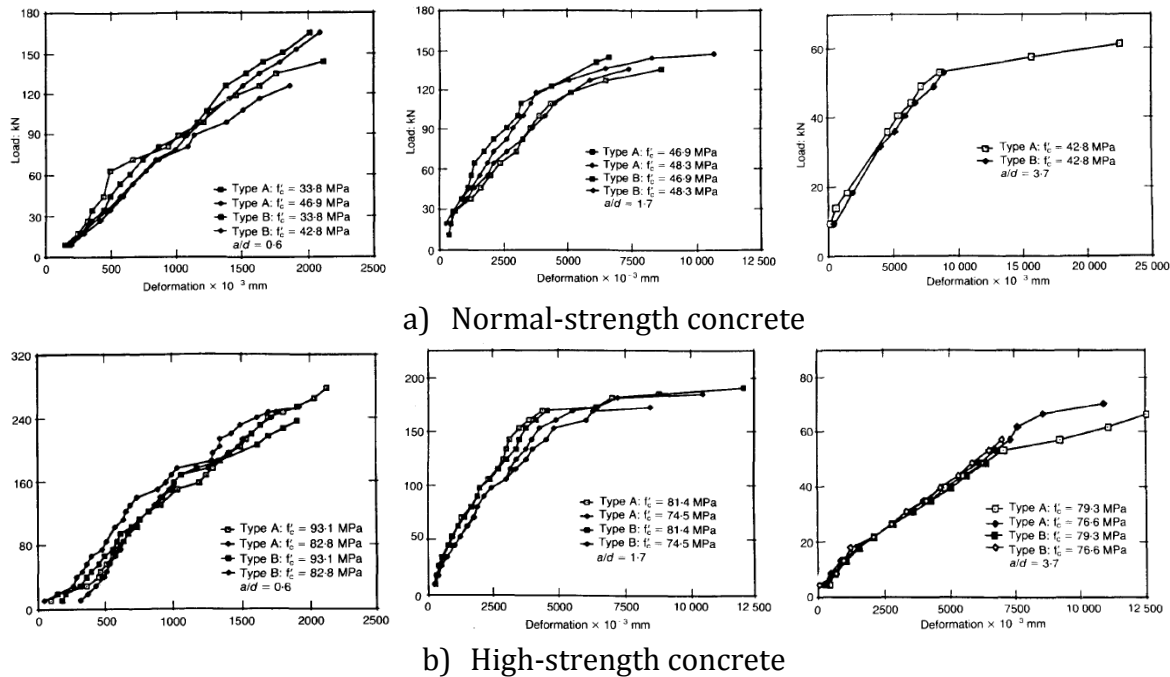


Figure 2-12 experimental results of the load-deflection curve for $a/d = 0.6$ to 3.7
[Adapted from Ahmad, S. H. et al (1995)]

2.3.1.2 A Study by Sarkar et al. (1997) on Flexural Behaviour

In a study by Sarkar et al. (1997), experiments were conducted on the flexural behaviour of high-strength reinforced concrete beams. The main variables in this study included the effects of concrete compressive strength and the steel tensile reinforcement ratio. Eighteen beams were tested in four-point bending having a clear span of 3240 mm between the two supports and a cross-section of 150 x 250 mm. The concrete compressive strength varied between 80 and 120 MPa. A variety of flexural reinforcement ratios were used (1.03%, 1.42%, 1.94%, and 4.04%), representing diameters of 12 mm and 20 mm steel bars with yield strengths of 470 and 442 MPa, respectively. Transverse reinforcements with a diameter of 6 mm were arranged in the shear spans at a spacing of 150 mm. The eighteen beams were divided into four groups (HSC1, HSC2, HSC3, and HSC4), based on the reinforcement

ratio, where the HSC1 and HSC2 groups were reinforced with the 12 mm flexural steel bars and the HSC3 and HSC4 groups were reinforced with the 20 mm longitudinal steel bars. Some of the conclusions and observations of the study are outlined below (Sarkar et al., 1997):

- Considering the load-deflection curve, beams with a higher reinforcement ratio exhibited a stiffer response. When compared to beam HSC2-1, the stiffness for beam HSC4-1 increased considerably by 56%. In addition, concrete compressive strength also affected the beam stiffness. An increase in stiffness of approximately 32% was observed when comparing beam HSC2-2 to beam HSC2-3, as shown in Figure 2-13.
- In terms of maximum deflection, lower deflection was observed as the flexural reinforcement ratio increased. Beam HSC4-1 ($\delta = 35$ mm) showed a 30% lower deflection at ultimate load when compared to beam HSC3-1 ($\delta = 50$ mm), as shown in Figure 2-13.
- Regarding the ductility index, both the reinforcement ratio and concrete compressive strength played a crucial role in changing the beam ductility. As the longitudinal reinforcement ratio increased, the ductility index decreased such that the ductility index dropped from 7.4 for beam HSC1-1 to 1.0 for beam HSC4-1. However, the ductility index declined from 7.4 for beam HSC1-1 to 4.46 for beam HSC1-3 due to higher concrete compressive strength, as shown in Table 2-6.
- Crack spacing decreased as the flexural reinforcement ratio and concrete compressive strength increased. The crack spacing dropped from 108 mm for beam HSC3-3 to 93.98 mm for beam HSC3-1 when concrete compressive strength increased from 78 MPa to 107 MPa. For the tensile reinforcement ratio, crack spacing gradually narrowed from 143 mm for beam HSC2-1 to 54.60 mm for beam HSC4-1 as ρ went from 1.42% to 4.04%, as shown in Table 2-6.

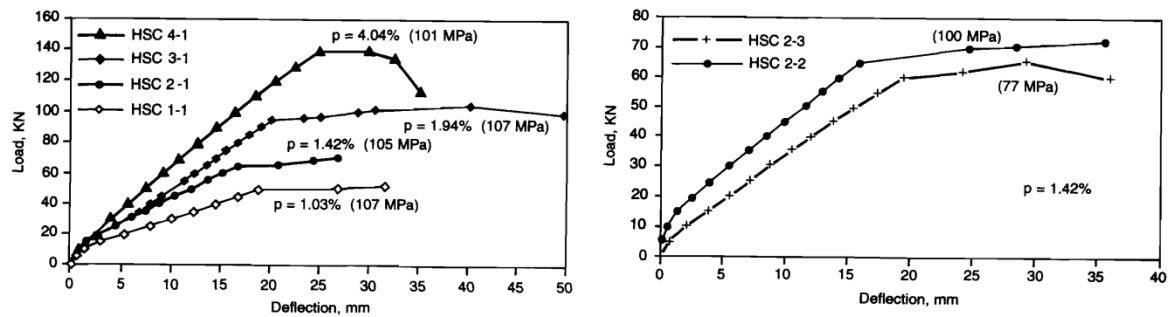


Figure 2-13 Experimental results for load-deflection curve
[Adapted from Sarkar, S. et al (1997)]

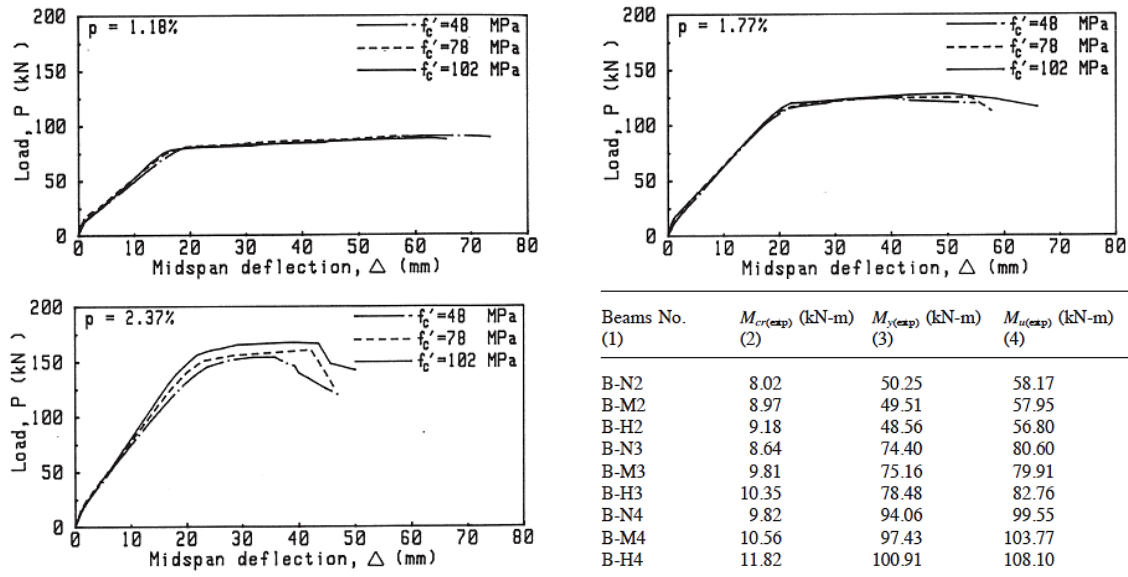
Table 2-6 Experimental results of yield and ultimate load, ductility index, and crack spacing
[Adapted from Sarkar, S. et al (1997)]

Fabrication and beams details						Yield stage		Ultimate stage		Ductility index	Average measured and calculated crack spacings		
Beam	f_{cu} (MPa)	E_c (MPa)	d (mm)	A_s (mm ²)	ρ (%)	Load (kN)	ψ_y (rad/m)	Load (kN)	ψ_u (rad/m)	$\mu = \psi_u / \psi_y$	Measured surface crack (mm)	Calc. surface crack (mm)	Ratio
HCS1-1	107	40640	220.0	339	1.03	45.10	0.01190	59.00	0.088	7.40	74.36	81.86	0.91
HSC1-2	97	41000	220.0	339	1.03	44.90	0.0168	54.00	0.089	5.30	132.00	81.86	1.61
HSC1-3	85	38550	220.0	339	1.03	49.00	0.0130	57.00	0.058	4.46	112.27	81.86	1.37
HSC2-1	105	49650	212.5	452	1.42	61.00	0.0118	70.20	0.056	4.75	143.00	79.87	1.79
HSC2-2	100	43600	212.5	452	1.42	63.30	0.0121	71.00	0.058	4.79	NA	NA	NA
HSC2-3	77	34000	212.5	452	1.42	60.00	0.0139	66.00	0.044	3.17	77.64	79.87	0.97
HSC2-4	90	40487	212.5	452	1.42	60.00	0.0147	74.00	0.059	4.01	NA	NA	NA
HSC3-1	107	53000	215.0	628	1.94	82.79	0.0118	103.0	0.053	4.49	93.98	83.44	1.13
HSC3-2	85	39000	215.0	628	1.94	75.00	0.0193	101.0	0.071	3.68	79.61	83.44	0.95
HSC3-3	78	34005	215.0	628	1.94	85.00	0.0198	99.00	0.0561	2.83	108.00	83.44	1.30
HSC4-1	101	41200	207.5	1257	4.04	125.00	0.0259	145.0	0.026	1.00	54.60	70.29	0.78
HSC4-2	87	42633	207.5	1257	4.04	128.00	0.033	140.46	0.0364	1.10	67.77	70.29	0.96
HSC4-3	82	40698	207.5	1257	4.04	140.57	0.024	169.00	0.029	1.20	70.41	70.29	1.00

2.3.1.3 A Study by Ashour (2000) on the Effects of Compressive Strength and the Tensile Reinforcement Ratio

Ashour (2000) investigated the effects of compressive strength and the tensile reinforcement ratio on the flexural behaviour of high-strength concrete beams. Nine beams were tested in four-point bending with a clear span of 3080 mm and a cross section of 200 mm x 250 mm. Three types of concrete compressive strength were used (47 MPa for normal strength, 78.5 MPa for medium strength, and 102.4 MPa for high strength). A variety of longitudinal reinforcement ratios were studied (1.18%, 1.77%, and 2.37%) using bars having a diameter of 18 mm and a yield strength of 530 MPa. Transverse reinforcements with a diameter of 8 mm were spaced at 150 mm, with two top steel bars with a diameter of 6 mm only included in the shear spans in order to avoid shear failure. The aim of this study was to observe the effects of concrete compressive strength and the flexural tensile reinforcement ratio on load-deflection behaviour and displacement ductility of HSC beams. Some of the conclusions and observations are outlined below (Ashour, 2000):

- The effects of concrete compressive strength appeared to be considerable when comparing the cracking moment for each reinforcement ratio. The cracking moment increased by 14.46% for rebar ratio 1.18%, by 19.8% for rebar ratio 1.77%, and by 20.4% for rebar ratio 2.37% when comparing normal-strength concrete with high-strength concrete ($M_{cr} [\rho=1.18\%] = 8.02$ kN.m for normal and 9.18 kN.m for high, $M_{cr} [\rho=1.77\%] = 8.64$ kN.m for normal and 10.35 kN.m for high, and $M_{cr} [\rho=2.37\%] = 9.82$ kN.m for normal and 11.82 kN.m for high), as shown in Figure 2-14(a).
- Increasing the longitudinal reinforcement ratio resulted in significantly improved performances by all the beams. For example, for the high-strength concrete beams, the ultimate moment increased from 56.80 kN-m at $\rho = 1.18\%$ rebar ratio, to 82.76 kN-m and 108.1 kN-m at $\rho = 1.77\%$ and 2.37% , respectively.
- Displacement ductility was effective as concrete compressive strength increased from 48 MPa to 78 MPa, showing increase factors of approximately 14.45%, 8.0%, and 23.49% for reinforcement ratios of $\rho = 1.18\%$, 1.77% and 3.88% ($\mu_d [\rho=1.18\%] = 3.39$ and 3.88 , $\mu_d [\rho=1.77\%] = 2.50$ and 2.70 , and $\mu_d [\rho=2.37\%] = 1.49$ and 1.84), as shown in Figure 2-14(b).
- However, when the concrete strength increased from 78 MPa to 102 MPa, the beams showed lower ductility as the ratio of flexural reinforcement increased. A reduction in displacement ductility of 4.38%, 10.0%, and 1.63% occurred when compared with medium concrete strength for the various reinforcement ratios ($\mu_d [\rho=1.18\%] = 3.88$ and 3.71 , $\mu_d [\rho=1.77\%] = 2.70$ and 2.43 , and $\mu_d [\rho=2.37\%] = 1.84$ and 1.81), as shown in Figure 2-14(b).



a) load-deflection curves and moment results

Beam (1)	Δ_y (mm) (2)	Δ_o (mm) (3)	μ_d (4)
B-N2	17.99	60.93	3.39
B-M2	16.47	63.95	3.88
B-H2	16.77	62.29	3.71
B-N3	20.38	50.99	2.50
B-M3	20.10	54.24	2.70
B-H3	20.52	49.82	2.43
B-N4	23.87	35.70	1.49
B-M4	22.96	42.20	1.84
B-H4	20.94	37.94	1.81

b) displacement ductility

Figure 2-14 Experimental results of the flexural behaviour of HSC beams

[Adapted from Ashour (2000)]

2.3.1.4 A Study by Magureanu et al. (2008) on Flexural Behaviour

Magureanu et al. (2008) reported findings on a study on the flexural behaviour of high-strength reinforced concrete beams. Twelve beams were tested in four-point bending having a clear span of 3000 mm between the two supports and a cross-section of 125 x 250 mm. The concrete compressive strength varied between 85 and 92 MPa. A variety of flexural reinforcement ratios were included (2.033, 2.621, and 3.357%). Transverse reinforcement with a diameter of 6 mm was provided in the shear spans at a spacing of 300 mm. The yield strength was 355 MPa for the longitudinal bars and 255 MPa for the lateral reinforcement. The main variables observed were the effects of the flexural reinforcement ratio and the factors affecting the design of flexural members in the European code. Some of the conclusions and observations of the study are noted below (Magureanu et al., 2008):

- The researchers observed that the maximum moment increased as the ratio of longitudinal reinforcement increased. When compared to beams with a

low reinforcement ratio, the ultimate flexural capacity increased by a factor of approximately 47% for beams with a higher reinforcement ratio (moment [2.033%]= 53 kN-m and moment [3.357 %]= 78 kN-m), as seen in Figure 2-15.

- The failure mode was flexural failure with concrete crushing in the compression region for all beams.
- The experimental results were compared with predictions for ultimate moment capacity in the European and Romanian codes; the experimental results were in good agreement with that predicted by the codes.

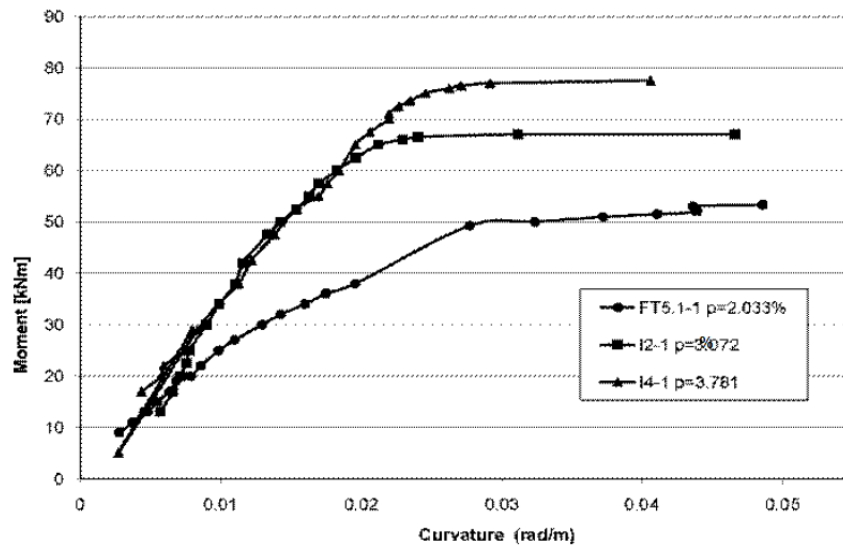


Figure 2-15 Experimental results of moment-curvature curve
[Adapted from Magureanu C., et al. (2008)]

2.3.2 High-Strength Fibre-Reinforced Concrete (HSFRC) beams

This section presents previous research on the behaviour of high-strength fibre-reinforced concrete beams (HSFRC). Most studies have focused on the effects of steel fibre on shear strength, flexural capacity, and the size effect of HSFRC beams. Table 2-7 summarizes some of the studies which have examined the flexural response of HSFRC beams. Details about these investigations and their outcomes are outlined in the subsequent sections.

Table 2-7 Summary of research studies on HSFRC beams

Authors	Geometry					Material Properties				Fibres				
	Beams No.	b _w (mm)	h (mm)	d (mm)	a/d	f' _c (MPa)	f _y (MPa)	ρ _{long} (%)	ρ _{trans} (%)	Type	L _f (mm)	d _f (mm)	f _y (MPa)	V _f (%)
Ashur et al. (1992)	18	125	250	215	1-6	92-101	460-500	0.374-4.58	-	HE	60	0.8	260	0-1.5
Imam et al. (1994)	16	200	350	300	1.75-4.5	108-111	550	1.87-3.08	-	HE	60	0.8	2000	0-0.75
Casanova et al. (1999)	5	125	250	225	2.89	90	540-565	2.2-3.5	0.2	HE	30	0.5	1200	1.20
Gustafsson et al. (1999)	20	200	250	180	3.3	98-129	590	4.47	-	ST	6.0	0.15	2600	0.5
		200	500	410	2.9			3.06		HE	30.0	0.6	1100	0.75
		300	700	570	3.0			2.87		HE	60.0	0.7	2600	1.0
Kwak et al. (2003)	12	125	250	212	2-4	31-65	442	1.5	-	-	50	0.8	1079	0.5-0.75
Guen et al. (2013)	11	150	250	-	-	60	-	0.77-2.7	0.31	HE	32	0.9	-	0.5-2.0
			300											
			350											

HE = Hooked end steel fibre

ST = Straight steel fibre

2.3.2.1 A Study by Ashour et al. (1992) Testing the Shear Behaviour of High-Strength Fibre-Reinforced Concrete (HSFRC) Beams

In this study, Ashour et al. (1992) tested the shear behaviour of high-strength fibre-reinforced concrete (HSFRC) beams. A total of 18 rectangular beams were tested under four-point bending tests. The beams were constructed with no shear reinforcements in order to investigate the effect of steel fibres under shear. The variables were the shear span-to-depth ratio, longitudinal reinforcement ratios, and the fraction volume of steel fibres. These beams were grouped into three series;

For series one, the shear span-to-depth ratio varied between $a/d = 2, 4$, and 6 and the ratios of reinforcement were 0.374% for longitudinal steel bars and 1.0% volumetric ratio for steel fibre. The second series used three different volumes of steel fibres ($0.5, 1.0$, and 1.5%) and four different shear span-depth ratios ($a/d = 1, 2, 4$, and 6) with the ratio of longitudinal reinforcement fixed at 2.835% . The beams in the third series had reinforcement ratio of 4.58% and had steel fibre content of 1.0% , while the shear span-depth ratio varied between $a/d = 2, 4$, and 6 . The cross-section was kept constant for all beams, 125×250 mm, while the span length varied between 930 mm for a/d of 1.0 , 1360 mm for a/d of 2.0 , 2220 mm for a/d of 4.0 , and 3080 mm for a/d of 6.0 . The concrete compressive strength varied between 92 and 101 MPa, while hooked-end fibres with a length of 60 mm and a diameter of 0.8 mm were used in the HSFRC beams. The beam nomenclature reflects the shear span-depth ratio, volume of steel fibres, and ratio of longitudinal reinforcement. For example, B-2.0-1.5-A is a beam with $a/d = 2.0$, 1.5 by volume of steel fibres, and an average ratio of longitudinal reinforcement of 2.84% (with L and M representing low and maximum ratios of 0.374% and 4.58%). Some of the conclusions and observations of the study are summarized below (Ashour et al., 1992):

- Shear span-depth ratio had an important effect on the shear strength, with shear capacity reducing as the a/d ratio increased. For example, the shear capacities dropped from 1.68 MPa to 0.56 MPa respectively for beams B-2-1.0-L and beam B-6-1.0-L, resulting in a reduction factor of approximately 66.7% as the a/d ratio went from 2 to 6 , as seen in Figure 2-16.
- Increasing the percentage of the longitudinal steel bars resulted in an improvement in shear capacity. As compared to beams with a low ratio of longitudinal reinforcement, the shear strength for beams with the maximum percentage of reinforcement increased significantly by 300% (from 1.68 MPa for beam B-2-1.0-L to 6.73 MPa for beam B-2-1.0-M), 334% (from 0.893 MPa for beam B-4-1.0-L to 3.88 MPa for beam B-4-1.0-M), and 423% (from 0.56 MPa for beam B-6-1.0-L to 2.93

MPa for beam B-6-1.0-M) as the reinforcement ratio increased from low to maximum (L to M), as seen in Figure 2-16.

- As the steel fibre content increased, the beams showed an increase in shear strength. For example, when comparing the beams in the A series (average ρ), shear strength increased by 53.47% and 9.50% when comparing beam B-1-1.5-A (with shear strength, $v = 13.95$ MPa) to beams B-1-0.5-A ($v = 9.09$ MPa) and B-1-1.0-A ($v = 12.74$ MPa), as seen in Figure 2-16.
- It was found that a high volume content of steel fibres transformed the beam failure from a brittle mode to a more ductile one. Furthermore, the beam stiffness increased when steel fibres were added, causing a reduction in the beam's deflection for a given load level.
- Two empirical equations were developed to predict the shear strength of HSFRC beams. It was found that the two empirical equations showed good results when compared to other equations proposed by other researchers.

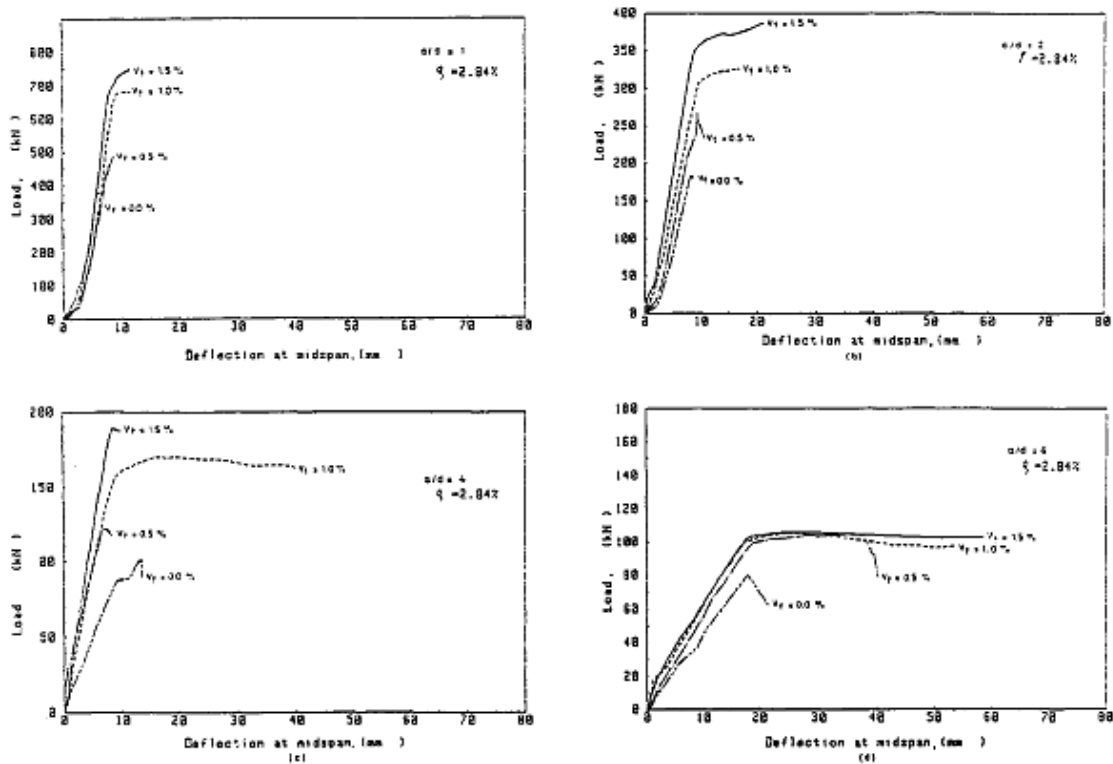


Figure 2-16 Effect of fibre content on load-deflection relationship

[Adapted from Ashour et al. (1992)]

2.3.2.2 A Study by Imam et al. (1994) on the Use of Steel Fibres as Shear Reinforcement

Imam et al. (1994) investigated the benefits of using steel fibres in high-strength concrete beams as shear reinforcement. Sixteen rectangular beams (200 x 350 x 3600 mm) were tested under four-point bending with concrete compressive strength of 110 MPa. The beams were divided into four groups based on the shear span-to-depth ratio, with four beams tested in each group. The main variables were shear span-to-depth ratio, the steel fibre content, and the ratio of the longitudinal reinforcement. None of the beams had shear reinforcement and were reinforced longitudinally with 22 mm bars for groups one and two (ratio of 1.87%), and with 28 mm bars for groups three and four (ratio of 3.08%). Hooked-end steel fibres were used with a length of 60 mm and a diameter of 0.8 mm where the volume content of the fibre was 0.0% for groups one and three, and 0.75% for groups two and four. The yield strength was 550 MPa for flexural reinforcement and 2000 MPa for the steel fibres. Four different shear span-to-depth ratios were used in each group (1.75, 2.5, 3.5, and 4.5). Some of the conclusions and observations of the study are outlined below (Imam et al., 1994):

- Shear span-to-depth ratio played a crucial factor in decreasing the ultimate shear strength. When comparing beams in group one that had the same ratio of longitudinal reinforcement and no steel fibres, the nominal shear strength dropped significantly by 67.29% for B9 ($v = 1.863$ MPa) for a/d of 2.5, by 72% for B8 ($v = 1.593$ MPa) for a/d of 3.5, and by 74.42% for B10 ($v = 1.457$ MPa) for a/d of 4.5, in comparison to beam B14 ($v = 5.696$ MPa) which had a/d ratio of 1.75, as shown in Table 2-8.
- The presence of steel fibres significantly increased the nominal shear strength of the beams. When comparing group I to group II, the ultimate shear strength increased considerably from 5.696 MPa (B14) to 6.737 MPa (B15) for a/d of 1.75, from 1.863 MPa (B9) to 4.483 MPa (B5) for a/d of 2.5, from 1.593 MPa (B8) to 3.292 MPa (B4) for a/d of 3.5, and from 1.457 MPa (B10) to 2.520 MPa (B11) for a/d of 4.5, showing a percentage increase factors of 18.28%, 140.63%, 106.65%, and 72.96% respectively, as shown in Table 2-8.
- Beams with steel fibres showed a reduction in deflection for a given load of 98 kN by 13.48% for a/d of 1.75 (B14 = 1.78 mm and B15 = 1.54 mm), by 28.70% for a/d of 2.5 (B9 = 3.38 mm and B5 = 2.41 mm), by 21.19% for a/d of 3.5 (B8 = 4.53 mm and B4 = 3.57 mm), and by 19.52% for a/d of 4.5 (B10 = 5.89 mm and B11 = 4.74 mm), when compared to companion beams without fibres.

- Increasing the ratio of longitudinal reinforcement raised the shear resistance capacity of the beams. In comparison to group one, group three showed an increase in shear capacity of 27.26% for a/d of 1.75(B14 = 5.696 MPa and B1 = 7.249 MPa), 123.0% for a/d of 2.5(B9 = 1.863 MPa and B2 = 4.161 MPa), 35.10% for a/d of 3.5(B8 = 1.593 MPa and B3 = 2.152 MPa), and 47.43% for a/d of 4.5(B10 = 1.457 MPa and B13 = 2.148 MPa).
- Failure mode changed from a brittle into a ductile one when steel fibres were added to the beams. Group two exhibited a change in failure from shear to flexural for all types of shear span-to-depth ratios when compared with group one which contained no steel fibres.
- Numerical analysis was conducted to predict the ultimate shear strength of steel fibre high-strength concrete beams without stirrups. The equations showed good agreement with the experimental results.

Table 2-8 Experimental results for the sixteen HSC and HSFRC beams
[Adapted from Imam et al. (1994)]

Group	Beam	a/d	V_f %	ρ %	f_{cyt} MPa	f_{sp} MPa	P^* KN	v_u^* MPa	δ^{**} mm	Type of Failure
I	B14	1.75	0.0	1.87	109.5	7.1	683.55	5.696	1.78	S ^{***}
	B9	2.5	0.0	1.87	108.5	7.0	223.60	1.863	3.38	S
	B8	3.5	0.0	1.87	111.0	7.2	191.14	1.593	4.53	S
	B10	4.5	0.0	1.87	111.5	7.3	174.86	1.457	5.89	S
II	B15	1.75	0.75	1.87	108.5	11.9	808.39	6.737	1.54	FL ^{***}
	B5	2.5	0.75	1.87	110.0	11.8	537.91	4.483	2.41	FL
	B4	3.5	0.75	1.87	109.0	13.0	395.03	3.292	3.57	FL
	B11	4.5	0.75	1.87	110.5	12.9	302.35	2.520	4.74	FL
III	B1	1.75	0.0	3.08	109.5	7.0	869.88	7.249	1.76	S
	B2	2.5	0.0	3.08	110.0	7.3	499.27	4.161	2.39	S
	B3	3.5	0.0	3.08	112.0	7.4	258.22	2.152	2.87	S
	B13	4.5	0.0	3.08	109.0	7.0	257.73	2.148	3.81	S
IV	B16	1.75	0.75	3.08	109.5	11.8	1056.51	8.804	1.48	S
	B6	2.5	0.75	3.08	110.0	12.0	568.81	4.740	2.26	S
	B7	3.5	0.75	3.08	111.5	12.8	417.58	3.480	2.82	S
	B12	4.5	0.75	3.08	110.8	12.4	423.66	3.531	3.82	S

* $P = 2 V$ = The Ultimate Failure Load v_u = The Ultimate Shear Strength = V/bd .

** δ = Midspan Deflection Measured at Load (P) = 98.07 KN (10 tons).

*** S = Shear Failure, FL = Flexural Failure.

2.3.2.3 A Study by Casanova and Rossi (1999) on the Effects of Stirrups and Steel Fibres in RC Beams Submitted to Shear

Casanova and Rossi (1999) conducted experiments to examine the contributions of stirrups and steel fibres to shear resistance in high-strength RC beams. Five rectangular beams (125 x 250 x 2300 mm) were tested under four-point bending with a concrete compressive strength of 90 MPa. Two beams denoted as HSRC1 and HSRC2 were cast as plain high-strength concrete with shear reinforcement consisting of 8 mm bars placed at a spacing of 180 mm and 140 mm, respectively. The yield strength of stirrups was 490 MPa for both beams. The remaining beams, denoted as HSFRC1, HSFRC2, and HSFRC3, were reinforced with a 100 kg/m³ (volumetric ratio, $V_f = 1.2\%$) of hooked-end steel fibres (30 mm long, 0.5 mm diameter, 1200 MPa yield strength) instead of shear reinforcement. The beams with transverse reinforcement were reinforced longitudinally with two 25 mm bars having yield strength of 565 MPa. HSFRC beams had two 25 mm bars for HSFRC1 and HSFRC2, and two 20 mm bars for HSFRC3, where the yield strength of the bars was 540 MPa. Some of the conclusions and observations of the study are described below (Casanova & Rossi, 1999).

- The five beams showed different modes of failure in tension and compression zones. Beam HSFRC3 exhibited a tension bending failure due to a lower longitudinal reinforcement. Beam HSRC1 failed with a diagonal tension failure due to a lower shear reinforcement, while beam HSRC2 failed in bending with failure of the concrete in the compression zone. Beams HSFRC1 and HSFRC2 showed similar failures to beams HSRC1 and HSRC2 but with reduced shear cracks for HSFRC2 and without brittleness for HSFRC1 due to the presence of steel fibres. The fifth beam, HSFRC3, failed in bending in the tension region and showed large ductility, as seen in Figure 2-17.
- The author monitored the shear crack opening in the beams. The HSFRC2 beam showed a significant reduction in the crack opening of 59.38% when compared to HSRC2 (HSRC2 = 0.32 mm wide and HSFRC2 = 0.13 mm wide) at a load of 200 kN. These results show that steel fibre provides a higher post-cracking residual stress and ductility for the HSFRC beams compared to that of the HSRC beams.
- A numerical analysis was conducted to get an equivalence relationship between shear reinforcement and steel-fibre reinforced concrete.

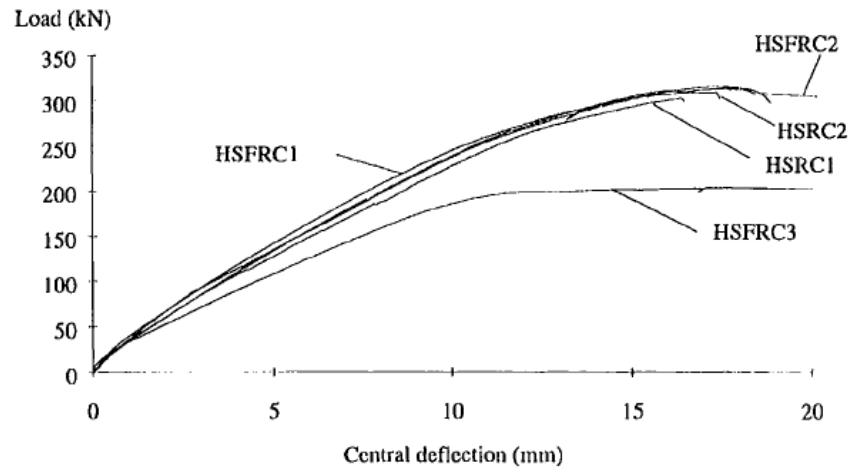


Figure 2-17 Load vs. mid-span deflection relationship
[Adapted from Casanova & Rossi (1999)]

2.3.2.4 A Study by Gustafsson & Noghabai (1999) on the Use of Steel Fibres for Lateral Reinforcement

In this study, Gustafsson and Noghabai examined the effect of steel fibres as a replacement for lateral reinforcement in high-strength concrete beams. The study also examines the effect of steel fibres on size effect in high-strength concrete beams. Twenty rectangular beams with varying compressive strength between 98 and 129 MPa were divided into three series (small, medium, and large specimens). The small beams (S) were tested under the three-point loading method while the medium (M) and large (L) beams were tested under four-point bending in order to keep shear span-to-depth ratios (a/d) constant at a value of about 3. The total length of the beams was 1500 mm for series S with a cross-section of 200 x 250 mm, 3600 mm for series M with a cross-section of 200 x 500 mm, and 6000 mm with a cross-section of 300 x 700 mm. Three types of steel fibre were used (type I was a straight fibre with a 0.15 mm diameter and 6 mm length, type II was a hooked at the ends with a 0.6 mm diameter and 30 mm length, and type III was a hooked at the ends with a 0.7 mm diameter and 60 mm length). The volumetric ratios of steel fibres varied between 0.5%, 0.75%, and 1%. One specimen of each series was built with a mix of 0.5 vol% of type I and 0.5 vol% of type II (marked as MIX in the nomenclature). Longitudinal reinforcements were installed in all beams with different sizes. Series S beams had six 16 mm bars, while series M and L beams had eight 20 mm and ten 25 mm bars respectively. Transverse reinforcement consisting of 8 mm stirrups was used in one specimen of each series and was arranged at a spacing of 130 mm for the S series, 300 mm for the M series, and 400 mm for the L series (marked as STIRRUPS in the nomenclature). In addition, two beams were cast without stirrups or fibres and were marked as REF. Some of the conclusions and observations of the research are outlined below (Noghabai & Gustafsson, 1999):

- In series S, the provision of 1.0% volumetric ratio of steel fibres increased the ultimate shear strength by 42% for beam S-6/0.15 ($V_u = 299$ kN) while by only 12% for S-STIRRUPS beam ($V_u = 237$ kN), when compared to the reference beam ($V_u = 211$ kN). However, when fibre content decreases from 1.0% to 0.5-0.75%, the ultimate shear capacity decreases as in specimens S-60/07 (I) and (II), which showed a reduction in shear strength of 15.72% for beam S-60/07 (I) ($V_u = 252$ kN) and 12.37% for S-60/07 (II) ($V_u = 262$ kN) as compared to beam S-6/0.15 ($V_u = 299$ kN).
- In series M, the fibrous beams were able to resist the ultimate shear strength by an average of 78% while the M-MIX beam (mixed fibres) resisted the ultimate shear capacity by 107% ($V_u = 367$ kN), when compared to M-REF beam ($V_u = 177$ kN). The M-STIRRUPS beam ($V_u = 329$ kN) increased the shear capacity by 86% in comparison to M-REF beam ($V_u = 177$ kN).
- As a comparison between series S and series M, the need for shear reinforcement increased with the increase of beam size.
- In series L, the fibrous beams showed stiffer behaviour up to the maximum shear strength than the L-STIRRUPS beam. The L-6/0.15 beam with shortest fibres failed at a lower load ($V_u = 445$ kN) than the other fibrous beams with an average shear capacity of 553 kN. The authors noted that longer hooked fibres with better anchorage to the concrete are needed when the beam size increases.
- The types of fibres had an effect on the ultimate shear strength of the HSFRC beams. The mixed-fibre beams proved to be the most effective reinforcement. The short straight fibres in series S were as effective as mixed fibres, whereas in series L the mixed-fibres were too short which resulted in a failure in early stage of loading. However, long hooked fibres enhanced the ultimate shear capacity in series M and L by 339 kN and 509 kN respectively, when compared to series S with ultimate shear strength of 262 kN.
- Beam size was found to have a significant effect in this research. While the difference in ultimate shear stresses between S-MIX and S-6/0.15 was very small, there was a big gap in ultimate shear capacity between L-MIX and L-6/0.15, indicating that the size effect changed the response of the beams that had short steel fibres as the beam size increased.

2.3.2.5 A Study by Kwak et al. (2003) on High-Strength Fibre-Reinforced Concrete Beams without Stirrups

The authors of this study (Kwak et al., 2003) conducted experiments on high-strength fibre-reinforced concrete (HSFRC) beams without stirrups. Twelve beams were grouped into nine beams composed of high-strength concrete and three beams composed of normal-strength concrete. Three clear spans were chosen with different shear span-to-depth ratios (a/d of 2, 3 and 4). The cross-section was kept constant for all beams (125 mm x 250 mm). The compressive strength was 31 MPa for normal-strength concrete and 65 MPa for high-strength concrete. The three volumetric ratios of steel fibre were 0%, 0.5 %, and 0.75% and the fibres were 0.8 mm in diameter, 50 mm long, and had a 1079 MPa nominal yield strength. The ratio of longitudinal reinforcement was 1.5% for all beams, using two steel bars of 16 mm diameter with a yield stress of 442MPa. No stirrups were installed in the beams except for a few lateral reinforcements which were put outside the clear span. The main variables studied were the effect of steel fibres, the influence of shear span-to-depth ratio, and the effect of concrete strength. Some of the conclusions and observations of this research are as follows (Kwak et al., 2003):

- The failure mode was found to change when steel fibres were added to the beams. Beams with an a/d of 2.0 exhibited a transformation in failure mode from pure shear to shear-flexure failure as steel fibres were added. Beams with $a/d = 3.0$ and 4.0 showed a change in failure mode from pure shear to pure flexure (see Table 2-9).
- The effect of shear span-to-depth ratio (a/d) on shear strength was clear. It was observed that the ultimate shear strength decreased when the a/d increased, as shown in Table 2-9. For beams without steel fibres, a reduction in shear stress of 16.2% and 34.4% occurred when the FHB1-2 specimen ($v_u = 3.02$ MPa) was compared with FHB1-3 ($v_u = 2.53$ MPa) and FHB1-4 ($v_u = 1.98$ MPa) respectively.
- Fibre content was significant in increasing the ultimate shear strength of all beams, particularly when the a/d ratio decreased. When compared with FHB1-2 specimen ($v_u = 3.02$ MPa and $v_f = 0\%$) with an a/d of 2.0, the shear stresses were increased by 68.54% for FHB2-2 specimen ($v_u = 5.09$ MPa and $v_f = 0.50\%$) and 80.13% for FHB3-2 specimen ($v_u = 5.44$ MPa and $v_f = 0.75\%$). The effect of increasing fibre content from 0% to 0.5-0.75% on shear strength was relatively more modest for beams with a/d ratios of 3.0 and 4.0. When compared with FHB1-3 specimen ($v_u = 2.53$ MPa and $v_f = 0\%$) with an a/d of 3.0, the increase in ultimate shear strength was 22.13% and 34.39% for FHB2-3 specimen ($v_u = 3.09$ MPa and $v_f = 0.50\%$) and FHB3-3 specimen ($v_u = 3.40$ MPa and $v_f = 0.75\%$) respectively. When compared with FHB1-4 specimen

($v_u = 1.98$ MPa and $v_f = 0\%$) with an a/d of 4.0, the ultimate shear strength was increased by 21.72% and 38.38% for FHB2-4 specimen ($v_u = 2.41$ MPa and $v_f = 0.50\%$) and FHB3-4 specimen ($v_u = 2.74$ MPa and $v_f = 0.75\%$) respectively, as shown in Table 2-9.

- Concrete strength did not significantly affect the ultimate shear strength, although concrete strength did increase the cracking shear stress.
- The empirical equation for estimating shear strength developed by the authors showed a good agreement when compared to the results of the beams tested in this study and in other investigations.

Table 2-9 Experimental results of the twelve beams
[Adapted from Kwak et al. (2003)]

Beam designation	Fiber-volume fraction V_f , %	Shear-span/depth ratio a/d	Concrete compressive strength f'_c , MPa	Average shear stress		Ultimate displacement, mm	Failure mode
				v_{cr} , MPa	v_u , MPa		
FHB1-2	0	2.0	62.6	1.67	3.02	6.08	Shear
FHB2-2	0.5	2.0	63.8	1.94	5.09	16.50	Shear-flexure
FHB3-2	0.75	2.0	68.6	2.22	5.44	34.35	Shear-flexure
FHB1-3	0	3.0	62.6	1.48	2.53	9.68	Shear
FHB2-3	0.5	3.0	63.8	1.67	3.09	18.2	Flexure
FHB3-3	0.75	3.0	68.6	1.80	3.40	33.59	Flexure
FHB1-4	0	4.0	62.6	1.26	1.98	13.86	Shear
FHB2-4	0.5	4.0	63.8	1.54	2.41	28.49	Flexure
FHB3-4	0.75	4.0	68.6	1.57	2.74	43.87	Flexure
FNB2-2	0.5	2.0	30.8	1.30	4.04	8.93	Shear
FNB2-3	0.5	3.0	30.8	1.11	2.55	10.81	Shear
FNB2-4	0.5	4.0	30.8	1.07	2.0	41.07	Flexure

2.3.2.6 A Study by Guan et al. (2013) on Flexural Behaviour

This study investigated the flexural behaviour of high-strength fibre-reinforced concrete beams. Eleven beams were tested in four-point bending with a span of 2700 mm and a constant shear span of 900 mm. The width of the cross section was kept at 150 mm but the height varied between 250 mm, 300 mm, and 350 mm. The concrete compressive strength was 60 MPa. Four steel fibre fractions were used (0.5%, 1.0%, 1.5%, and 2.0%), with a diameter of 0.9 mm and a length of 32 mm. Four different ratios of longitudinal reinforcement were chosen (0.77%, 1.39%, 2.10%, and 2.70%) in order to investigate their effect on beam response. Stirrups were provided in the shear spans at a spacing of 130 mm. The main variables of this study were the effect of steel fibres, the influence of longitudinal

reinforcement ratio, and the effect of beam height. Some of the conclusions and observations of this research are outlined below (Guan et al., 2013):

- The longitudinal reinforcement had a significant effect on ultimate flexural capacity. For example beams with a depth of 300 mm and 1.5% fibres showed, an increase in flexural capacity of 120.5% when the longitudinal reinforcement ratio increased from 0.77% (flexural capacity = 57.15 kN.m) to 2.70% (flexural capacity = 126 kN.m) (see Figure 2-18).
- The ultimate flexural capacity of the beams increased as the steel fibre content increased. For example in the case of beams with 300 mm depth and $\rho = 1.39\%$, increasing the fibre content from 0.5% to 2% resulted in an increase in ultimate flexural capacity by a factor of approximately 16% (flexural capacity [0.5%] = 81 kN.m and flexural capacity [2.0%] = 94 kN.m; see Figure 2-18).
- Varying the beam height also affected ultimate flexural capacity when all other parameters such as fibre content and reinforcement ratio were kept constant.

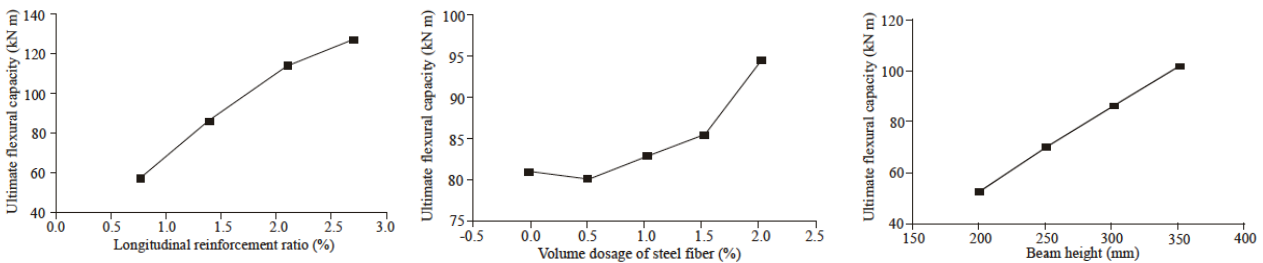


Figure 2-18 experimental results of the eleven beams
[Adapted from Guan et al. (2013)]

2.4 Previous Research on Impact/Blast Behaviour of Reinforced Concrete (RC) and Steel Fibre Reinforced Concrete (SFRC) Beams

This section summarizes some of the previous studies which have been conducted to study the dynamic response of reinforced concrete (RC) elements subjected to impact and blast loads. The research summarized in the following sections examines the effects of blast loads on normal and high-strength reinforced concrete beams/columns with and without steel fibres.

2.4.1 Reinforced Concrete (RC) Beams

2.4.1.1 A Study by Fujikake et al. (2009) on Impact Responses of Reinforced Concrete (RC) Beams

The authors studied the impact responses of reinforced concrete (RC) beams using impact testing and provided an analytical model to predict the maximum mid-span deflection and maximum impact load using a two-degree-of-freedom mass-spring-damper system model. A drop hammer with a mass of 400 kg and a striking head radius of 90 mm was used and dropped on the beams from four different heights. Three series of reinforced concrete beams were tested, each having a cross-section of 150 x 250 mm and a length of 1700 mm. Various longitudinal reinforcement ratios were used: 1.26% for the series S1616 beams, 2.46% for the series SI322 beams, and 4.28% for the series S2222 beams. The drop heights were 0.15 m, 0.3 m, 0.6 m, and 1.2 m for the series S1616 beam specimens and were 0.3 m, 0.6 m, 1.2 m, and 2.4 m for the series SI322 and series S2222 beams. The concrete compressive strength was 42 MPa and the yield strength of reinforcing bars was 397 MPa for steel in series S1322, 426 MPa for steel in series S1616, and 418 MPa for steel in series S2222. A sufficient number of stirrups were installed at a spacing of 75 mm with a yield strength of 295 MPa. The variables investigated in this study were the influence of drop height and the degree to which longitudinal steel reinforcement contributes to the response of RC beams. A number of the conclusions and observations of this study are outlined below (Fujikake et al., 2009):

- When comparing the three series at the same drop height, series SI616 beams showed an increase by approximately 40% compared to that of series SI322 and series S2222 beams in terms of duration of impact load, the maximum mid-span deflection, and the time taken for the maximum mid-span deflection, as seen in Figure 2-19.
- At the same drop height, the maximum impact load and the impulse were found to be approximately the same when comparing the three series with each other, as shown in Figure 2-20.
- An increase in the steel reinforcement ratio caused a reduction in maximum mid-span deflection at each drop height, as shown in Figure 2-19(b).
- The results of the analytical model showed good agreement with the experimental results.

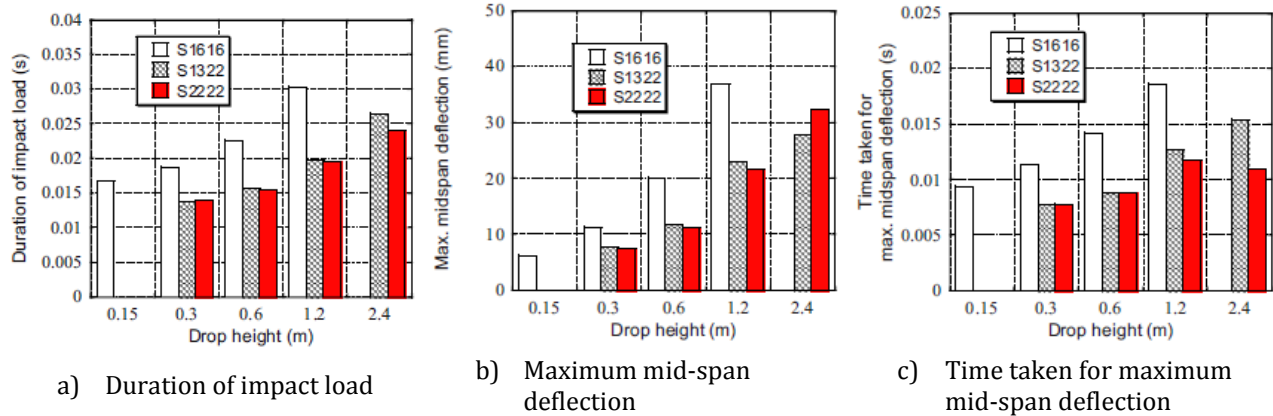


Figure 2-19 Results of max. mid-span deflection and duration of impact load and max. mid-span deflection
[Adapted from Fujikake et al. (2009)]

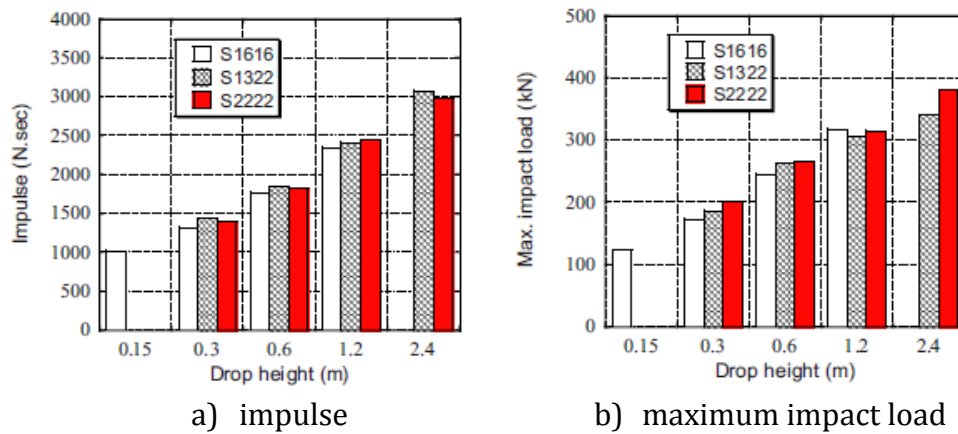


Figure 2-20 Results of maximum impact load and impulse
[Adapted from Fujikake et al. (2009)]

2.4.2 Steel Fibre-Reinforced Concrete (SFRC) Beams

2.4.2.1 A Study by Magnusson et al. (2010) on the Behaviour of Beams Subjected to Air Blast Loading

In this study by Magnusson et al. (2010), experiments were conducted to examine the structural behaviour of normal-strength concrete (NSC) and high-strength concrete (HSC) beams under air blast loading. Forty-nine beams were tested statically and dynamically, with thirty-eight beams subjected to air blast loading and the remaining eleven beams subjected to four-point quasi-static bending. All beams had cross-sections of 300 x 160 mm and lengths of 1720 mm where the clear span was 1500 mm for both static and dynamic tests. Concrete with compressive strengths of 100, 140, 150, and 200 MPa for HSC and 40 MPa for NSC were used. All beams were designed with varied longitudinal reinforcement ratios having yield strengths of 550-600 MPa, and were designed with stirrups that were arranged at a spacing of 200 mm. Some of the beams had steel fibres at volume fractions of either 1.0% or 2.4%. The straight steel fibres had length of 12.5 mm and a diameter of 0.4

mm. A number of the conclusions and observations of the study are outlined below (Magnusson et al., 2010):

- Failure mode for some of the beams changed from flexural under static loads to shear under air blast loading. It was observed that beams with a higher stiffness tended to fail in shear while beams with a lower stiffness failed in flexure.
- For the static tests, the failure mode was concrete crushing in the compression zone and yielding of the flexural reinforcement. Beams with steel fibres had tensile failure of the longitudinal steel bars.
- For the air blast tests, beams containing a high ratio of flexural reinforcement showed shear failure, whereas beams with a low ratio of longitudinal reinforcement exhibited flexural failure. In general, the addition of steel fibres transformed the beam failure mode into a flexure under dynamic loading; it was determined that adding steel fibres can enhance the shear strength and ductility of beams (e.g. compare response of beams B200 and B200F beams in Figure 2-21).
- For the HSC beams, it was found that the ultimate dynamic load capacity was larger compared to the ultimate static load capacity (see Figure 2-22). This was not the case for the NSC beam.

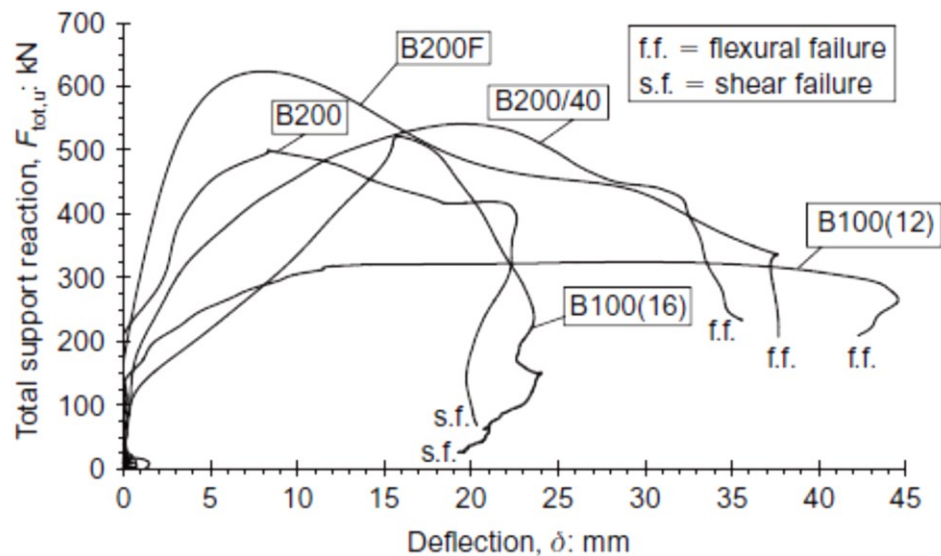


Figure 2-21 Support reaction- deflection curve for beams subjected to air blast loading
[Adapted from Magnusson et al. (2010)]

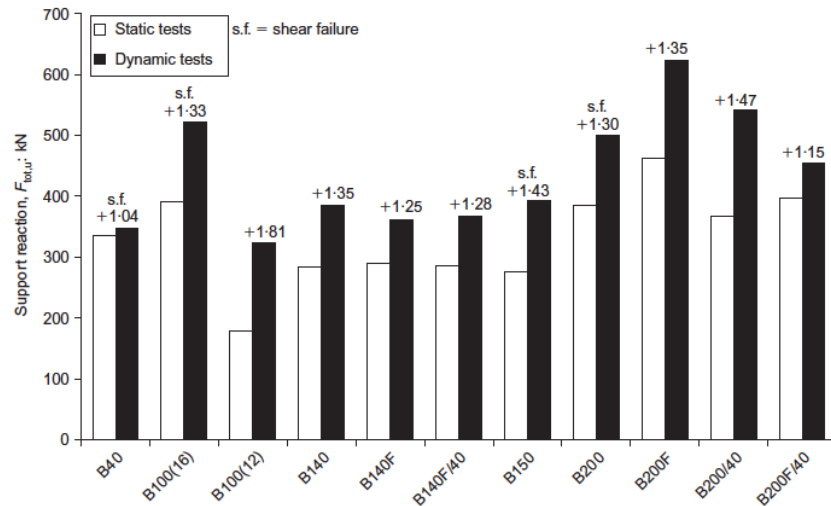


Figure 2-22 comparison between ultimate static load and ultimate dynamic load
[Adapted from Magnusson et al. (2010)]

2.4.2.2 A Study by Burrell (2012) on the Effects of Shock Wave Loading on Columns

Burrell (2012) investigated the effect of steel fibre reinforced concrete (SFRC) and ultra-high performance concrete (UHPC) on the behaviour of reinforced concrete columns subjected to shock wave loading. Thirteen half-scale columns were tested, including eight specimens built with normal-strength SFRC and five specimens built with UHPC. All columns had a cross-section of 152 x 152 mm and length of 2468 mm and were reinforced with four 10M longitudinal reinforcement bars and 6 mm ties arranged at a spacing of 75 mm or 38 mm. The concrete strength varied between 40 and 60 MPa for normal-strength and between 144 and 165 MPa for the UHPC. A variety of steel fibre contents was used (0 to 1.5% for SFRC and 2.0 to 6.0% for UHPC). The main parameters investigated were the effects of concrete strength, the influence of the type and content of steel fibres, and the spacing of transverse reinforcement. A few of the findings are outlined below (Burrell, 2012):

- Steel fibres showed an ability to reduce the maximum and residual displacements during each shock-wave when compared to companion columns without steel fibres. In addition, steel fibres prevented spalling and crushing of concrete when compared to the columns without fibres.
- Columns with closer spacing of transverse reinforcement decreased the displacements in comparison to columns with larger spacing. In addition, closer spacing of transverse reinforcement prevented the buckling of compression reinforcement while steel fibres were not able to prevent buckling.
- Concrete strength changed the mode of failure into rupture of the tension reinforcement when UHPC was used.

2.5 Summary

This chapter presented a literature review related to blast loads parameters and the effects of high-strain rate loading on material properties. It also reviewed previous studies on high-strength concrete and high-strength steel fibre reinforced concrete beams under static and dynamic loading. Some of the conclusions and observations from the literature review are outlined as follows:

- The primary blast loading parameters include the blast overpressure, duration and impulse. Shockwaves generated by far-field blasts can be safely and accurately simulated using a shock-tube.
- Blast loads result in the application of dynamic loads at high-strain rates. The properties of concrete, SFRC and steel reinforcement are affected the rate of loading, with the apparent increase in strength taken into account using dynamic increase factors (DIF). For design of blast-resistant structures, the UFC 3-340-02 documentation proposes conservative DIF values which are independent of strain rate. Researchers have also proposed strain-rate dependent DIF models for concrete, SFRC and steel reinforcement.
- For concrete in compression some models indicate reduced DIF as concrete strength is increased (e.g. CEB model), while other models propose DIF equations which are independent of concrete strength (e.g. Saatcioglu et al. model). For steel reinforcement, research indicates that the increase in ultimate stress under dynamic loading is relatively lower when compared to the increase in yield stress. Furthermore, the DIF has been found to be lower for higher grade (strength) steel reinforcement.
- The behaviour of SFRC at high strain-rates is not well understood and data in the literature is conflicting on the effectiveness of steel fibers at high strain-rates. Researchers have proposed DIF models for SFRC under dynamic compression, however there is a lack of models for the DIF of SFRC under dynamic tension.
- Previous research on HSC beams under flexural static loading shows that increasing the reinforcement ratio improves load resistance, although it reduces maximum displacement and ductility. The presence of stirrups increases the shear capacity of HSC beams and improves beam ductility. Concrete strength was also found to affect beam ductility (with ductility generally reducing as the concrete strength is increased).

- Research shows that the provision of steel fibres enhances the response of high-strength concrete beams by increasing shear capacity and promoting flexural failure and ductility. The shear span-to-depth ratio, reinforcement ratio, steel fiber content and fibre type are important parameters that affect the shear capacity, load resistance and ductility of HSFRC beams tested under flexural loading.
- Limited research exists on the response of reinforced concrete beams under blast loads. Previous research on beams tested under impact loads shows that increasing the reinforcement ratio improves beam response by reducing maximum mid-span displacements.
- A limited number of studies have investigated the response of SFRC structural members under blast loading. Research on columns indicates that provision of fibers improves the blast response of reinforced concrete flexural members by reducing maximum displacements and improving damage tolerance. Research on SFRC beams indicates that failure mode can change from flexure under static loading to shear under dynamic loads, particularly in the case of beams with larger reinforcement ratio.

Chapter 3: Experimental Program

3.1 Chapter Overview

This chapter describes an experimental program which involves the design, construction and testing of twenty reinforced concrete beams built with high-strength concrete and steel fibres. The testing program includes companion beam specimens tested under simulated blast loading and quasi-static four-point bending. Parameters considered in this study include the effect of concrete strength, longitudinal reinforcement ratio, volumetric ratio and type of steel fibres, and presence of transverse reinforcement on the flexural and shear response of beams tested under static and dynamic loading. This chapter includes description of the test specimens, material parameters, specimen preparation, and the experimental setup with its related equipment and testing procedure.

3.2 Specimen Specifications

A total of twenty reinforced concrete beams were designed for this research. Eleven of the beams were tested under simulated blast loading using the University of Ottawa shock-tube, with the remaining companion set of nine beams tested under quasi-static four-point bending. The majority of the beams were cast using high-strength concrete (HSC) or high-strength steel fibre-reinforced concrete (HSFRC), with one beam cast with normal-strength self-consolidating concrete (SCC). The specimen details are summarized in Figure 3-1. All specimens had a length of 2440 mm, a width of 125 mm, a depth of 250 mm, and were tested over a simply supported span of 2232 mm, with shear spans of 741 mm. Each beam was reinforced with two bottom steel bars which consisted of either #4 (US size), 15M or 20M (Canadian size) reinforcement. The bars had either 90° hooks (for #4 and 15M) or 180° hooks (for 20M) to ensure proper development of the tension steel, and were spaced at approximately 70 mm from each other. Non-deformed 6 mm diameter steel bars were used as shear reinforcement in five specimens, with stirrups arranged at a spacing of 100 mm in the shear spans (two 6 mm size top bars were also installed in the shear spans in all beams to facilitate construction). Clear cover was restricted to 35 mm on the tension side for beams with stirrups and 41 mm for beams without shear reinforcement. Two types of hooked-end steel fibres were considered in this study (ZP or 5D), at fibre content of 1.0% (78 kg/m³) or 0.5% (39 kg/m³), by volume of concrete. As shown in Table 3-1, the specimens can be divided into three categories based on longitudinal reinforcement ratio (rebar size): series #4, series 15M and series 20M. Table 3-1 provides the details of the beam design within each series, including specimen ID, concrete mix type, fibre volumetric ratio (v_f), fibre type and steel reinforcement properties. The specimen nomenclature follows this simple logic:

CONCRETE TYPE – FIBRE % and TYPE – STEEL REBAR SIZE – STIRRUP

For example, HSC-F1(ZP)-#4-0 would be the specimen ID for a beam cast with high strength concrete containing "ZP" steel fibres at a volumetric fibre content of 1.0%, with 2-#4 reinforcing bars, and no stirrups (the "0" in the specimen ID designates the lack of stirrups, whereas the "S" indicates provision of stirrups at a spacing of 100 mm).

Table 3-1 Specimen categories and specifications

Series	Specimen ID	Concrete Type [f'_c (MPa)]	Steel fibre properties		Steel reinforcement properties		Test type
			Type	Content v_f (%)	Stirrups Spacing (mm)	Flexural Steel [ρ (%)]	
#4	NSC-F0-#4-S	SCC [58]	None	0.0%	100	2 - #4 [1.02]	S/D
	HSC-F0-#4-S	HSC					
	HSC-F1(ZP)-#4-0	[108]	ZP	1.0%	0		
15M	HSC-F0-15M-S	HSC [107]	None	0.0%	100	2 - 15M [1.59]	
	HSC-F1(ZP)-15M-0		ZP	1.0%	0		
20M	HSC-F0-20M-0	HSC [104]	None	0.0%	0	2 - 20M [2.41]	
	HSC-F0-20M-S				100		
	HSC-F1(ZP)-20M-0		ZP	1.0%	0		
	HSC-F1(ZP)-20M-S				100		
	HSC-F0.5(ZP)-20M-S		0.5%				
	HSC-F0.5(5D)-20M-S			5D			

*S = Static Test

*D = Dynamic Test

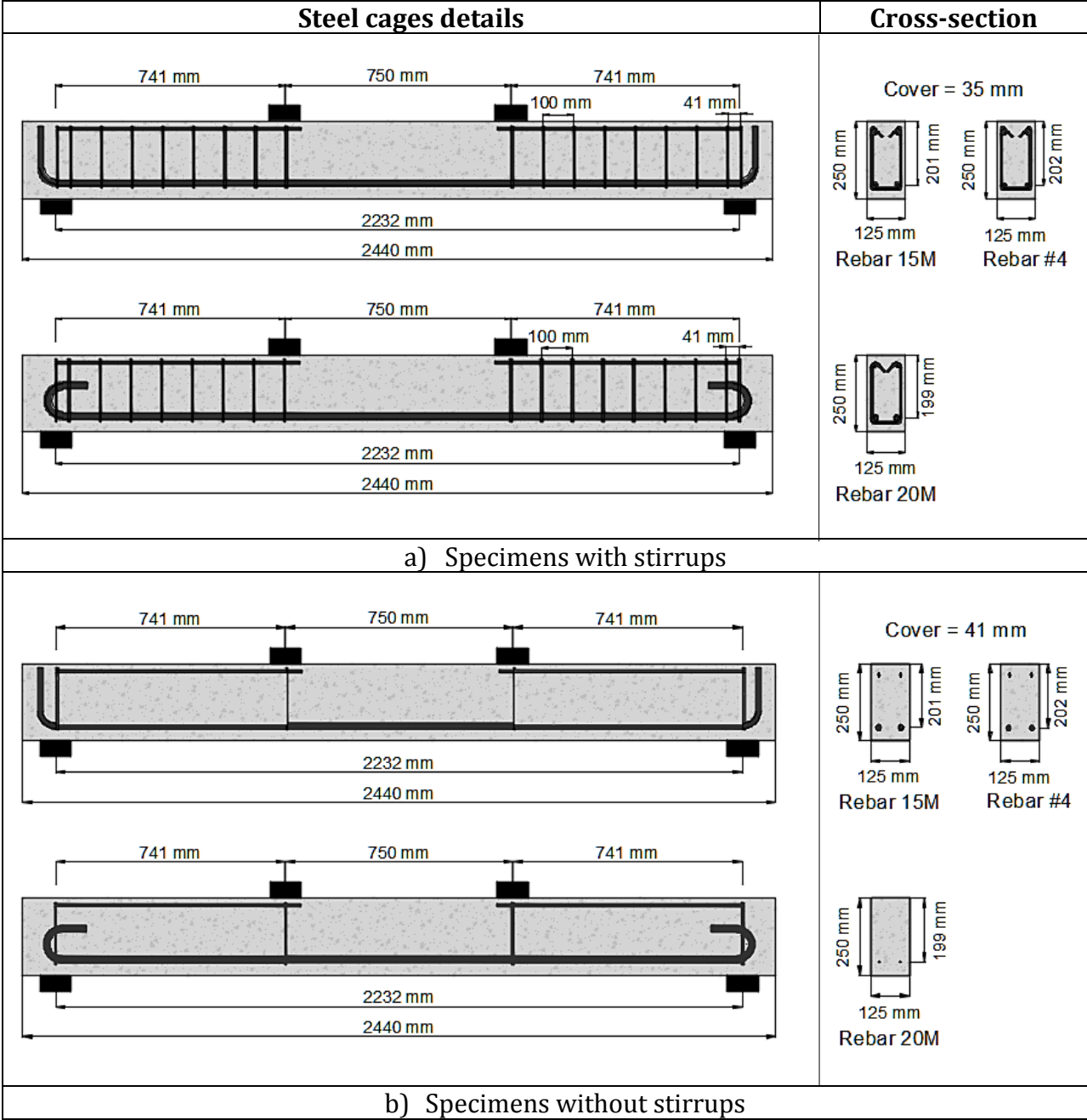


Figure 3-1 Specimen cross-section dimensions and steel reinforcement details

3.2.1 Series #4

The beams in this series were reinforced with two US size #4 reinforcing bars. Parameters investigated in this series include the effects of normal vs. high strength concrete and the effect of steel fibres or transverse reinforcement. A total of six specimens are included in this series, including three specimens tested under blast loads and three companion specimens tested under static loads, including beams NSC-F0-#4-S, HSC-F0-#4-S and HSC-F1(ZP)-#4-0. The fibre type used was ZP305 with a volumetric fraction of 1.0%.

3.2.2 Series 15M

This series consists of four high-strength reinforced concrete beams with 15M reinforcement and either structural steel fibres or transverse reinforcement. Two specimens were tested under blast loads and the other two specimens were tested under static loads, including the following specimens: HSC-F0-15M-S and HSC-F1(ZP)-15M-0. The fibre type used was ZP305 with a fraction volume of 1.0%.

3.2.3 Series 20M

Ten high-strength reinforced concrete beams built with 20M reinforcement were included in this series. The series included specimens with either stirrups, steel fibres, or both. The beams can be divided into two groups, with six beams tested in the shock-tube, and four beams tested under quasi-static loading. The series includes the following specimens: HSC-F0-20M-0, HSC-F0-20M-S, HSC-F1(ZP)-20M-0, HSC-F1(ZP)-20M-S, HSC-F0.5(ZP)-20M-S and HSC-F0.5(5D)-20M-S. The fibre type used was ZP305 (ZP) and 5D with a volumetric ratio varying between 0.5% and 1.0% of the total volume. The two specimens with 0.5% of steel fibres were only tested under blast loads.

3.3 Materials

The following sub-sections describe the various materials used for this research. This includes the different concrete mixes, steel reinforcement bar types and steel fibre types.

3.3.1 Concrete parameters

Two different types of concrete were used for this research: *self-consolidating concrete* (SCC) by KING Packaged Materials Company, and *high strength concrete* (HSC) mixed in the University of Ottawa Structures Laboratory.

Two specimens were cast using self-consolidating concrete in order to observe the effects of increasing the concrete strength. Specified for a nominal compressive strength of 40 MPa, the pre-packaged mix "KING SCC" provided high workability as seen in Figure 3-2. Properties provided by the manufacturer for this mix are listed in Table 3-2. The maximum aggregate size was specified as 10 mm, while admixtures are incorporated in the mix in the

form of dry powder including an air-entrained admixture, superplasticizer and a viscosity modifying admixture.

The remaining specimens were cast using high-strength concrete mix made in the University of Ottawa Structures Laboratory, and capable of reaching high compressive strengths of over 100 MPa. This mix was composed of cementitious materials, sand, and coarse aggregate ($\frac{1}{2}$ " and $\frac{3}{4}$ "). The cementitious materials included Portland cement, slag, and silica fume as seen in Figure 3-3. Liquid admixtures were incorporated in the mix and included a super-plasticizer (MasterGlenium 7500) and retarder (MasterSet R 100). Steel fibres were added in a 0.5-1.0% volumetric ratio, and the water-to-cement ratio was 0.3. An example of the content of the HSC mix is given in Table 3-3.

Table 3-2 KING SCC properties

Solid Component	Content (kg/m³)
HSF Cement	500
Coarse Aggregate	765
Fine Aggregate	915
Mass Density	2300
Other Properties	Properties
Ratio Fine/Total Aggregate	0.55
Air Content (%)	7
Water-Cement Ratio	0.42

Table 3-3 High strength concrete mix properties

Solid Component	Content (kg/m³)	
	HSC	HSFRC
Cement	373	373
Slag	164	164
Silica Fume	48	48
Sand	738.89	738.89
Fine Aggregate 1/2"	560	560
Coarse Aggregate 3/4"	560	560
Fibres (1.0%)	-	78
Fibres (0.5%)	-	39
Liquid Component	Content (kg/m³)	
Water	157.22	157.22
Super-plasticizer	11.11	13.89
Retarder	2.78	2.78



Figure 3-2 KING SCC



Figure 3-3 High strength concrete materials

3.3.2 Steel Reinforcement Parameters

Four different sizes of steel reinforcement were used in the beams considered for this research. Non-deformed 6.3 mm wire was used as a transverse and compression reinforcement in shear spans; this wire had a diameter of $d_b = 6.35$ mm and a cross section of $A_b = 32$ mm². The three remaining deformed steel bars were used for longitudinal reinforcement, including #4 U.S. size, 15M and 20M Canadian size. Rebar #4 had a diameter of $d_b = 12.70$ mm and a cross section of $A_b = 129$ mm². Similarly, rebar 15M had a diameter of $d_b = 16.0$ mm and a cross section of $A_b = 200$ mm² while rebar 20M had a diameter of $d_b = 19.5$ mm and a cross section of $A_b = 300$ mm².

Three steel coupons were taken from each steel rebar type and tested at the University of Ottawa Structures Laboratory with a GALDABINI SUN 60 Universal Floor Standing Testing Machine as shown in Figure 3-4. The test was conducted under axial tension at a rate of 7.5 mm/min, while displacements were measured using an axial extensometer (model 3542 *Epsilon Technology Corporation*) with a gauge length of 50 mm. The average properties of the various steel reinforcing bars are summarized in Table 3-4, including yield strength and strain, strength and strain at ultimate and the rupture strain. Figure 3-5 and Figure 3-6 show the stress-strain relationships for each bar type.

The 6.3 mm wire had an average yield strength of 540 MPa at a strain of 0.0024 mm/mm, ultimate strength of 610 MPa at a strain of 0.051 mm/mm, and rupture strain of 0.051 mm/mm (it is noted two batches were used for this steel, which explains the difference in the coupon results shown in Figure 3-5(a)). The #4 rebar had a yield strength of 449 MPa at a strain of 0.0015 mm/mm, ultimate strength of 680 MPa at a strain of 0.079 mm/mm and rupture strain of 0.108 mm/mm. The 15M reinforcement was found to have a yield strength of 471 MPa at a strain of 0.0027 mm/mm, ultimate strength of 581 MPa at a strain of 0.153 mm/mm and rupture strain of 0.319 mm/mm. Finally Rebar 20M had a yield strength of 460 MPa at a strain of 0.0025 mm/mm, ultimate strength of 600 MPa at a strain of 0.140 mm/mm and rupture strain of 0.210 mm/mm.

Figure 3-7 compares the stress-strain relationships of the different bars used in this study. This graph illustrates that the 15M bar had higher ductility than the other steel bars, whereas the #4 bar had the highest ultimate strength of the four bars.

Table 3-4 Steel reinforcement mechanical properties

ID	Steel Reinforcement	Bar Diameter d_b (mm)	Bar Area, A_b (mm ²)	Yield		Ultimate		Rupture
				ϵ_y	f_y (MPa)	ϵ_u	f_u (MPa)	ϵ_r
6.3NS	Non-deformed	6.35	32	0.0024	540	0.051	610	0.051
#4	Regular	12.7	129	0.0015	449	0.079	680	0.108
15M	Regular	16.0	200	0.0027	471	0.153	581	0.319
20M	Regular	19.5	300	0.0025	460	0.140	600	0.210

* ϵ_r = Strain at rupture



Figure 3-4 GALDABINI Universal Floor Standing Testing Machine and Axial Extensometer; testing steel rebar

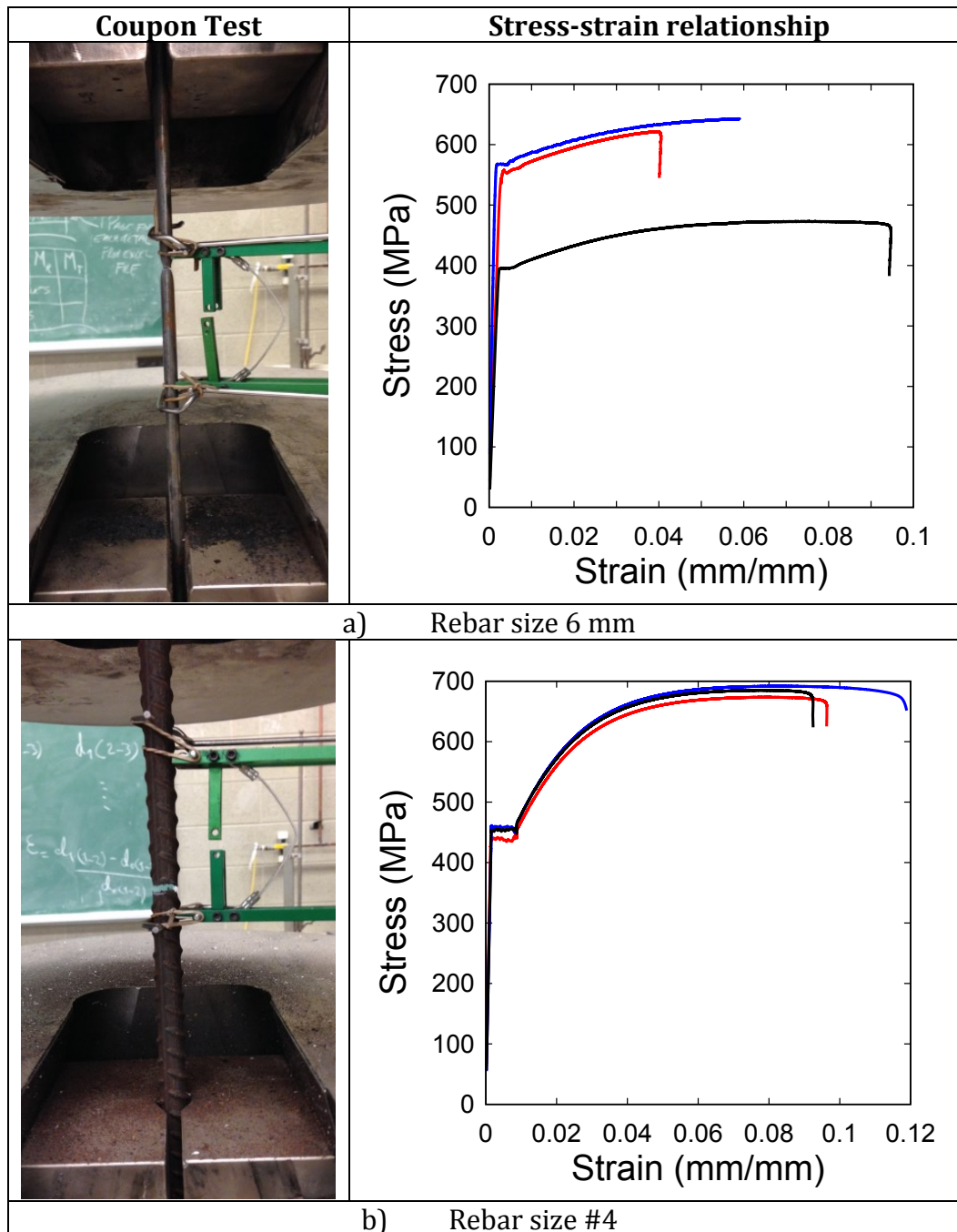


Figure 3-5 Types of steel reinforcements stress-strain relationship and photographs

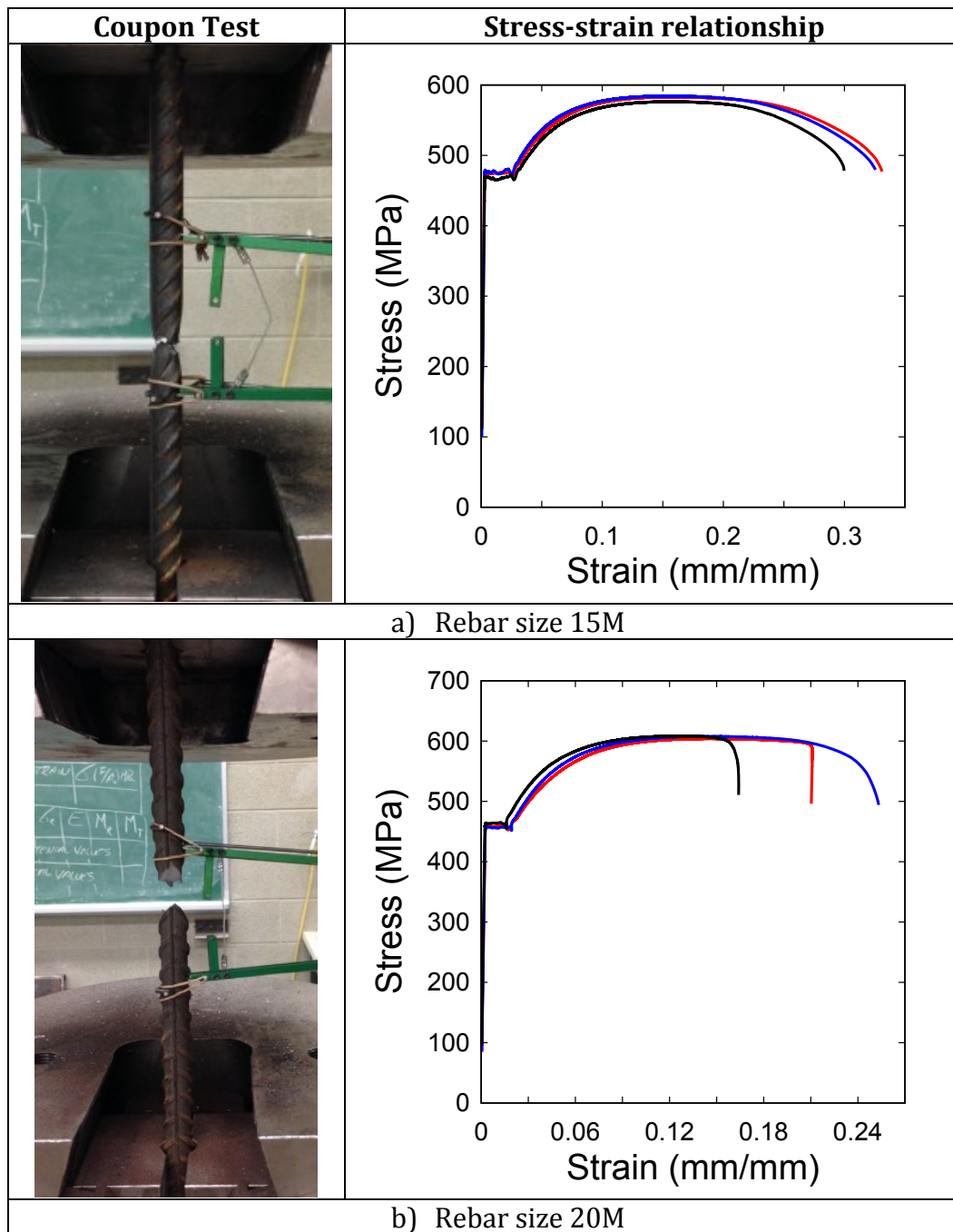


Figure 3-6 Types of steel reinforcements stress-strain relationship and photographs

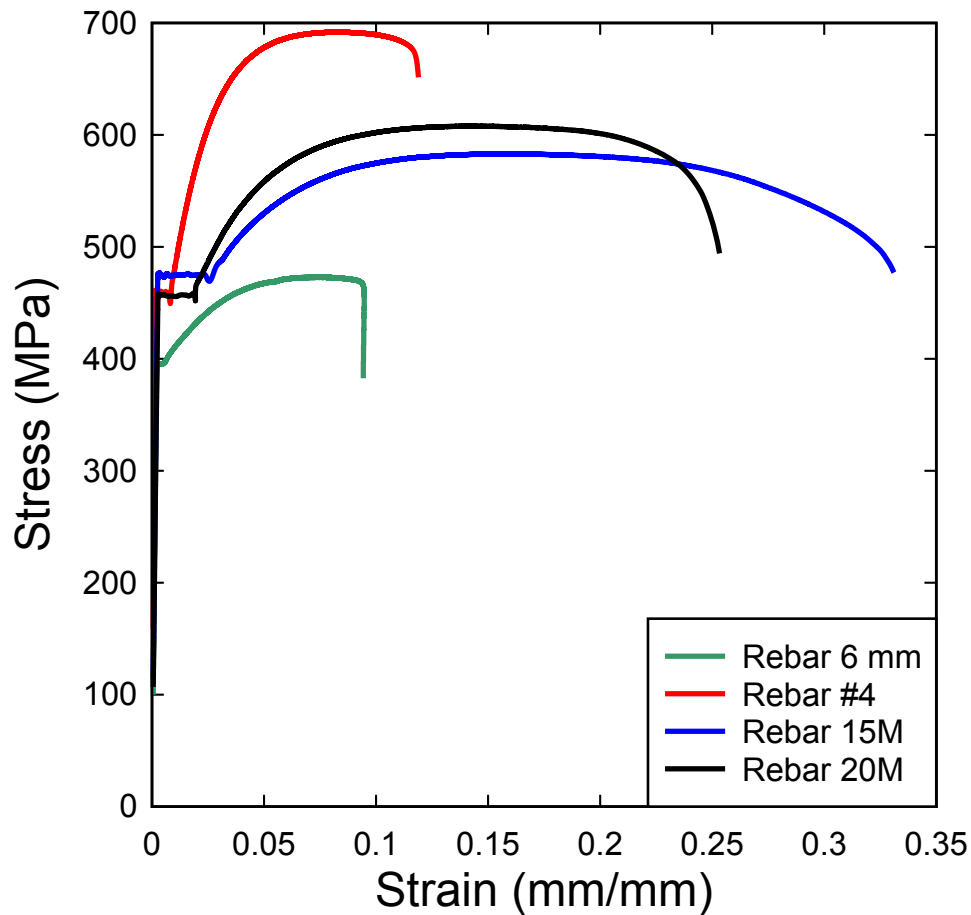


Figure 3-7 Steel reinforcement stress-strain relationship

3.3.3 Steel Fibres Parameters

Two types of steel fibres were considered for this experimental program. The summary of the steel fibre properties can be found in Table 3-5, followed by photographs of the fibres in Figure 3-8. Ten of the HSFRC specimens were cast with high-strength concrete incorporating *BAKAERT Dramix® ZP305* hooked-end steel wires (labeled at "ZP" in the specimen ID). These fibres have a length of 30 mm, an aspect ratio of 55 and a tensile strength estimated at 1350 MPa. The brass-colored *BAKAERT Dramix® 5D* fibres were used in only one specimen; these fibres have a longer length of 60 mm, higher aspect ratio of 65, a greater tensile strength of 2300 MPa and a double-hook configuration for improved anchorage as shown in Figure 3-8(b). The ZP fibres were added at a ratio of 1.0% or 0.5% by volume of concrete, while the 5D fibres were added at a ratio of 0.5%.

Table 3-5 Properties of Steel fibres

Fibre ID	Fibre Name	Length l_f (mm)	Diameter d_f (mm)	Aspect Ratio (mm/mm)	Tensile Strength (MPa)
ZP	ZP 305	30	0.55	55	1350
5D	5D	60	0.92	65	2300

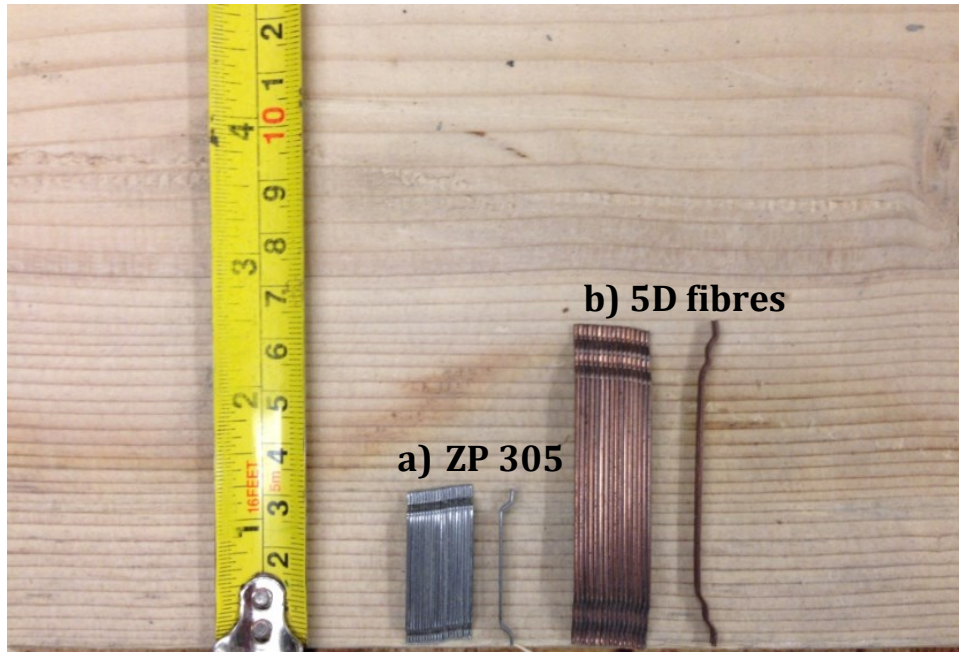


Figure 3-8 Photographs of Steel fibres

3.4 Construction of Test Specimens

The construction of the test specimens was completed at the University of Ottawa Structures Laboratory. Steps included preparing the formwork, bending reinforcement steel bars, applying strain gauges, casting, and curing the specimens.

3.4.1 Preparation and Casting

Wood formworks were constructed from 19 mm thick, 2440 x 2440 mm plywood sheets resting on 38 x 64 mm wood studs for solidification. Three rectangular units containing two boxes which allowed for casting of two specimens each were built over the plywood base as shown in Figure 3-9. Braces were fixed at the edges of the formwork for support. Before casting, two sheets of 19 mm thick plywood measuring 250 x 2440 mm were attached on the north and south side of the formwork to prevent the concrete from spilling out of the units. The bottom and sides of the inner formwork walls were brushed with motor oil to ease the removal of the specimens and allow the formwork to be reused. Plastic chairs were distributed on the tension side of the specimens to maintain a clear cover of 35 mm.

Two types of steel cages were made for this project, either with or without transverse shear reinforcement. The 6.3 mm wire was used to fabricate the U-shaped stirrups and for the top reinforcement bars in the shear spans. For steel cages with shear reinforcement, the transverse reinforcement was installed at a spacing of 100 mm in all cases. The 20M steel cage was built using two 20M rebars which were hooked with 180 degree at both ends and nine stirrups with two top reinforcements in both shear spans. The #4 and 15M steel cages were made using two longitudinal rebars with 90 degree hooks at both ends and nine stirrups with two top reinforcements in both shear spans. For steel cages with no shear reinforcement, stirrups were only provided at the supports and point load locations in order to allow for the placement of the top 6.3 mm bars for casting. Figure 3-10 shows the design of each steel cage and includes photographs of typical cages after construction. One longitudinal steel rebar in each cage was smoothed at mid-span using a hand held belt-grinder in order to install strain gauge. Afterwards, the steel cage was placed in the formworks and attached to the plastic chairs with wires to restrict movement during casting.

Concrete was prepared at the University of Ottawa Structures Laboratory using a 420 volt electric multiflow pan mixer shown in Figure 3-11 and the concrete materials outlined in section 3.3.1. After pouring and leveling the beams, wet burlap sheets and plastic sheets were used to cure the beams for seven days. When this period had finished, the curing sheets were removed and the specimens were air-cured until testing occurred.

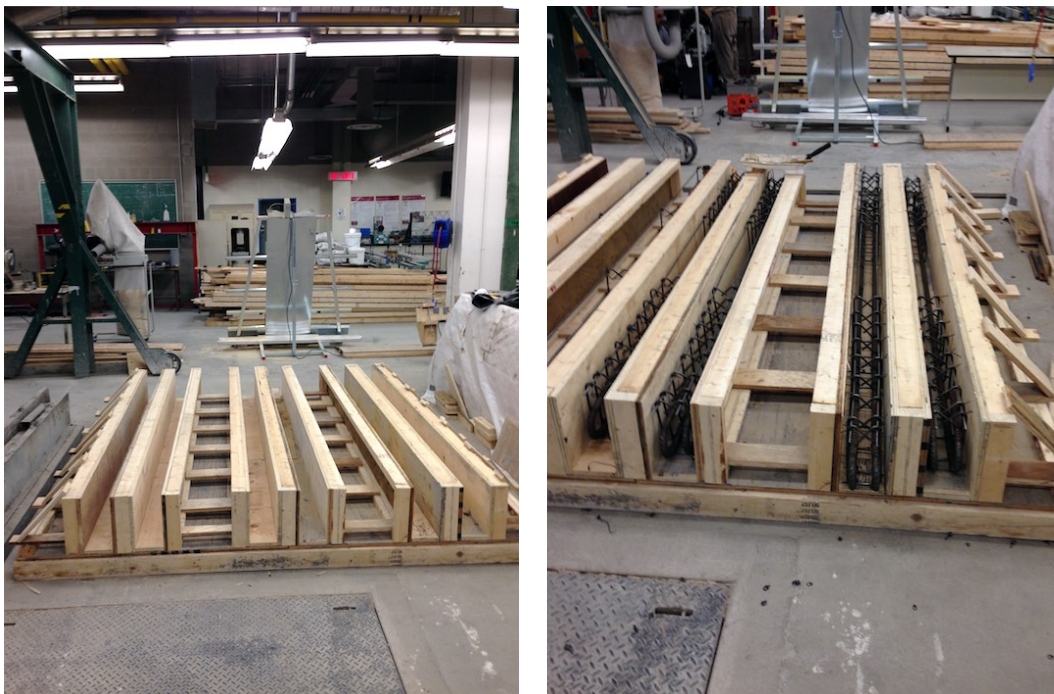


Figure 3-9 Preparation of wood formworks for specimens

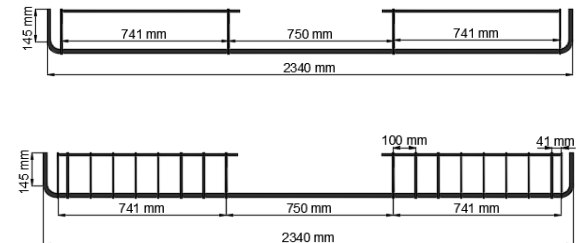
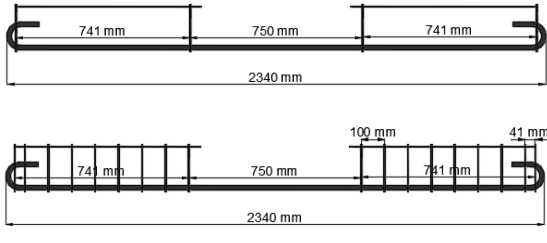
Steel cages with 90 degree hooks	Steel cages with 180 degree hooks
 #4 steel bars 15M steel bars	 20M steel bars
a) Rebar #4 and 15M with/without transverse steel bars	b) Rebar 20M with/without transverse steel bars

Figure 3-10 Preparation of steel cages



Figure 3-11 Concrete pan mixer and beams casting

3.4.2 Fresh State Properties

Slump tests were conducted in accordance with the ASTM C1611 “Standard Test Method for Slump Flow of Self-Consolidating Concrete” for KING-SCC and the ASTM C143 “Standard Test Method for Slump of Hydraulic-Cement Concrete” for the HSC mix. The concrete mix was placed in a metal mold in the shape of a cone and released on a flat, unconfined surface. This allowed for the measurement of the flowability and workability of each concrete mix. The results have been summarized in Table 3-6 and photographs are

included in Figure 3-12. A higher slump means the mix has a greater flow potential and should require less vibration during placement.

Table 3-6 Concrete fresh state properties

Specimen ID	Concrete Mix	Fibre Type	Fibre Ratio	Slump (mm)
NSC-F0-#4-S	SCC	None	0.0%	700
HSC-F0-#4-S	HSC	ZP		250
HSC-F1(ZP)-#4-0		None	1.0%	130
HSC-F0-15M-S			0.0%	230
HSC-F1(ZP)-15M-0		ZP	1.0%	220
HSC-F0-20M-0		None	0.0%	240
HSC-F0-20M-S				240
HSC-F1(ZP)-20M-0		ZP	1.0%	230
HSC-F1(ZP)-20M-S				200
HSC-F0.5 (ZP)-20M-0		5D	0.5%	220
HSC-F0.5 (5D)-20M-0				240

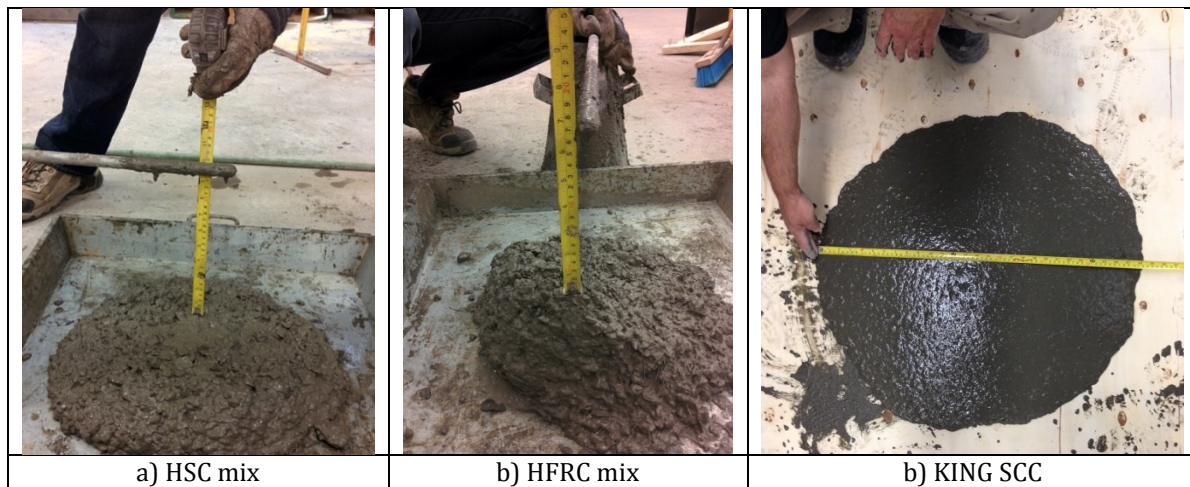


Figure 3-12 Typical concrete slump test results

3.4.3 Hardened State Properties

After finishing casting the beams, concrete compression strength was measured in three periods: after 7 days, 14 days, and 28 days (test day). Furthermore, the flexure toughness of each mix was investigated after the concrete mix hardened. This section summarizes the results from these tests.

3.4.3.1 Compressive Cylinder Test

Six cylindrical specimens with 100 mm diameter and 200 mm height were cast from each batch for measuring the compressive strength and stress-strain relationship concrete

used in the beams. The 1000 KN *PILOT Control System* cylinder testing machine was connected to a StrainSmart data acquisition to record the force applied on the specimens as shown in Figure 3-13. An Extensometer from *Controls Group* with a gauge length of 140 mm was also attached to the cylinder and connected to the StrainSmart data acquisition system in order to measure axial strain. The average compressive strengths at 7 days, 14 days, and 28 days have been tabulated in Table 3-7, while samples of the stress-strain curves at 28 days are included in Figure 3-14. Sample pictures of failed cylinders can be seen in Figure 3-15. The stress-strain curves show that mixes with fibres have greater toughness than plain concrete and were capable to sustain greater axial strains prior to failure. The provision of fibres was also effective in preventing the brittle and explosive failures observed in the HSC samples.



Figure 3-13 Cylinder testing setup

Table 3-7 Concrete compressive strength summary

Series	Specimen ID	Concrete Type	Fibre Type	Fibre Ratio	Cylinder Test Compressive Strength (MPa)		
					7 days	14 days	28 days
#4	NSC-F0-#4-S	SCC	None	0.0%	43.78	55.97	58.23
	HSC-F0-#4-S				72.26	89.8	106.8
	HSC-F1(ZP)-#4-0				76.06	99.2	108.4
15M	HSC-F0-15M-S	HSC	None	0.0%	79.58	99.06	110.6
	HSC-F1(ZP)-15M-0		ZP	1.0%	75.61	87.85	105.5
20M	HSC-F0-20M-0		None	0.0%	53.15	92.93	110.2
	HSC-F0-20M-S				84.25	89.56	105.5
	HSC-F1(ZP)-20M-0		ZP	1.0%	66.01	89.66	104.8
	HSC-F1(ZP)-20M-S				77.65	87.51	110.4
	HSC-F0.5(ZP)-20M-S		5D	0.5%	69.21	94.21	101.4
	HSC-F0.5(5D)-20M-S				69.21	94.21	94.95

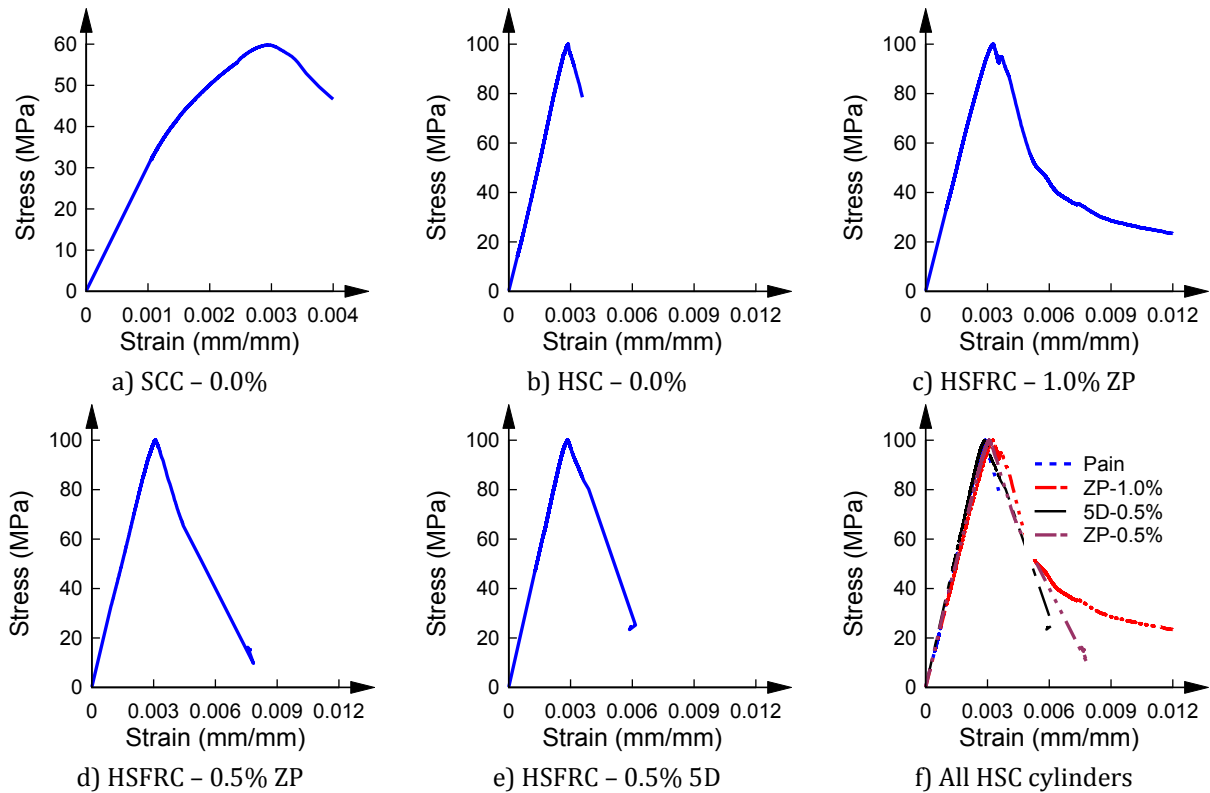


Figure 3-14 Concrete stress-strain relationship samples

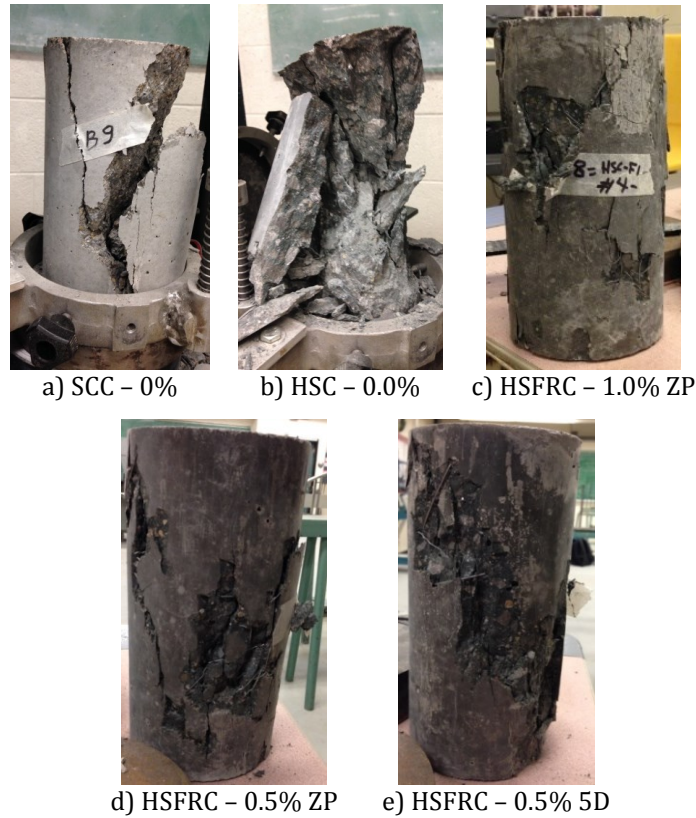


Figure 3-15 Sample concrete cylinder testing failures

3.4.3.2 Flexure Toughness

A small beam with dimensions of 100 mm x 100 mm x 400 mm was cast from each batch for measuring the modulus of rupture and flexural toughness in accordance with the ASTM C1609 standard. The prisms were tested using a GALDABINI SUN 60 Universal Floor Standing Testing Machine under four-point loading with a clear span of 300 mm, as shown in Figure 3-16. Displacements were captured using an LVDT and "Japanese Yoke" as specified in the ASTM C1609 standard. The resulting toughness properties obtained are summarized in Table 3-8.

Five types of concrete mix were chosen in order to investigate their flexure toughness: SCC-0%, HSC-0%, HSFRC-1.0% (ZP), HSFRC-0.5% (ZP), and HSFRC-0.5% (5D). The prism with steel fibre 5D exhibited ductility after peak load and highest peak load. Prisms without fibres resulted in a brittle tension rupture at failure, which confirms that the steel fibre play an important role in improving the flexural ductility of concrete. Photographs of the small beams at failure are included in Figure 3-17, and the load-deformation curves are shown in Figure 3-18.



Figure 3-16 Flexural beam testing setup

Table 3-8 Results from ASTM C1609 toughness tests

Concrete Mix	ASTM C1609										
	P_1	δ_1	P_p	δ_p	f_1	f_p	P_{600}	f_{600}	P_{150}	f_{150}	T_{150}
SCC-0%	-	-	23.4	0.10	-	7.02	-	-	-	-	-
HSC-0%	-	-	28.42	0.09	-	8.53	-	-	-	-	-
HSFRC-1.0%ZP	-	-	32.53	0.06	-	9.76	13.81	4.14	4.41	1.32	21.71
HSFRC-0.5%ZP	-	-	28.91	0.04	-	8.67	7.73	2.32	2.0	0.6	13.39
HSFRC-0.5%5D	20.5	0.056	34.25	1.01	6.15	10.28	26.78	8.03	27.74	8.32	59.91

L = Span Length (300 mm), ($L/600 = 0.5$, $L/150 = 2$) P_1 = First-Peak Load (kN) δ_1 = Net Deflection at First-Peak Load (mm) P_p = Peak Load (kN) δ_p = Net Deflection at Peak Load (mm) f_1 = First-Peak Strength (MPa) f_p = Peak Strength (MPa) P_{600} = Residual Load at net deflection of $L/600$ (kN) f_{600} = Residual Strength at net deflection of $L/600$ (MPa) P_{150} = Residual Load at net deflection of $L/150$ (kN) f_{150} = Residual Strength at net deflection of $L/150$ (MPa) T_{150} = Area under load vs. net deflection curve (0 to $L/150$), (kN*mm) FT = Flexural Toughness Factor = $(T_{150} * L) / (L/150 * b * d^2)$	
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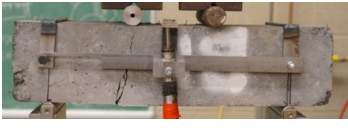
Beam specimens	Comments
	a) SCC-0% Failure: mid-span crack
	b) HSC-0% Failure: rupture at mid-span
	c) HSFRC-1.0% Failure: crack-failure near left side point load location
	d) HSFRC-0.5% (ZP) Failure: crack-failure at mid-span
	e) HSFRC-0.5% (5D) Failure: crack-failure within central flexural span

Figure 3-17 Photographs of the five prisms and their failure type

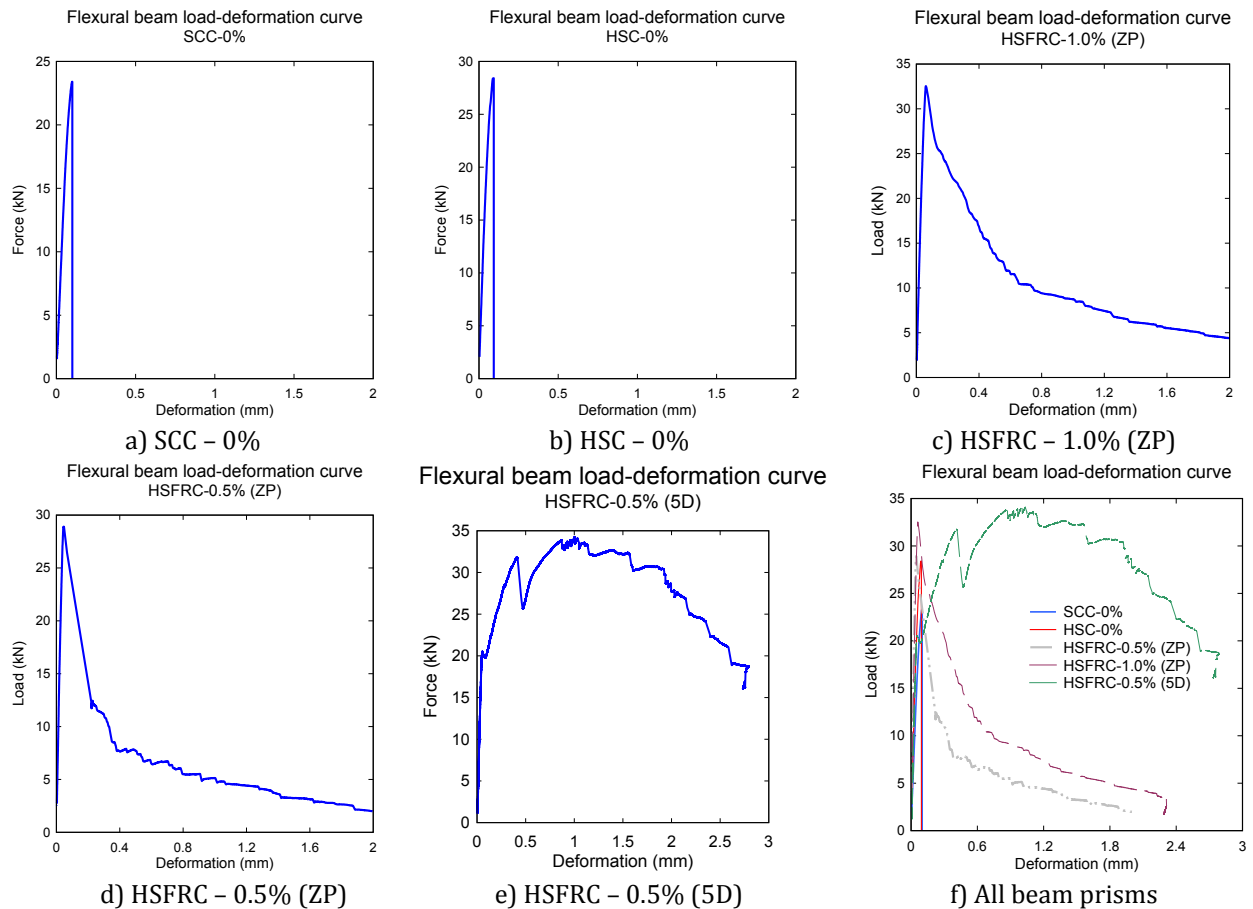


Figure 3-18 Flexural beam load-deflection curve samples

3.5 Experimental Setup

This section describes the experimental setup of the blast load tests and the quasi-static load tests. For the blast tests, the various elements composing the experimental setup are outlined, such as the shock-tube, the load transfer device, and the related instrumentation including high speed cameras, LVDT, strain gauges and data acquisition equipment, as well as a summary of the shockwave properties. For the static load testing, this section describes the experimental setup and testing procedure.

3.5.1 Blast Testing

3.5.1.1 Shock-tube

Blast testing was conducted using the University of Ottawa Shock-tube testing facility, previous research has shown the Shock-Tube is an installation capable of simulating shock waves generated by the detonation of high explosions (Lloyd et al. 2010). As shown in Figure 3-19 to Figure 3-21, the pneumatically-driven Shock-Tube has four main sections which include (1) a variable length driver, (2) a spool section, (3) an expansion section and (4) a rigid end test frame with a 2 m x 2 m square opening. The driver pressure controls the magnitude of the peak reflected pressure while the driver length controls the positive phase duration; in this research the driver length was kept constant at 2743 mm (9 ft).

The process of transferring the air pressure through Shock-Tube involves a few steps. First, air is pumped in both the driver and spool sections based on the blast load needed for testing. A double diaphragm firing system located between the driver and the expansion chamber section is responsible for releasing the compressed air which generates the shockwave. Two sets of aluminum sheets are placed in the diaphragm system, where one set of sheets blocks the driver opening and the other set blocks the expansion chamber opening, allowing pressure to build-up in the driver and spool sections. Once the desired pressure is reached, the air is drained from the spool section, resulting in the rupture of the aluminum foils and the rapid release of compressed air into the expansion chamber, where it travels along its length until it interacts with the specimen at the rigid end frame. In this study a lateral load transfer device (LTD) is used to collect and transfer the blast load as two concentric point loads onto the specimens. After finishing the test, the aluminum sheets are replaced with new ones in order to prepare the shock-tube for the subsequent shot. This process was used for each blast load, and the shockwave was gradually increased until the specimen failed.

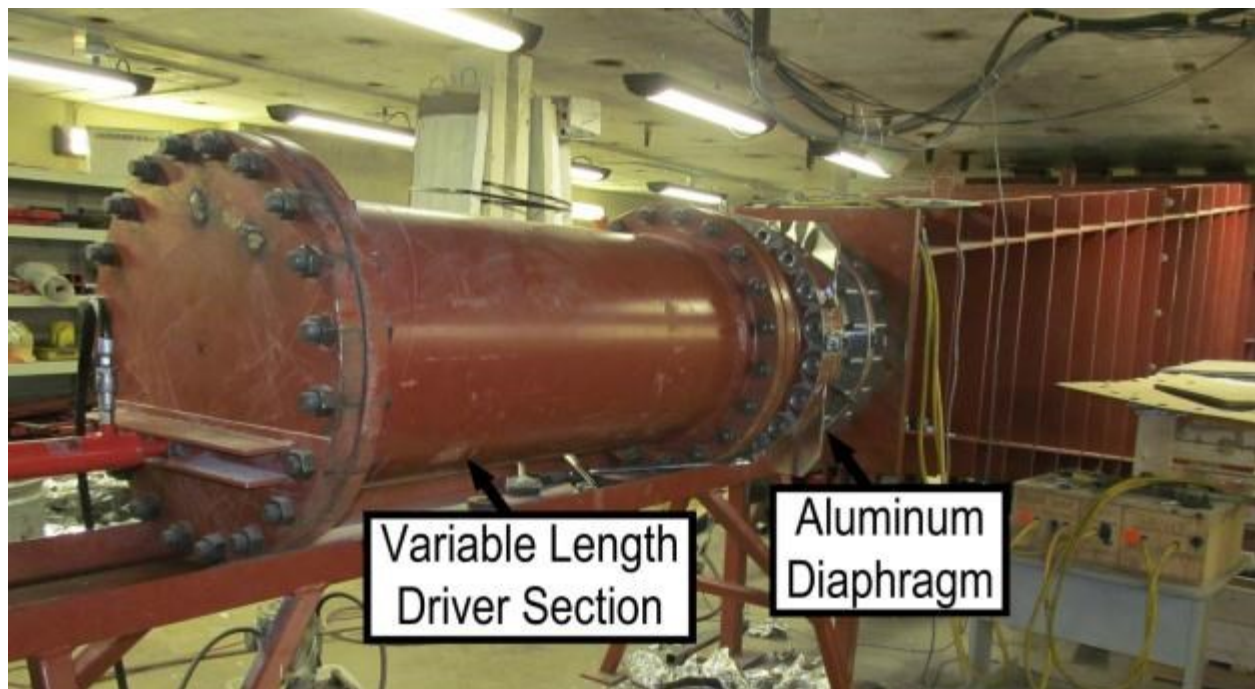


Figure 3-19 Shock-tube's driver section

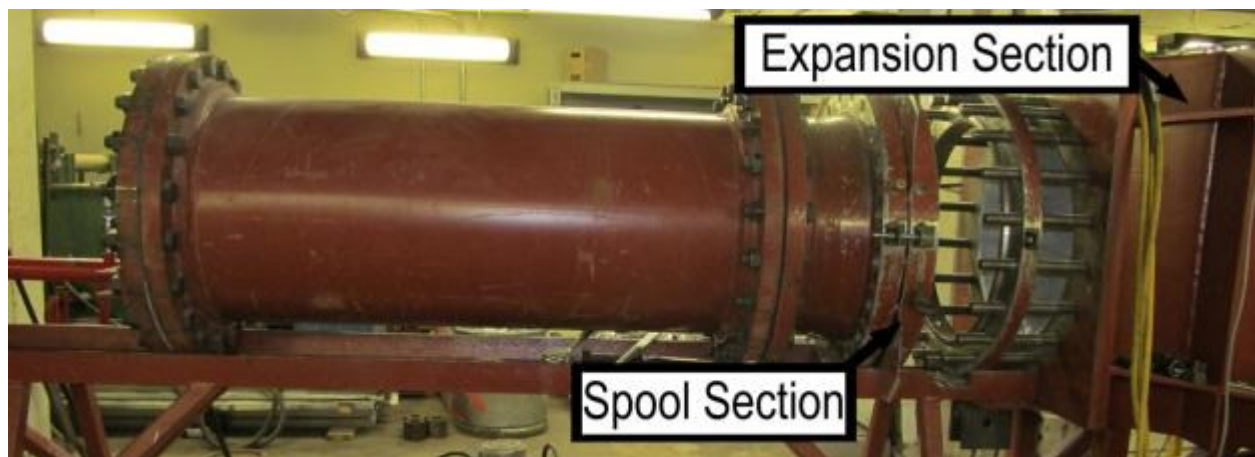


Figure 3-20 Shock-tube's spool section

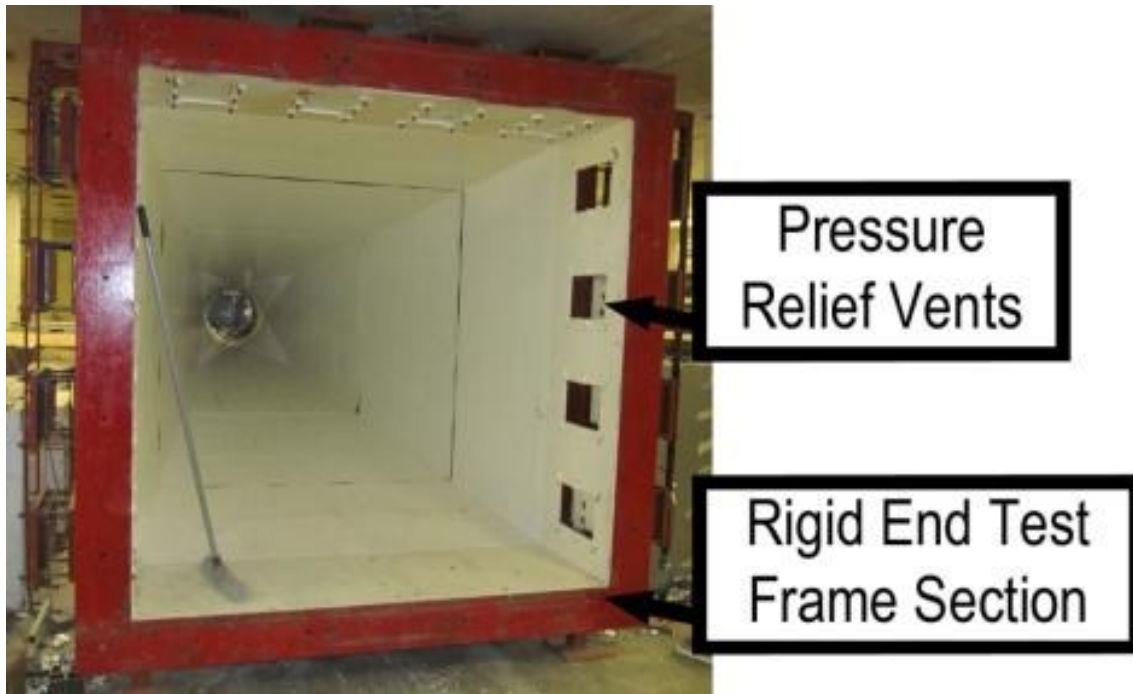


Figure 3-21 Shock-tube's rigid end test frame section

3.5.1.2 Lateral Load Transfer Device

A load transfer device (LTD) is used to redirect the shockwaves generated at the shock-tube opening onto the non-planar beam specimens. The test fixture closely replicates the load and a support condition used for the static tests, and applies the shockwave generated by the Shock-Tube as two point loads. The lateral load transfer device, which covers the entire 2 m x 2 m opening is composed of two side-by-side rigid steel panels which are 2032 mm tall and 1000 mm wide. The LTD can be seen in Figure 3-22 in detail with the rigid end test frame. The load transfer panels can be further subdivided into several parts which consist of rigid plates and hollow steel section (HSS) members (see Figure 3-23). Sliding hinges are provided near the top and bottom supports, allowing free lateral movement of the middle portion of the LTD without causing reactions at the hinge locations (Jacques, 2016). In order to redirect the blast loads to the specimen, two I-steel load-transfer beams (160 mm x 165 mm x 1200 mm) are attached to the LTD to transfer the pressure as two concentric point loads, as can be seen in Figure 3-23.

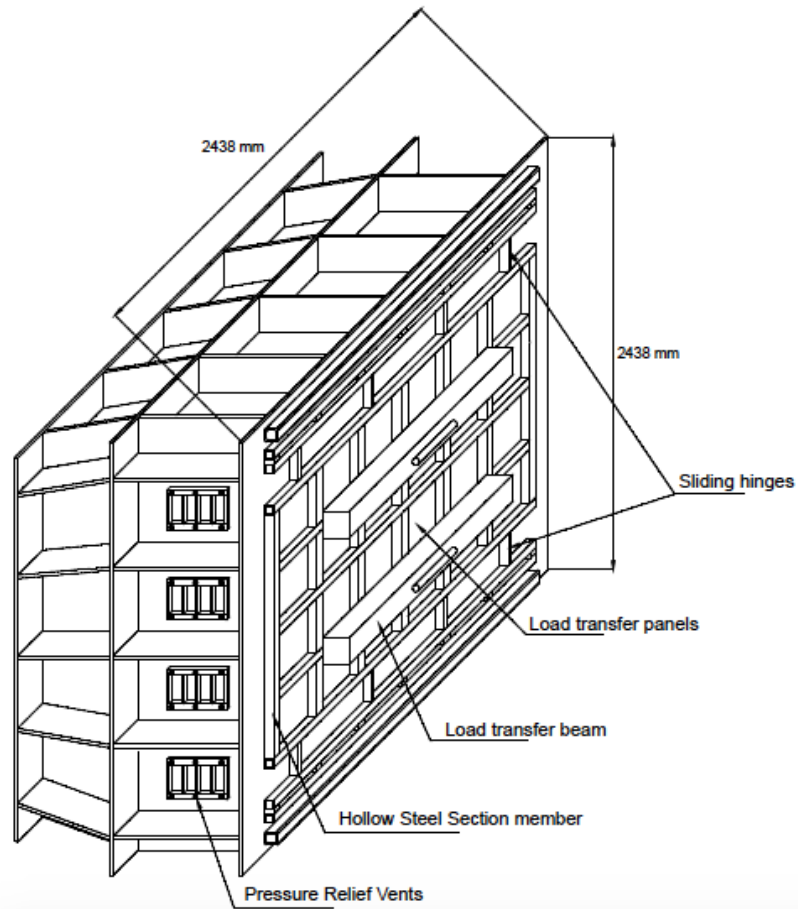


Figure 3-22 Shock-tube's isotropic view of lateral load transfer device

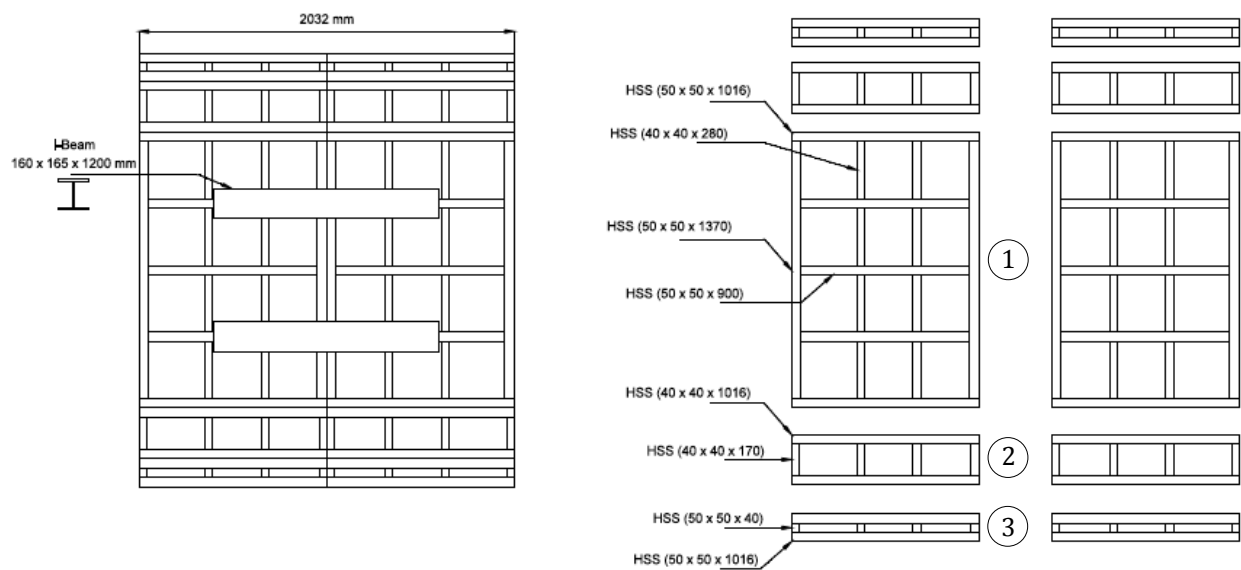


Figure 3-23 Shock-tube's lateral load transfer section

3.5.1.3 Supports

The support setup was configured to closely simulate simply supported beam conditions with loads applied as two point loads over a constant moment region of 750 mm, with two shear spans of equal length of approximately 741 mm. The top and bottom reaction assemblies consist of a stiffened front support and rear support. The front support consisted of a 1000 mm long square 152 x 152 mm square HSS section. Two circular load cells with pin joints were welded onto a front HSS support with the assembly tightened using four 19 mm diameter threaded steel rods, as seen in Figure 3-24. The rear supports consist of 500 mm long 51 mm x 51 mm square HSS with a 440 mm long smaller HSS section which simulates roller condition. A photograph showing the details of the supports and load setup is shown in Figure 3-25.

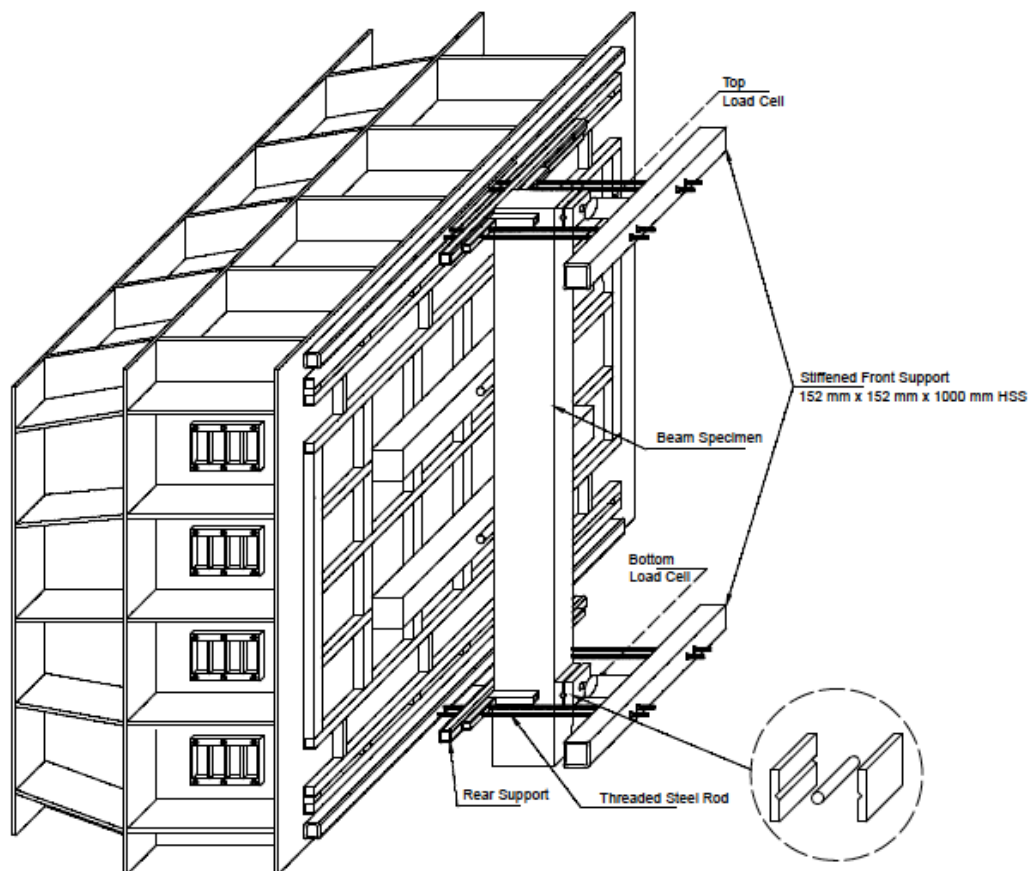


Figure 3-24 Supports details and load cells



Figure 3-25 Photograph, top and bottom support

3.5.1.4 Pressure Sensors

Two PCB Piezotronics Model #112A22 piezoelectric pressure sensors were attached at the side and bottom of the shock-tube's end frame and connected to the data acquisition unit. These pressure sensors are capable of sending the trigger signal to the data acquisition unit in order to record data samples. Furthermore, pressure-time histories can be obtained when the test is finished.

3.5.1.5 Linear Variable Displacement Transducers (LVDT)

Three Celesco CLWG-300 linear variable displacement transducers (LVDT) with a gage length of 300 mm were used to measure the displacements of the beam specimen and the shock-tube. As seen in Figure 3-26, two LVDTs were attached at the specimen mid-span and one-third span in order to measure the complete deflection of the beam specimens. One LVDT was also installed at the bottom of the shock tube to subtract any displacement from the installation itself.

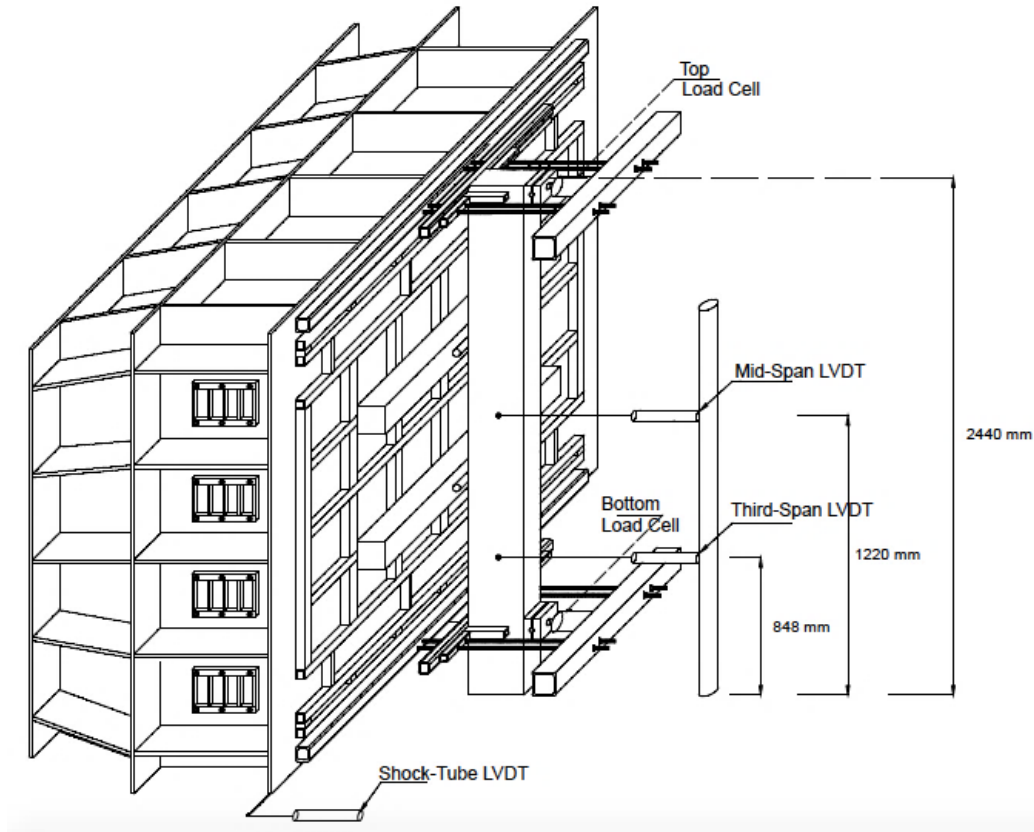


Figure 3-26 The location of the linear variable displacement transducers (LVDT)

3.5.1.6 Data Acquisition System

A Yokogawa SL1000 High Speed Data Acquisition Unit, which is capable of recording 100,000 samples of data per second, was used to record the data received from the pressure sensors, LVDTs for displacement measurement, and reinforcement strain gauges during testing. At each blast load, the data acquisition unit was set to loop, record, and constantly overwrite data until the pressure sensor captured a signal caused by the shock wave, which is called the trigger. The data is saved over a period of 3.0 seconds, including a 10.0% pre-trigger phase lasting 0.3 seconds. After finishing the test, the data was imported into a csv format file using the XViewer software for further analysis.

3.5.1.7 High Speed Video Camera

The AOS Technology X-PRI high speed camera was used to record the response of the beam specimens during the blast tests. This camera records full color images with an 800 x 600 pixel resolution at a rate of 1000 frames per second. The camera was aimed at the south side of the beam specimens and adjusted to record at a rate of 500 frames per second. The same scenario used on the data acquisition unit was used for the high speed camera, which ran in a loop until triggered, recording an uncompressed video file with a 10.0% pre-trigger phase.

3.5.1.8 Strain Gauges

In order to capture strains in the reinforcing bars FLA-6-350-11 strain gauges with a length of 6 mm and a resistance of 350 ± 1.0 ohms were attached at the middle of the reinforcing bars. For each specimen, one strain gauge was installed and connected to the data acquisition system in order to record the yielding of the reinforcement steel bars as seen in Figure 3-27.

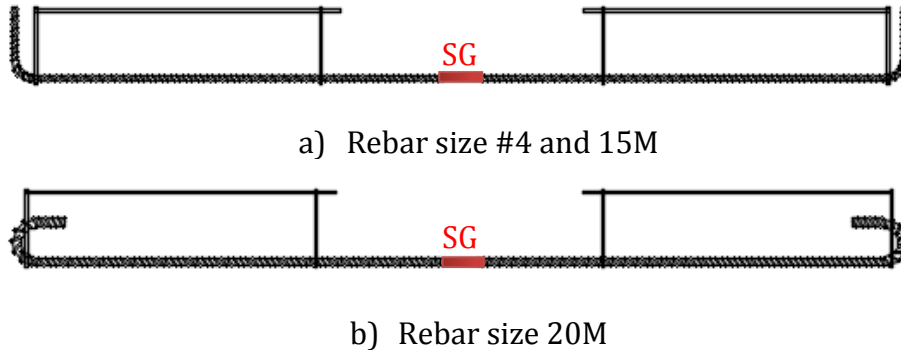


Figure 3-27 Strain gauges location

3.5.1.9 Experimental Procedure

Each specimen was subjected to repeated blast loads until failure. Failure included concrete crushing, severe concrete splitting and spalling. Two different repeated blast procedures were used, one for series #4 and the other for series 15M & 20M. Table 3-9 and Table 3-10 report the driver pressure, driver length, as well as the resulting average reflected pressures, reflected impulses and positive durations for each blast. The impulse was gradually increased, as seen in Table 3-9 and Table 3-10 by fixing the driver length at 9 ft (2473 mm) and gradually increasing the driver pressures in 10 to 20 psi increments. The initial test (Blast 1) was expected to keep the specimens within the elastic range, while the second blast (Blast 2) was expected to lead to yielding in the steel reinforcement. Beyond Blast 2, the driver pressures were incremented by 10 psi (69 kPa) for series #4 until 60 psi (414 kPa). For the 15M/20M series driver pressure was incremented by 20 psi (138 kPa) upto 70 psi (483 kPa). For some specimens which had not failed, the last pressures were further increased by 10 psi (69 kPa) to reach an impulse of 774 kPa-ms. Sample pressure-time histories for each blast are shown in Figure 3-28.

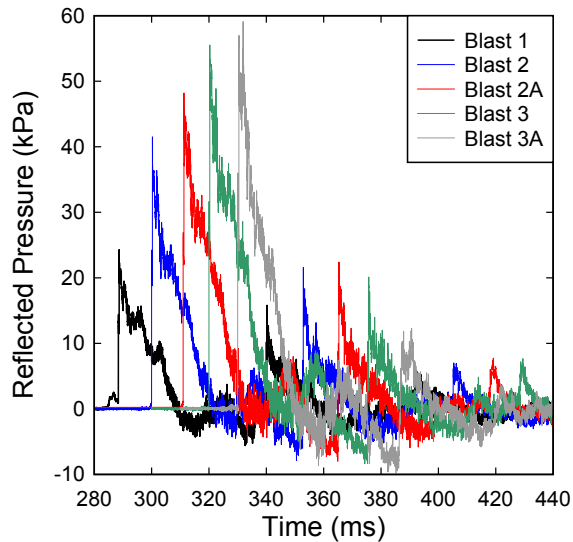
Table 3-9 Blast test properties for beams reinforced with #4 rebars

Test #	Driver Pressure kPa (psi)	Driver Length mm (ft)	Avg. Reflected Impulse (I_r) kPa-ms	Avg. Reflected Pressure(P_r) kPa	Avg. Positive Phase Duration (t_p) ms
Blast 1	117 (17)	2743 (9)	240.14	24.24	19.83
Blast 2	207 (30)	2743 (9)	365.05	41.23	17.73
Blast 2A	276 (40)	2743 (9)	432.82	46.15	18.78
Blast 3	345 (50)	2743 (9)	541.30	58.16	18.63
Blast 3A	414 (60)	2743 (9)	562.62	68.75	16.37

Table 3-10 Blast test properties for beams reinforced with 15M and 20M rebars

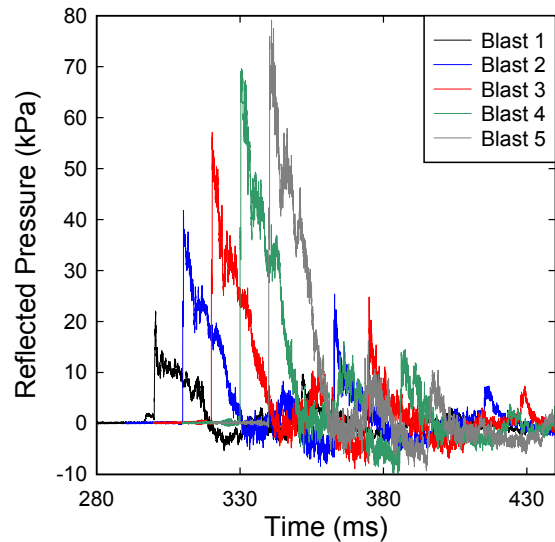
Test #	Driver Pressure kPa (psi)	Driver Length mm (ft)	Avg. Reflected Impulse (I_r) kPa-ms	Avg. Reflected Pressure(P_r) kPa	Avg. Positive Phase Duration (t_p) ms
Blast 1	117 (17)	2743 (9)	240.14	24.24	19.83
Blast 2	207 (30)	2743 (9)	365.05	41.23	17.73
Blast 3	345 (50)	2743 (9)	541.30	58.16	18.63
Blast 4	483 (70)	2743 (9)	719.62	71.95	20.02
Blast 5	552 (80)	2743 (9)	774.15	84.00	18.52

Sample Pressure Time Histories



a) Blasts 1-4 for beams reinforced with rebar #4

Sample Pressure Time Histories



b) Blasts 1-5 for beams reinforced with rebar 15M and 20M

Figure 3-28 Sample pressure time histories for blasts 1-4 & blasts 1-4 for beams reinforced with rebar #4, 15M, and 20M

3.5.2 Static Loading Test

3.5.2.1 Load Setup and Instrumentation

The setup shown in Figure 3-30 was used to test all the specimens under quasi-static four-point bending loading. All the beams were simply supported over a length of 2232 mm and subjected to two point loads, spaced at 750 mm apart resulting in a constant moment region with two equal shear spans of 741 mm (shear span-to-effective depth ratio a/d of approximately 3.69). The loading was applied using a hydraulic jack with manual load-pump, mounted on a circular load cell. The jack rested on a rigid I-shaped spreader beam which transferred the loading as two point loads onto the concrete beams. The load and support details are shown in Figure 3-29.

The tensile strain in the steel reinforcement was monitored using electrical resistance strain gauges. This gauge was glued to one reinforcing bar in each specimen and located in the mid-span as shown in Figure 3-27. Beam deflections were recorded at mid-span using one displacement cable transducer (DCT). The load cell, the strain gauge wires and the DCT were hooked to a data acquisition system programmed to record loading and deformation readings.

3.5.2.2 Testing Sequence

All the specimens were subjected to the same loading sequence. The specimens were tested first under load-control, with an interval of 5 kN between load stages until 20 kN in order to observe the first hairline cracks. After that, the load interval was increased to 10 kN and testing continued until failure in shear or the onset of yielding. After yielding, loading was switched to deflection-control (with 5 mm left in between subsequent load stages) and continued until the beam failed by crushing of the concrete or until the maximum deflection reached displacements which exceeded 70 mm.

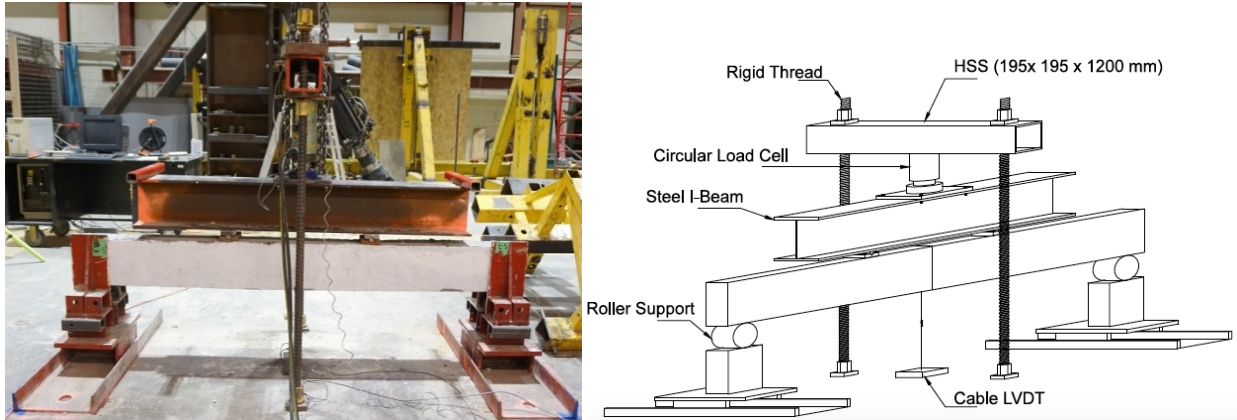


Figure 3-29 Photograph, static loading test setup and detailed graph

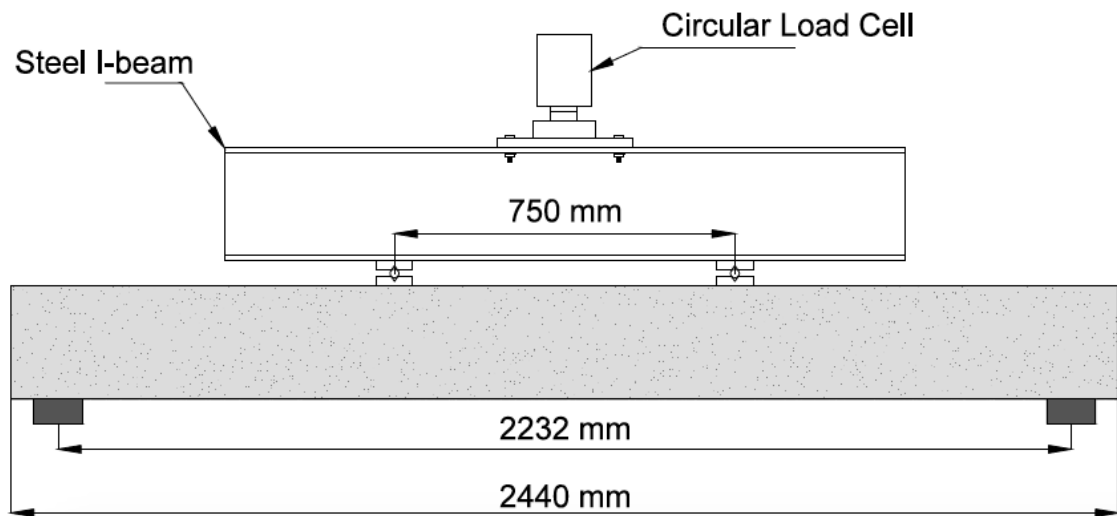


Figure 3-30 Static test geometry and dimensions

Chapter 4: Results of the Static Test Experiments

4.1 Chapter Overview

This chapter summarizes the results of the experiments conducted on the nine reinforced concrete beams tested under quasi-static (low strain-rate) loading. The chapter presents the results for each individual specimen which includes load-deformation response, brief summary of major events during testing along with photographs of the specimens at various load stages. The chapter is divided into four sections: section 4.2 provides a table with a summary of the experiment results, while sections 4.3, 4.4 and 4.5 present the results of the beams in each series.

4.2 Summary of Experimental Results

A summary of the results for the beams in series #4, 15M and 20M, is included in Table 4–1. The table includes a summary of yield (P_y) and maximum (P_{max}) loads, displacement at yield (Δ_y) and maximum displacement at failure (Δ_{max}), and failure type.

Table 4–1 Static test data summary

Specimen ID	Load (kN)		Displacement (mm)		Failure Type
	P_y	P_{max}	Δ_y	Δ_{max}	
NSC-F0-#4-S	56.0	64.2	14.4	45.1	Flexure: Concrete crushing & spalling of concrete in tension; steel yielding
HSC-F0-#4-S	58.1	75.5	10.9	58.6	Flexure: Concrete crushing & spalling of concrete in tension; steel yielding
HSC-F1(ZP)-#4-0	70.0	75.1	12.8	82.2	Flexure: Controlled concrete crushing with fibre pullout; steel yielding
HSC-F0-15M-S	94.5	104.6	14.9	40.7	Flexure: Concrete crushing; steel yielding
HSC-F1(ZP)-15M-0	111.6	115.9	14.5	124.0	Flexure: Controlled concrete crushing with fibre pullout; steel yielding
HSC-F0-20M-0	-	83.8	-	10.7	Shear
HSC-F0-20M-S	118.2	137.5	15.0	31.0	Flexure: Concrete crushing; steel yielding
HSC-F1(ZP)-20M-0	138.2	141.8	14.6	84.9	Flexure: Controlled concrete crushing with fibre pullout; steel yielding
HSC-F1(ZP)-20M-S	145.5	148.4	16.9	125.9	Flexure: Controlled concrete crushing with fibre pullout; steel yielding

4.3 Description of Experiment Results – Series #4

4.3.1 Specimen NSC-F0-#4-S

The beam specimen in this experiment was constructed using self-consolidating concrete, which is considered to be normal-strength concrete, in order to investigate the effects of concrete strength on beam response. The beam was reinforced longitudinally with 2-#4 bars, and transversely with 6.3 mm stirrups arranged at spacing of 100 mm. The experimental load-displacement relationship is shown in Figure 4-1. Photographs showing the progress of beam damage and failure with load are included in Figure 4-2.

At the first load stage, the first hairline cracks were caught at a load of approximately 5 kN. After two load stages, flexural cracks occurred on the flexural region at a load of 15.7 kN. When additional loads were applied, further cracks formed in the moment region. Several horizontal cracks at the level of the longitudinal steel developed in the tension zone between the flexural cracks when the applied load reached 53.9 kN ($\Delta = 17.2$ mm). There was also the formation of flexural-shear cracks in the two shear spans with crack spacing stabilizing. As the applied load increased, the electric strain gauge recorded the yield of the longitudinal reinforcement at a load of approximately 56.0 kN ($\Delta_y = 14.4$ mm). The flexural cracks widths gradually increased as the load increased. The ends of the horizontal cracks connected, creating a main horizontal crack when the applied load reached 64.2 kN at a mid-span displacement of 39.1 mm. The beam then failed at a maximum deflection of 45.1 mm, due to concrete crushing in the compression zone. This failure was accompanied with concrete spalling in the tension zone.

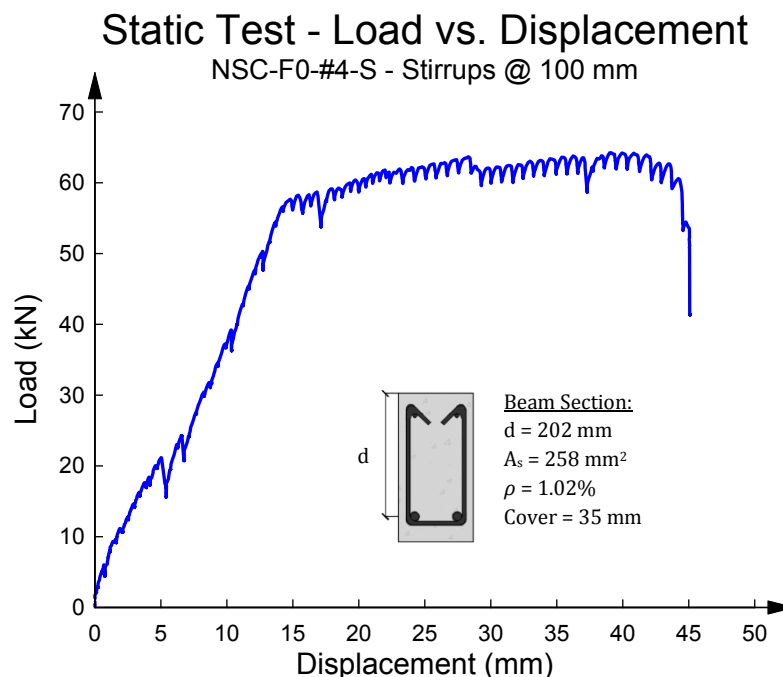


Figure 4-1 Load-deflection response for beam NSC-F0-#4-S

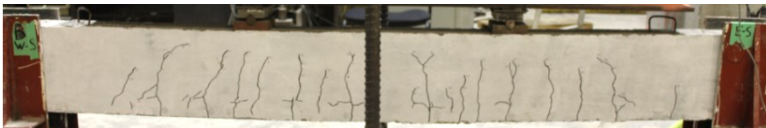
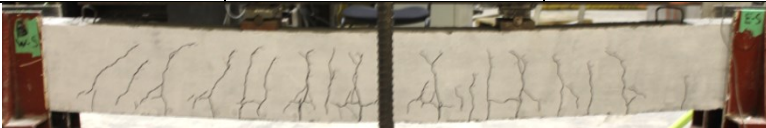
Experiment parameters			Comments
Load stage	2P (kN)	Mid-span Δ (mm)	- First hairline cracks appear in the constant moment region
L1	4.4	0.4	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Further flexural cracks appear in the constant moment region - Horizontal crack begins to form the level of long. steel
L3	15.7	3.3	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Flexural-shear cracks appear in the E & W shear spans - Cracks extend further in moment region
L6	53.9	17.2	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Horizontal cracks extended
L11	60.7	37.6	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Cracks become wider - Horizontal cracks joining each other forming one main crack at level of steel
L13	64.2	39.1	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Specimen fails: concrete crushing in compression zone and spalling in the tension zone
Failure	41.4	45.1	
			

Figure 4-2 Major events for beam NSC-F0-#4-S

4.3.2 Specimen HSC-F0-#4-S

This beam specimen was composed of high-strength concrete, and was designed with #4 longitudinal steel bars and transverse reinforcement arranged at spacing of 100 mm. The experiment results in terms of load-displacement relationship are shown in Figure 4-3. Photographs showing the progress of the cracking and damage with loading are included in Figure 4-4.

In the second load stage, the first hairline crack was observed to occur in the constant moment region at a load of 11.7 kN. Soon thereafter flexural cracks began to form in the constant moment region. At a load of 47.5 kN, flexural-shear cracks spread on the west side of the beam and the flexural crack in the mid-span became wider. New flexural-shear cracks were distributed on the east side of the beam when the applied load reached 57.6 kN. The yield of the longitudinal reinforcement was recorded at a load of 58.1 kN ($\Delta_y = 10.9$ mm). As test progressed, a couple of forking cracks formed in the flexural region, and a small horizontal crack appeared on the tension zone at the level of the tension steel at a load of approximately 62.2 kN. As strains exceeded yielding, the crack widths in the moment region became wider and the horizontal cracks connected to each other when further load was applied. Failure then occurred with concrete spalling in the tension zone and concrete crushing in the compression zone at a maximum deflection of 58.6 mm.

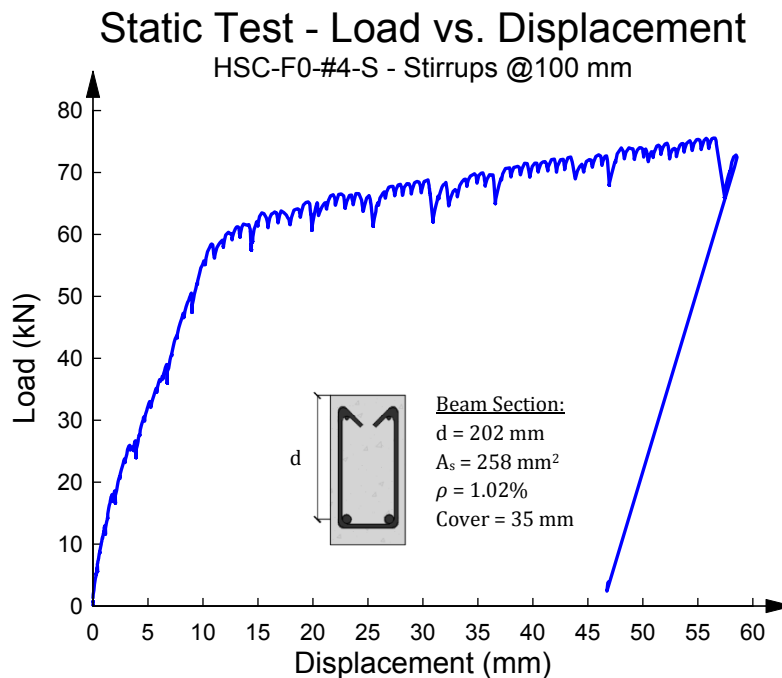


Figure 4-3 Load-deflection response for beam HSC-F0-#4-S

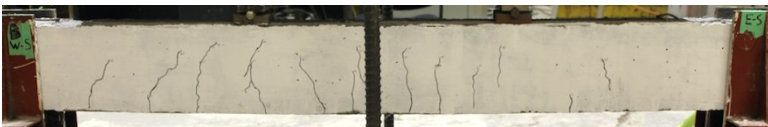
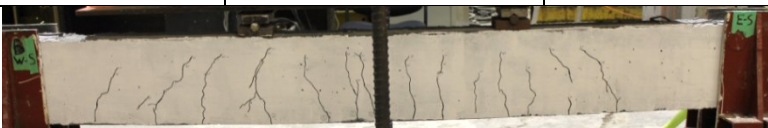
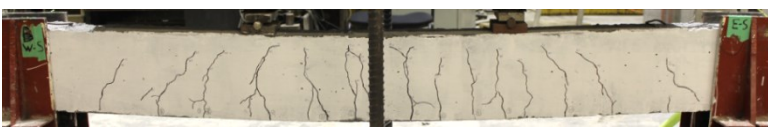
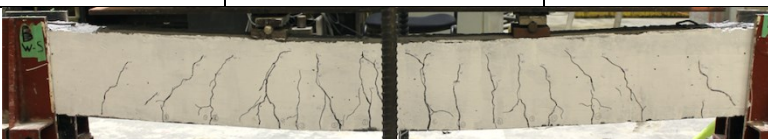
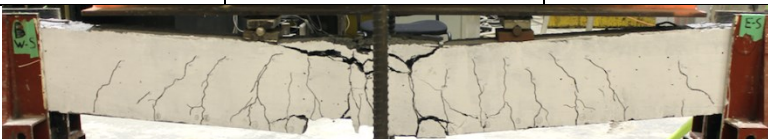
Experiment parameters			Comments
Load stage	2P (kN)	Mid-span Δ (mm)	- First hairline crack appeared in the constant moment region
L2	11.7	1.1	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- major cracks appeared in the constant moment region - Flexural-shear cracks forming on the W- side shear span
L6	47.5	8.9	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Flexural-shear cracks formed on the E- side shear span
L7	57.6	14.4	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Cracks extend further with forking cracks forming - Horizontal cracks appeared at the level of tension steel
L8	62.2	30.9	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Major cracks became wider
L11	68.1	46.9	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Specimen fails: concrete crushing on compression zone and spalling on tension zone
Failure	4.5	58.6	
			

Figure 4-4 Major events for beam HSC-F0-#4-S

4.3.3 Specimen HSC-F1(ZP)-#4-0

This beam was composed of high-strength concrete, #4 bars, and steel fibres at a volumetric ratio of 1.0%. It is noted that this specimen did not have transverse reinforcement. The experiment results in terms of load-displacement relationship are shown in Figure 4-5. Photographs showing the progress of the cracking and damage with loading are included in Figure 4-6.

The first hairline crack was observed in the moment region, at a load of 10.2 kN. When the applied load increased, further flexural cracks began to form and flexural-shear cracks developed on the east and west side shears spans of the beam at a load of 56.8 kN. After several load stages, a forking crack emerged in the moment region at a load of approximately 64.3 kN. The strain gauge measured the yield of the reinforcing bar when the applied load reached 70 kN ($\Delta_y = 12.8$ mm). As the applied load and deflection increased, the strain in the longitudinal steel increased, causing an increase in the crack widths in the moment region. Some of the steel fibres that were bridging the major crack were pulled out when the applied load reached 69.9 kN with a displacement of 50.4 mm; as well, a new major flexural crack appeared in the moment region. The beam then failed at a maximum displacement recorded in the mid-span of 82.2 mm, with a very large crack opening in the mid-span region.

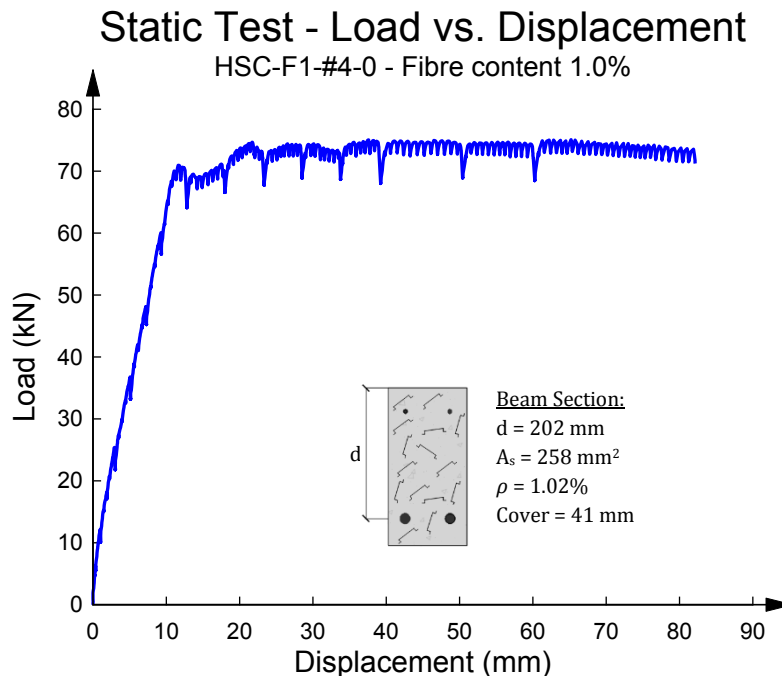


Figure 4-5 Load-deflection response for beam HSC-F1(ZP)-#4-0

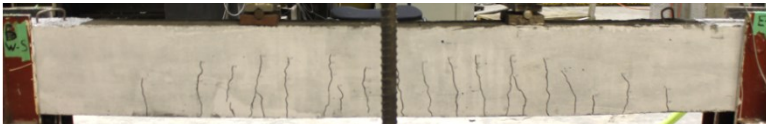
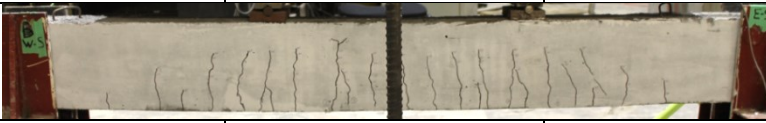
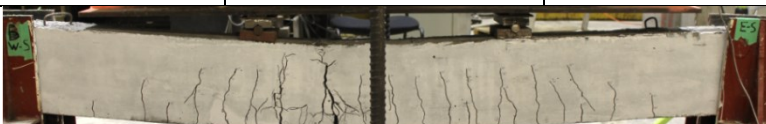
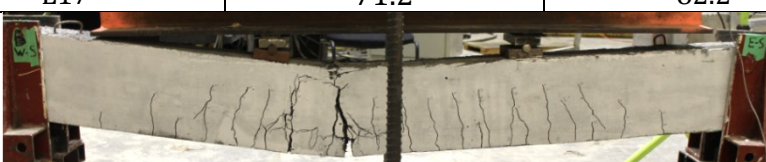
Experiment parameters			Comments
Load stage	2P (kN)	Mid-span Δ (mm)	- First hairline crack appeared in the constant moment region
L2	10.2	1.0	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- major crack observed at constant moment region - flexural-shear cracks in the east and west sides
L6	56.8	9.3	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Forking crack appeared on the moment region
L8	64.3	12.8	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Crack at the mid-span became wider
L10	67.9	23.4	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Some steel fibres were pulled out - New major crack appeared
L15	69.9	50.4	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Specimen fails: Very large displacement and large opening of major crack in the mid-span
L17	71.2	82.2	
			

Figure 4-6 Major events for beam HSC-F1(ZP)-#4-0

4.4 Description of Experiment Results – Series 15M

4.4.1 Specimen HSC-F0-15M-S

This beam was composed of the same materials as the HSC-F0-#4-S beam specimen, but included 15M instead of #4 steel bars. The experiment results for the load-displacement relationship are shown in Figure 4-7. Photographs showing the progress of damage and cracking are included in Figure 4-8.

The first hairline crack was observed at a load of 16.3 kN. As the applied load increased, flexural cracks appeared in the moment region, and flexural-shear cracks spread over the west side of the beam at a load of 57.6 kN. When more loading was applied on the beam, new flexural-shear cracks appeared on the east side of the beam at a load of approximately 81.6 kN. The major crack in the moment region became more visible to the eye when the applied load reached 89.4 kN. The longitudinal reinforcement yielded at a load of 94.5 kN ($\Delta_y = 14.9$ mm). As the load increased, the crack width increased gradually until the beam failed at a maximum deflection of 40.7 mm in the mid-span, with failure caused by concrete crushing in the compression zone.

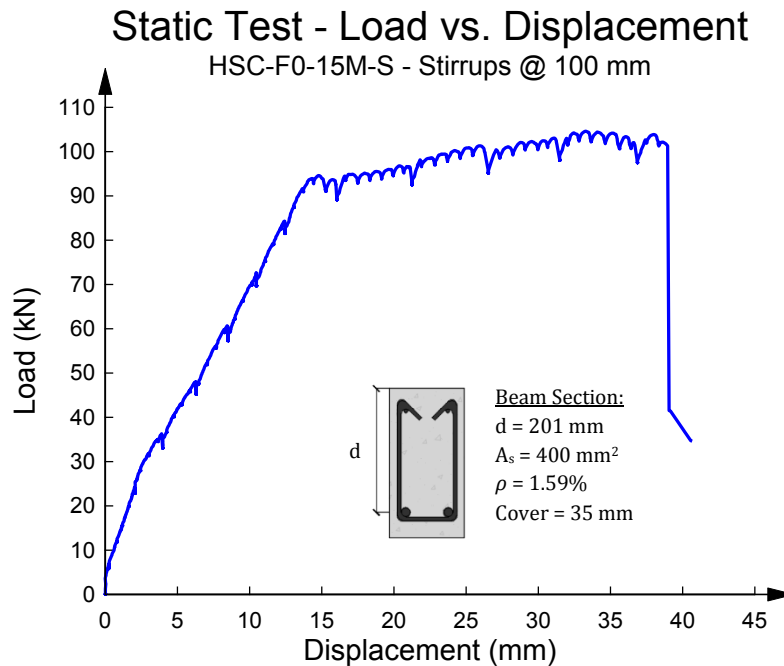


Figure 4-7 Load-deflection response for beam HSC-F0-15M-S

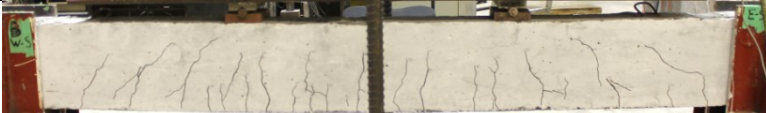
Experiment parameters			Comments
Load stage	2P (kN)	Mid-span Δ (mm)	- First hairline crack appeared in the constant moment region
L3	16.3	2.7	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Flexural cracks distributed on moment region - Flexural-shear cracks on W- side
L7	57.6	8.5	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Flexural-shear cracks appeared on E- side
L9	81.6	12.5	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Major crack on the moment region begins to increase in width - Cracks spread over beam span
L10	89.4	16.0	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Cracks became wider
L13	98.2	31.5	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Specimen fails: concrete crushing on compression zone
Failure	37.5	40.7	
			

Figure 4-8 Major events for beam HSC-F0-15M-S

4.4.2 Specimen HSC-F1(ZP)-15M-0

This specimen was constructed with the same components as the HSC-F0-15M-S beam, except for the addition of steel fibre reinforcement with a fraction volume of 1.0% and the omission of shear reinforcement. The experiment results in terms of load-displacement relationship are shown in Figure 4-9. Photographs showing the progress of damage and cracking are included in Figure 4-10.

In the third load stage, the first hairline crack was observed at a load of 17.0 kN. When further loading was applied on the beam, flexural cracks formed in the moment region and flexural-shear cracks appeared on the east and west sides of the beam at a load of approximately 57.9 kN. These cracks developed gradually as the applied load increased. At the mid-span of the beam, the major crack became wider and more visible when the load reached 104.8 kN, and the yielding of the reinforcing bars was recorded at a load of 111.6 kN ($\Delta_y = 14.5$ mm). After applying further loads, the major crack became wider and the steel fibres that were bridging the major crack began to gradually pull out as the load increased. At a load of 107.0 kN ($\Delta = 42.1$ mm), the steel fibres pull out of fibres from the concrete became more obvious. When the mid-span deflection reached 62.9 mm, the steel fibres were completely pulled out and the main crack split wide open. The beam was considered failed at a very large maximum deflection of 124.0 mm due to splitting of concrete at the major crack location in the tension zone.

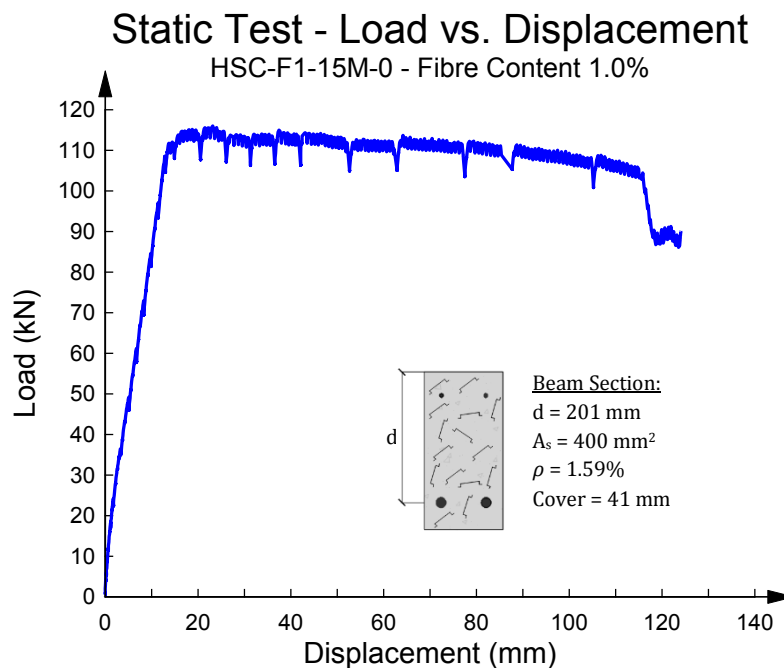


Figure 4-9 Load-deflection response for beam HSC-F1(ZP)-15M-0

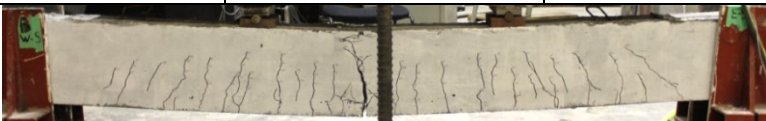
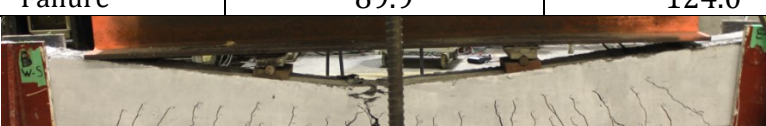
Experiment parameters			Comments
Load stage	2P (kN)	Mid-span Δ (mm)	- First hairline crack appear in the constant moment region
L3	17.1	1.2	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Flexural cracks formed - Flexural-shear cracks formed
L7	57.9	6.8	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Flexural-shear cracks developed on the E- & W-side
L4	81.6	9.8	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Major crack on the moment region
L11	104.8	12.9	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Major cracks became wider and visible - Some steel fibres were pulled out
L16	107.0	42.1	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- The major crack became bigger and the steel fibre was pulled out completely
L18	105.5	62.9	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Specimen fails: concrete split and large opening of major crack in the mid-span
Failure	89.9	124.0	
			

Figure 4-10 Major events for beam HSC-F1(ZP)-15M-0

4.5 Description of Experiment Results – Series 20M

4.5.1 Specimen HSC-F0-20M-0

This beam was composed of high-strength concrete and 20M bars. Steel fibres and transverse reinforcement were absent in this specimen. The experiment results in terms of load-displacement relationship are shown in Figure 4-11. Photographs showing the progress of the damage to the beam are included in Figure 4-12.

In the third load stage, the first hairline crack formed at the mid-span at a load of 16.9 kN. As the applied load increased, flexural and flexural-shear cracks appeared on the east and west sides of the beam, and a forking crack developed in the constant moment region at a load of approximately 45.8 kN. Flexural-shear cracks developed on both sides of the beam when the applied load reached 69.5 kN. After that, the beam failed suddenly at a maximum deflection of 10.7 mm, with shear failure observed on the east side of the beam.

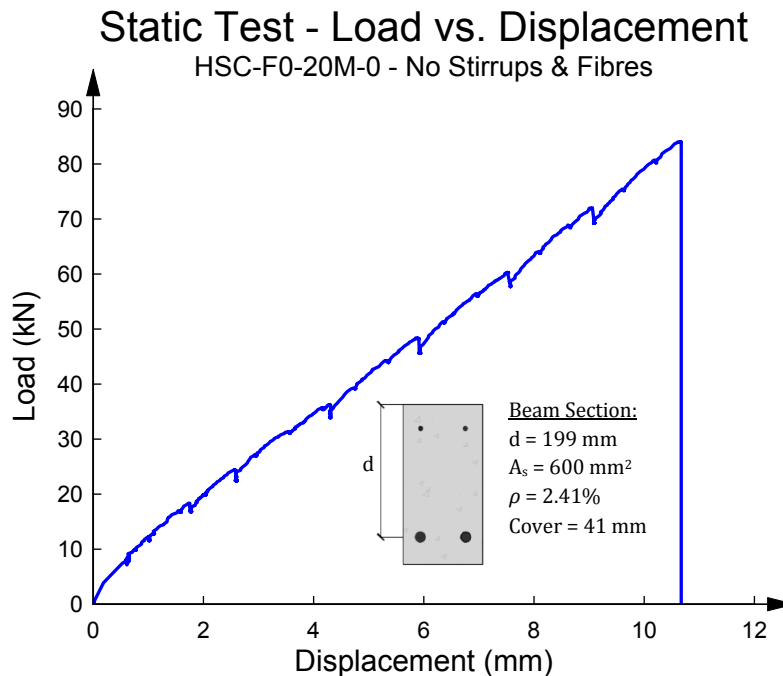


Figure 4-11 Load-deflection response for beam HSC-F0-20M-0

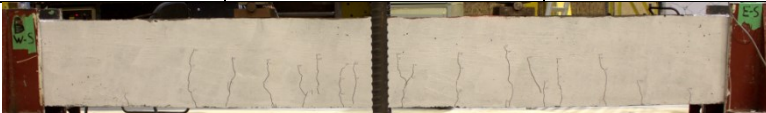
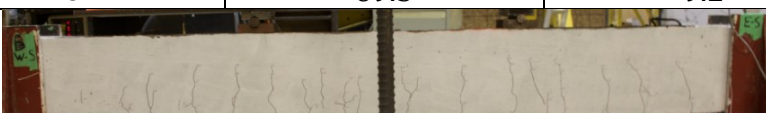
Experiment parameters			Comments
Load stage	2P (kN)	Mid-span Δ (mm)	- First hairline crack appeared in the constant moment region
L3	16.9	1.8	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Forking crack appear at constant moment region - Flexural-shear cracks developed
L6	45.8	5.9	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Further flexural-shear crack formed on the E- & W-side
L8	69.5	9.2	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Specimen fails: Sudden shear failure on the E-side
Failure	83.8	10.7	
			

Figure 4-12 Major events for beam HSC-F0-20M-0

4.5.2 Specimen HSC-F0-20M-S

This specimen matches the composition of the HSC-F0-20M-0 specimen, but contained 6.3 mm transverse shear reinforcement, arranged at spacing of 100 mm. The experimental load-displacement relationship is shown in Figure 4-13. Photographs showing the progress of the damage to the beam are included in Figure 4-14.

When the test reached the third load stage, the first hairline cracks were seen at a load of 17.3 kN. Flexural cracks developed on the moment region and small flexural-shear cracks appeared on the east side when the applied load reached a load of approximately 46.2 kN. During the same loading, flexural-shear cracks grew on both the east and west sides of the beam and continued to develop as load reached 81.6 kN. A major crack that was visible and measurable was observed in the moment region when the load reached 107.3 kN, followed by the yielding of the reinforcement bars at a load of 118.2 kN ($\Delta_y = 15.0$ mm). As the applied load increased, the major crack width in the flexural span increased slowly. The beam then failed at a maximum deflection of 31.0 mm, with failure caused by concrete crushing in the compression zone.

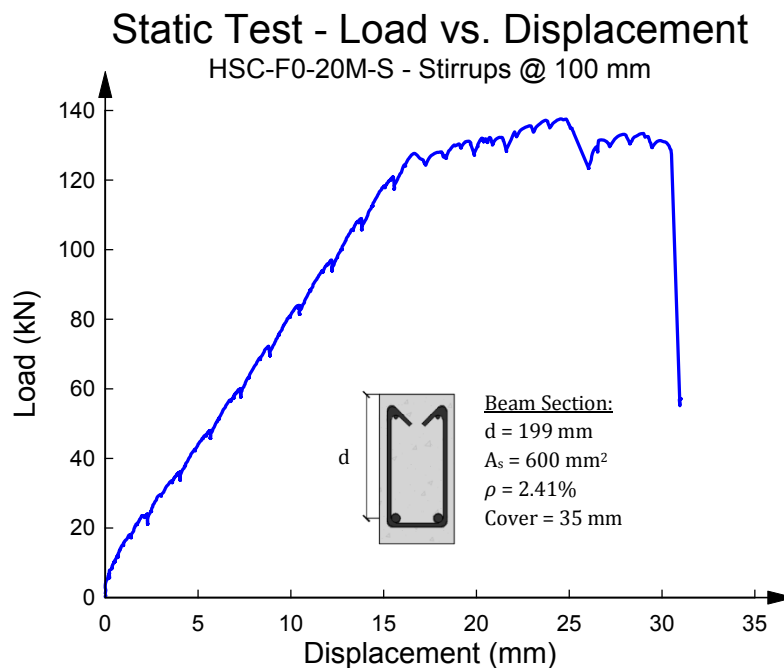


Figure 4-13 Load-deflection response for beam HSC-F0-20M-S

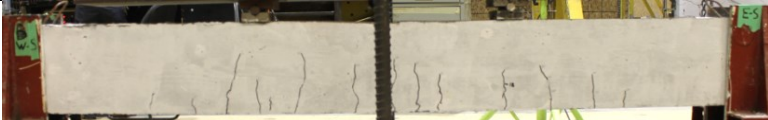
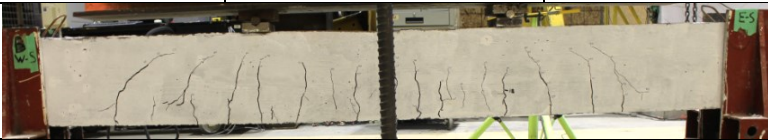
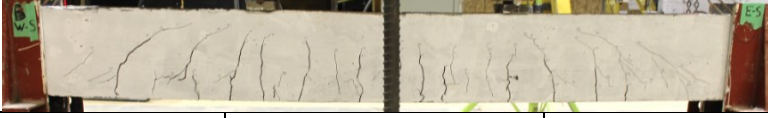
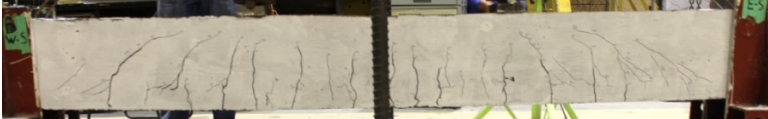
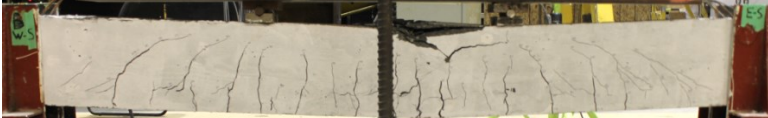
Experiment parameters			Comments
Load stage	2P (kN)	Mid-span Δ (mm)	- First hairline cracks appear in the constant moment region
L3	17.3	1.4	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Flexural cracks developed on the flexural region
L6	46.2	5.7	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Flexural-shear crack spread on the E- & W- side of the beam
L9	81.6	10.5	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Major crack appeared on the constant moment region
L11	107.3	13.8	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Major crack became wider
L12	117.5	15.6	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Specimen fails: concrete crushing on compression zone
Failure	57.1	31.0	
			

Figure 4-14 Major events for beam HSC-F0-20M-S

4.5.3 Specimen HSC-F1(ZP)-20M-0

This beam was composed of high-strength concrete, 20M bars, and steel fibre added at a volumetric ratio of 1.0% without stirrups. This beam was identical to HSC-F0-20M-0, with the exception of the fibres. The experimental results in terms of load-displacement relationship are shown in Figure 4-15. Photographs showing the progress of damage and cracking are included in Figure 4-16.

In the fourth load stage, the first hairline crack appeared on the flexural span at a load of 21.9 kN. When the applied load increased, new cracks developed over the flexural and shear spans at a load of approximately 92.9 kN. After progressing to the next load stages, the appearance of flexural-shear cracks on both the east and west side of the beam was recorded, and a major crack was observed in the moment region at a load of 127.7 kN. It is noted that the crack spacings were reduced when compared to companion beams without fibres. At a load of 138.2 kN ($\Delta_y = 14.6$ mm), the strain gauge captured the yield of the longitudinal reinforcement. As the applied load increased, the deflection of the beam specimen increased slowly, resulting in a growth of the major crack width and a partial pull out of the steel fibres. The beam then failed at a maximum deflection of 84.9 mm recorded in the mid-span, with a complete pull-out of the steel fibres from the concrete; this failure was accompanied by gradual crushing of the concrete in the compression zone.

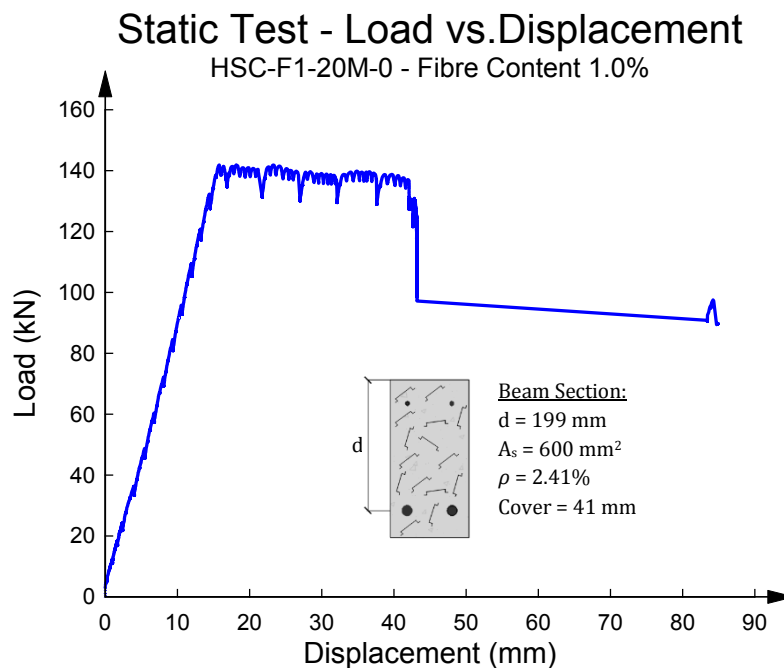


Figure 4-15 Load-deflection response for beam HSC-F1(ZP)-20M-0

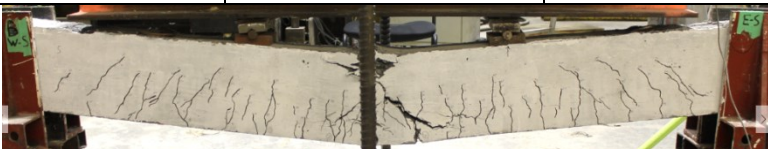
Experiment parameters			Comments
Load stage	2P (kN)	Mid-span Δ (mm)	- First hairline crack appear on the constant moment region
L4	21.9	2.4	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- More cracks distributed over the moment region and E- & W- sides
L10	92.9	10.6	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Major crack appeared on the constant moment region. Cracks are well distributed.
L13	127.7	13.9	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Further flexural-shear cracks on E- & W- side
L14	131.3	14.7	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Cracks became wider
L17	130.2	37.6	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Major cracks became wider
Failure	88.5	84.9	
			
			- Specimen fails: concrete crushing on compression zone

Figure 4-16 Major events for beam HSC-F1(ZP)-20M-0

4.5.4 Specimen HSC-F1(ZP)-20M-S

The composition of this beam matches that of the HSC-F1(ZP)-20M-0 specimen, except for the addition of 6.3 mm stirrups arranged at spacing of 100 mm. The experimental load-displacement curve is shown in Figure 4-17. Photographs showing the progress of the damage to the beam are included in Figure 4-18.

During the third load stage, the first hairline crack was observed in the moment region at a load of 16.8 kN. Flexural cracks developed in the moment region and flexural-shear cracks spread over the shear spans when the applied load reached 80.9 kN. The flexural-shear cracks developed on the east and west sides of the beam, and the major crack became more visible and measurable at a load of approximately 136.1 kN. The strain gauge recorded the yield of the longitudinal reinforcement at a load of 145.5 kN ($\Delta_y = 16.9$ mm). As the load increased, the width of the major crack increased, with the eventual formation of a diagonal crack in the moment region below the left load-bearing support. The steel fibres started pulling out when the load dropped to 97.5 kN ($\Delta = 90.7$ mm), and a new major crack formed on the right side of the flexural span. The steel fibres were pulled out on both of the major cracks, resulting in the failure of the beam at the maximum deflection of 125.9 mm; this failure was accompanied with concrete crushing in the compression zone.

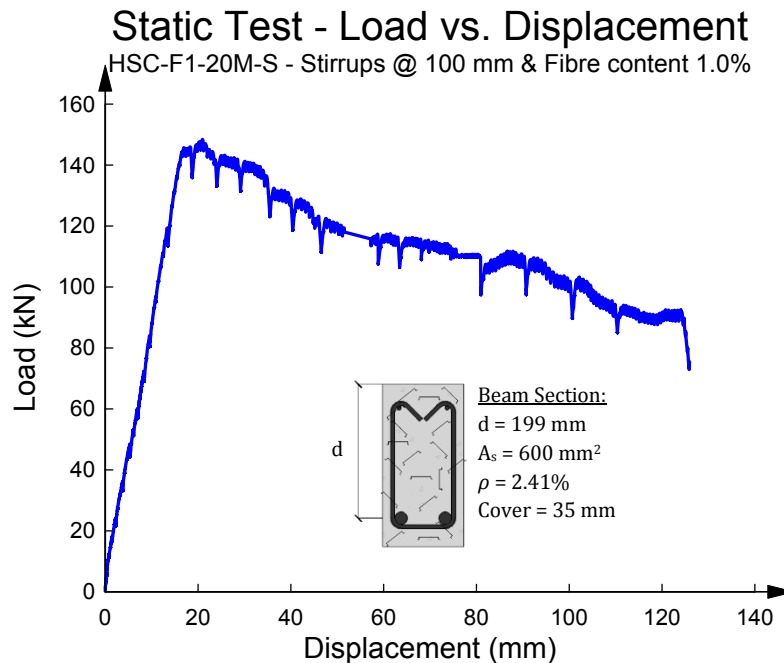


Figure 4-17 Load-deflection response for beam HSC-F1(ZP)-20M-S

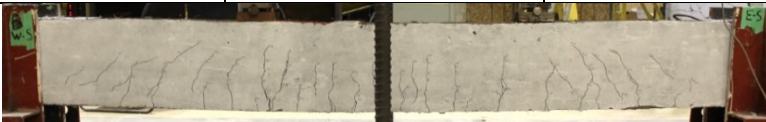
Experiment parameters			Comments
Load stage	2P (kN)	Mid-span Δ (mm)	- First hairline crack appeared in the constant moment region
L3	16.8	1.6	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- More cracks distributed over moment region and E- & W- sides
L9	80.9	9.6	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Major crack emerged - Flexural-shear cracks developed on E- & W- side
L11	136.1	18.7	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Major crack widen while remaining crack widths remain limited
L16	111.9	46.5	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Steel fibre started to be pulled out - New major crack appeared below the right load-bearing support
L21	97.5	90.7	
			
Load stage	2P (kN)	Mid-span Δ (mm)	- Specimen fails: concrete crushing on compression zone and new crack appeared with pulling out of the steel fibres
Failure	75.5	125.9	
			

Figure 4-18 Major events for beam HSC-F1(ZP)-20M-S

Chapter 5: Discussion of the Static Experimental Results

5.1 Chapter Overview

This chapter presents the experimental results of the nine reinforced concrete beams which were tested under quasi-static four-point bending. The effects of various parameters such as: concrete strength, fibres, fibre content/type, longitudinal steel reinforcement ratio and presence of stirrups, are investigated. The results are compared in terms of overall load-deflection response, initial stiffness, maximum load carrying capacity, maximum displacement, ductility, toughness and failure mode.

5.2 General Observations

This section presents a general summary of the experimental results of the nine beams which were tested under static four-point loading. The load-displacement curves are shown in Figure 5-1, while key parameters obtained from the load-displacement response are summarized in Table 5-1 and Table 5-2. These parameters include load and displacement at yield (P_y and Δ_y), initial beam stiffness (k), maximum load carrying capacity (P_{max}), maximum displacement at failure (Δ_{max}), ductility (defined as Δ_{max}/Δ_y) and toughness (area under the load-deflection curves upto $2\Delta_y$, $3\Delta_y$, $4\Delta_y$, and Δ_{max}). Comparisons of these parameters in the form of bar charts are shown in Figure 5-2 and Figure 5-3 along with notes on failure modes. A brief examination of the results indicates that all tested variables influence the static load-deflection response of the beams, with response and failure mode most strongly affected by the longitudinal reinforcement ratio and the addition of steel fibres. As expected, providing stirrups in HSC beams affects failure mode, while concrete strength has a moderate effect on load-deflection response. Further discussions on the effect of the test variables are provided in the sections that follow.

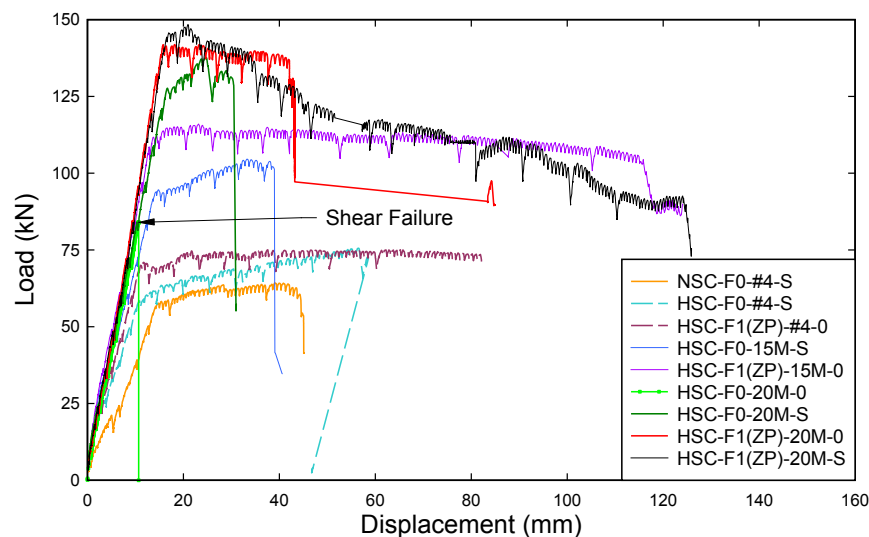


Figure 5-1 Load-displacement curves for all specimens

Table 5-1 Summary of static test results (1/2)

ID	Load		Displacement		Stiffness
	P_{max} (kN)	P_y (kN)	Δ_y (mm)	Δ_{max} (mm)	$k \left(\frac{N}{mm} \right)$
NSC-F0-#4-S	64.2	56.0	14.4	45.1	4529
HSC-F0-#4-S	75.5	58.1	10.9	58.6	5287
HSC-F1(ZP)-#4-0	75.1	70.0	12.8	82.2	5448
HSC-F0-15M-S	104.6	94.5	14.6	40.7	5573
HSC-F1(ZP)-15M-0	115.9	111.6	14.5	124.0	8318
HSC-F0-20M-0	83.8	-	-	10.7	7274
HSC-F0-20M-S	137.5	118.2	15.0	31.0	7911
HSC-F1(ZP)-20M-0	141.8	138.2	14.7	84.9	10314
HSC-F1(ZP)-20M-S	148.4	145.5	16.9	125.9	10031

Δ_{max} = maximum displacement, Δ_y = displacement at yield, P_y = Load at yield, P_{max} = Maximum load

Table 5-2 Summary of static test results (2/2)

ID	Ductility	Toughness (kN-mm)			
	$\frac{\Delta_{max}}{\Delta_y}$	$2\Delta_y$	$3\Delta_y$	$4\Delta_y$	Δ_{max}
NSC-F0-#4-S	3.13	1280	2182	-	2286
HSC-F0-#4-S	5.35	1056	1784	2545	3180
HSC-F1(ZP)-#4-0	6.42	1457	2396	3345	5621
HSC-F0-15M-S	2.74	2255	-	-	3272
HSC-F1(ZP)-15M-0	8.55	2592	4211	5843	12888
HSC-F0-20M-0	-	-	-	-	470
HSC-F0-20M-S	2.07	2899	-	-	2998
HSC-F1(ZP)-20M-0	5.83	2987	4910	6382	8827
HSC-F1(ZP)-20M-S	7.46	3668	5786	7743	13676

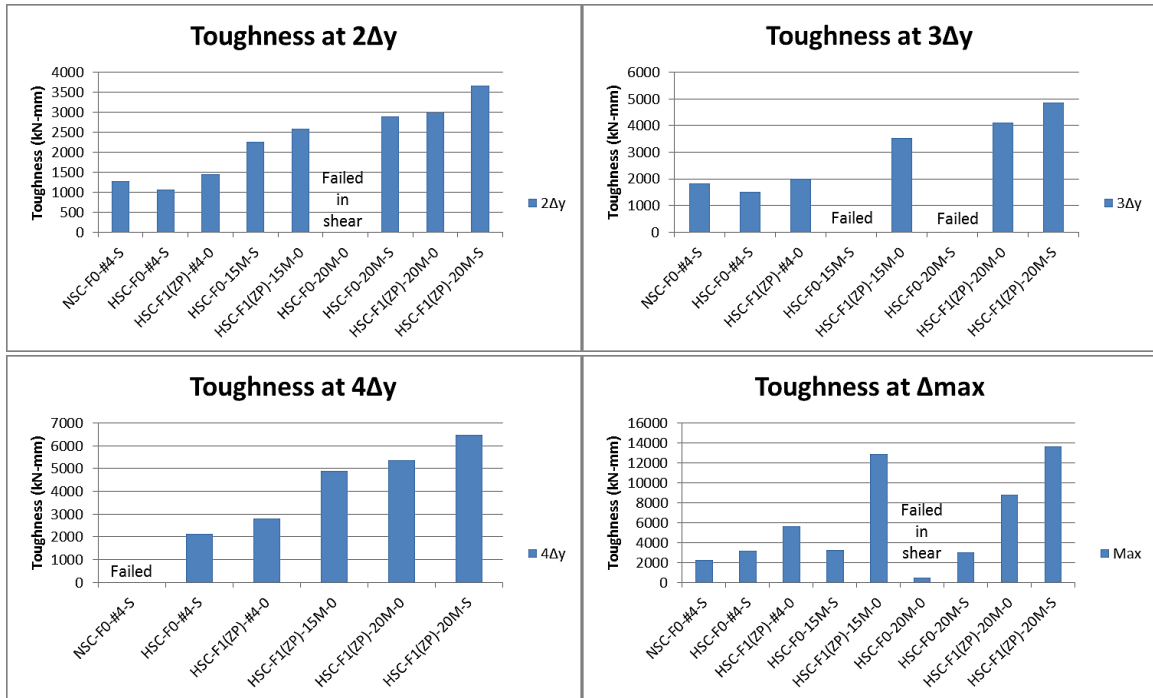


Figure 5-2 Toughness for each beam up to $2\Delta_y$, $3\Delta_y$, $4\Delta_y$, and Δ_{max}

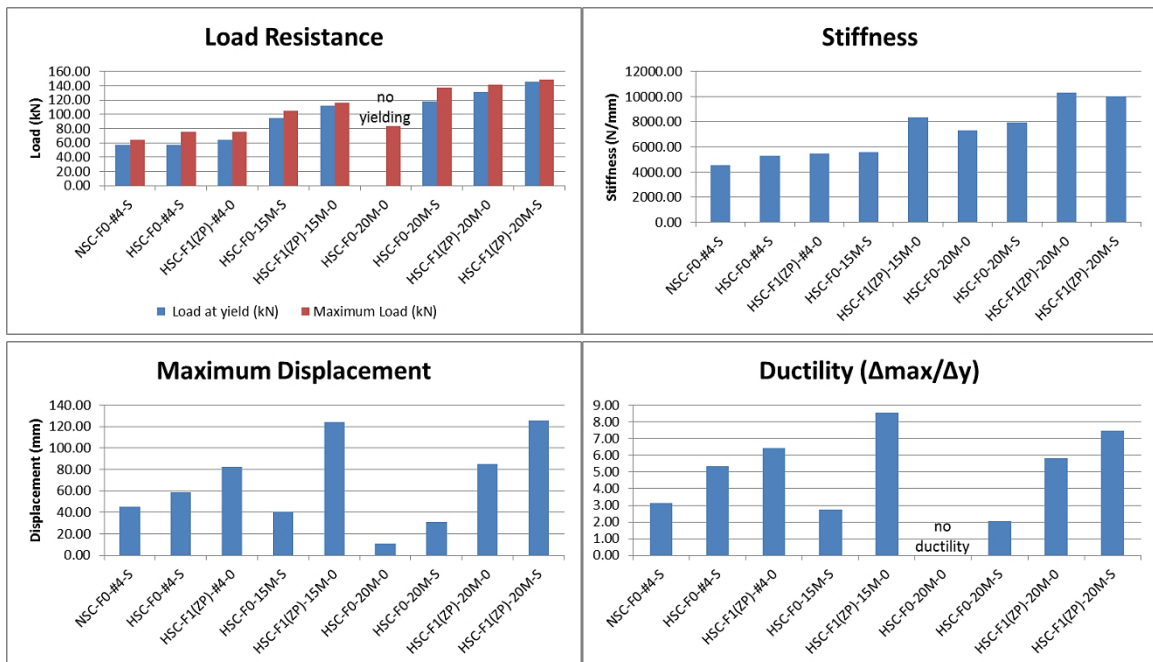


Figure 5-3 Comparative results resistance loads and displacements for each beam

5.3 Effects of Reinforcement Ratio

Based on the data obtained from the static results, the reinforcement ratio has a strong influence on the strength and ductility of the high-strength reinforced concrete beams. The effects of this test variable can be studied by examining the response of the following beams:

- HSC beams with shear reinforcement
 - HSC-F0-#4-S vs. HSC-F0-15M-S vs. HSC-F0-20M-S
- HSC with 1.0% steel fibres
 - HSC-F1(ZP)-#4-0 vs. HSC-F1(ZP)-15M-0 vs. HSC-F1(ZP)-20M-0

5.3.1 Effects of Reinforcement Ratio: HSC Beams

Specimens HSC-F0-#4-S, HSC-F0-15M-S, and HSC-F0-20M-S were all designed with high strength concrete and transverse reinforcement, but contained varying amounts of longitudinal reinforcement (with 2-#4, 2-15 and 2-20M bars, respectively). Photographs showing the beams at failure are included in Figure 5-4. The load-displacement curves of the beams and bar chart comparisons of stiffness, load resistance, maximum displacement, ductility, and toughness are provided in Figure 5-5 through Figure 5-7.

Regarding failure mode, all specimens failed in flexure with concrete crushing in the compression zone. Specimen HSC-F0-#4-S showed additional concrete spalling in the tension zone at failure as seen in Figure 5-4. Examining the load-deflection response shows that strength, stiffness and ductility were all affected by the reinforcement ratio. Overall, it can be observed that the HSC-F0-#4-S specimen performed better in terms of ductility and toughness, whereas the HSC-F0-20M-S specimen showed an improvement in initial stiffness and maximum load resistance.

The magnitude of load resistance and stiffness varied between the three beams when the reinforcement ratio increased. The maximum load resistance of the HSC-F0-20M-S specimen ($P_{\max} = 137.5$ kN) was 32% and 82% larger when compared to the loads sustained by the HSC-F0-15M-S specimen ($P_{\max} = 104.6$ kN) and HSC-F0-#4-S specimen ($P_{\max} = 75.5$ kN) respectively. In terms of stiffness, the HSC-F0-20M-S specimen (stiffness = 7911 N/mm) showed increased stiffness by 42% and 50% when compared to the HSC-F0-15M-S specimen (stiffness = 5573 N/mm) and the HSC-F0-#4-S specimen (stiffness = 5287 N/mm) respectively.

When comparing the maximum displacements of the three specimens, it can be observed that the HSC-F0-#4-S specimen recorded a larger displacement before failure ($\Delta_{\max} = 58.6$ mm) when compared to the HSC-F0-15M-S and HSC-F0-20M-S specimens ($\Delta_{\max} = 40.7$ mm and 31.0 mm) by 44% and 89%, respectively. Thus, the ductility was higher in the HSC-F0-#4-S specimen, with ductility ratio ($\Delta_{\max}/\Delta_y = 5.35$) increasing by factors of

96% and 159% when compared to the HSC-F0-15M-S and HSC-F0-20M-S specimens ($\Delta_{max}/\Delta_y = 2.74$ and 2.07, respectively).

In order to measure the toughness, the area under the load-displacement curves was calculated upto certain displacements after the yield, including $2\Delta_y$, $3\Delta_y$, $4\Delta_y$, and Δ_{max} . The largest overall toughness (upto Δ_{max}) was observed for the HSC-F0-15M-S specimen (toughness = 3272). This specimen's toughness was higher by 3% and 9% when compared to the HSC-F0-#4-S specimen (toughness = 3180) and HSC-F0-20M-S specimen (toughness = 2998). When comparing the HSC-F0-20M-S specimen (toughness = 2899) with the two companion beams at $2\Delta_y$, the result showed an increase in energy absorption by 29% when compared to beam HSC-F0-15M-S (toughness = 2255) and by 175% when compared to beam HSC-F0-#4-S (toughness = 1056). It is noted that the HSC-F0-20M-S and HSC-F0-15M-S specimens failed before reaching $3\Delta_y$ while the HSC-F0-#4-S specimen continued to carry load up to $4\Delta_y$. Based on this observation, it can be concluded that ductility and energy absorption was improved in the HSC-F0-#4-S specimen.

In summary, the results demonstrate that the response of high-strength concrete beams is affected by reinforcement ratio. Increasing the reinforcement ratio increases strength and stiffness but results in a reduction in ductility after yielding, although overall toughness remains comparable. In all cases the beams showed typical response expected for under-reinforced concrete beams.

Specimen	Comments	
	Failure mode	Mid-span Δ (mm)
HSC-F0-#4-S	Concrete crushing and spalling	58.6
		
HSC-F0-15M-S	Failure mode	Mid-span Δ (mm)
	Concrete crushing	40.7
		
HSC-F0-20M-S	Failure mode	Mid-span Δ (mm)
	Concrete crushing	31.0
		

Figure 5-4 Photographs; effects of reinforcement ratio in HSC beams without steel fibres

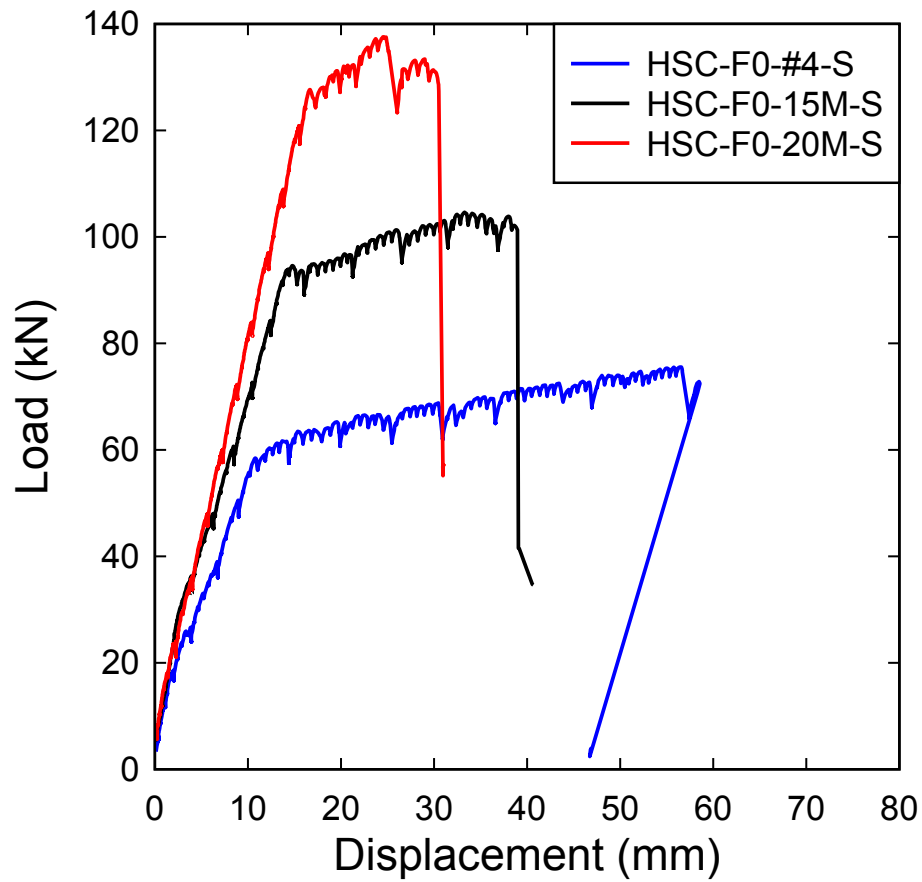


Figure 5-5 Load-displacement curve; effects of reinforcement ratio HSC beams without steel fibres

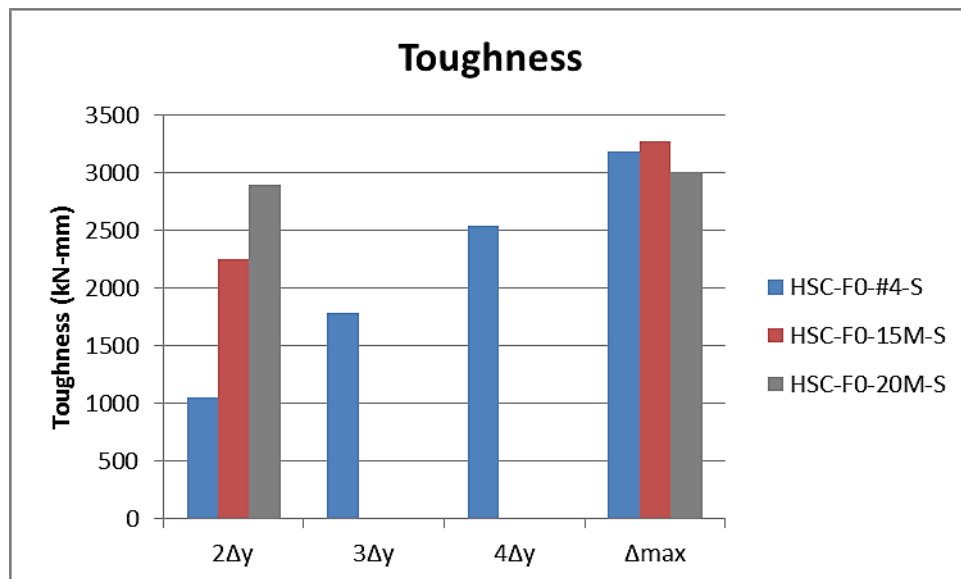


Figure 5-6 Toughness at $2\Delta_y$, $3\Delta_y$, $4\Delta_y$, and Δ_{max} ; effects of reinforcement ratio HSC beams without steel fibres

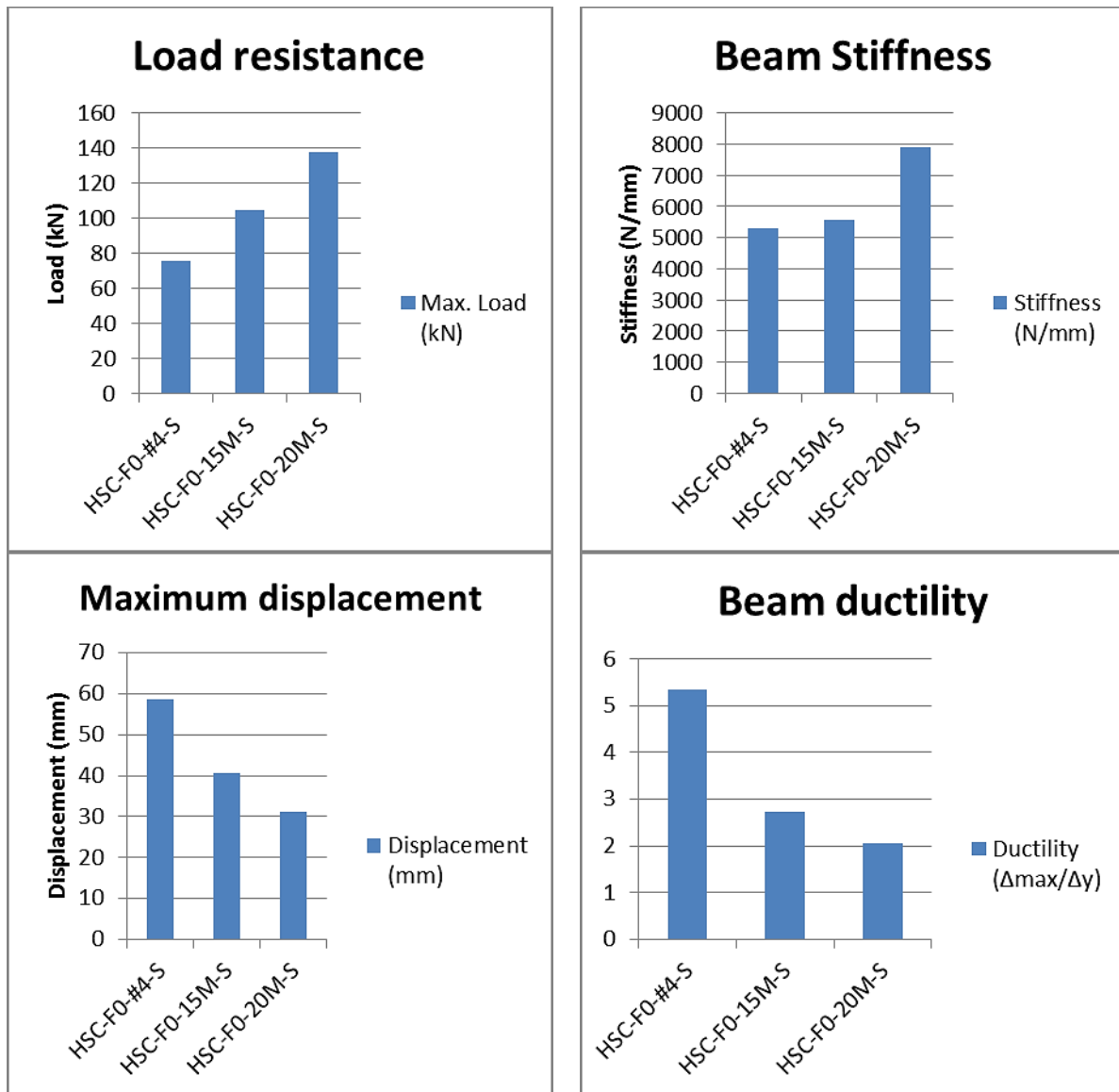


Figure 5-7 Load resistance, stiffness, ductility, and displacement; effects of reinforcement ratio in HSC beams without steel fibres

5.3.2 Effects of Reinforcement Ratio: HSC Beams with Steel Fibres

The high-strength reinforced concrete specimens HSC-F1(ZP)-#4-0, HSC-F1(ZP)-15M-0, and HSC-F1(ZP)-20M-0 were designed without transverse reinforcement and contained 1.0% of ZP steel fibres. The beams had varying amounts of longitudinal steel allowing for an examination of the effect of reinforcement ratio in HSFRC beams. Photographs showing the specimens at failure are included in Figure 5-8. The load-displacement curves and bar chart comparisons of stiffness, load resistance, maximum displacement, ductility, and toughness, are provided in Figure 5-9 through Figure 5-11.

All the HSFRC specimens failed in flexure, regardless of reinforcement ratio. Crushing of concrete on the compression face was gradual and delayed to large displacements with well distributed cracking. At larger displacement, crack opening concentrated at one major location near the mid-span of the beams. As shown in Figure 5-8, beam HSC-F1(ZP)-20M-0 showed an inclined crack in the mid-span region, which resulted in a reduction in load carrying capacity, although the beam continued to carry loading upto large displacements. Overall, it can be observed that the HSC-F1(ZP)-15M-0 specimen had better load-deflection performance in terms of ductility and toughness when compared to the remaining specimens, while the HSC-F1(ZP)-20M-0 specimen showed the largest load carrying capacity and pre-yield stiffness.

The stiffness for the HSC-F1(ZP)-20M-0 specimen (stiffness = 10314 N/mm) showed an increase of 24% and 89% when compared to the HSC-F1(ZP)-15M-0 specimen (stiffness = 8318 N/mm) and HSC-F1(ZP)-#4-0 specimen (stiffness = 5448 N/mm) respectively. Similarly, the HSC-F1(ZP)-20M-0 specimen ($P_{max} = 141.8$ kN) showed higher load carrying capacity, with ultimate resistance increased by factors of 22% and 89% when compared to the more lightly reinforced HSC-F1(ZP)-15M-0 and HSC-F1(ZP)-#4-0 specimens ($P_{max} = 115.9$ kN and 75.1 kN, respectively).

In terms of maximum displacement, the HSC-F1(ZP)-15M-0 specimen was able to sustain a larger displacement before failure with $\Delta_{max} = 124.0$ mm, an increase of 46% and 51% when comparing to the HSC-F1(ZP)-20M-0 and HSC-F1(ZP)-#4-0 specimens ($\Delta_{max} = 84.9$ mm and 82.2 mm, respectively). Similarly, the moderate amount of steel in HSC-F1(ZP)-15M-0 allowed this specimen to have larger ductility ($\Delta_{max}/\Delta_y = 8.55$) with increases of 33% and 47% when compared to the companion HSC-F1(ZP)-#4-0 ($\Delta_{max}/\Delta_y = 6.42$) and HSC-F1(ZP)-20M-0 ($\Delta_{max}/\Delta_y = 5.83$) specimens.

The HSC-F1(ZP)-15M-0 specimen also recorded the highest energy absorption upto Δ_{max} (toughness = 12888) with increases of 46% when compared to HSC-F1(ZP)-20M-0 (toughness = 8826) and 129% when compared to HSC-F1(ZP)-#4-0 (toughness = 5621). On the other hand, when toughness is measured upto $2\Delta_y$, $3\Delta_y$, and $4\Delta_y$, the HSC-F1(ZP)-20M-0

specimen (toughness = 2987 at $2\Delta_y$, 4910 at $3\Delta_y$, and 6382 at $4\Delta_y$) showed increases of 15%, 17%, and 9% when compared to HSC-F1(ZP)-15M-0 (toughness = 2592, 4211, 5843 at $2\Delta_y$, $3\Delta_y$, at $4\Delta_y$, respectively) and 105%, 105%, and 91% when compared to HSC-F1(ZP)-#4-S (toughness = 1457, 2396, 3345 at $2\Delta_y$, $3\Delta_y$, at $4\Delta_y$, respectively). However, beam HSC-F1(ZP)-20M-0 failed before reaching $6\Delta_y$, whereas the HSC-F1(ZP)-15M-0 specimen (toughness = 9122) showed an increase in toughness by 74% when compared with the HSC-F1(ZP)-#4-S specimen (toughness = 5240) when area is measured upto this displacement.

In summary, the results demonstrate that the response of HSFRC beams is affected by reinforcement ratio. As with the companion HSC beams, increasing the reinforcement ratio increases strength and stiffness. The provision of fibres allows the beams to continue to sustain large displacements even as reinforcement ratio is increased, and therefore the effect of ρ on ductility and toughness is less significant. In the current study, the ductility and toughness improved as the reinforcement was increased from 2-#4 to 2-15M bars, with a loss in ductility as the longitudinal reinforcement was further increased to 2-20M.

Specimen	Comments	
	Failure mode	Mid-span Δ (mm)
HSC-F1(ZP)-#4-0	Controlled concrete crushing with major crack in tension zone	82.1
		
HSC-F1(ZP)-15M-0	Failure mode	Mid-span Δ (mm)
	Controlled concrete crushing with major crack in tension zone	124.0
		
HSC-F1(ZP)-20M-0	Failure mode	Mid-span Δ (mm)
	Controlled concrete crushing with diagonal crack in moment region	84.9
		

Figure 5-8 Photographs; effects of reinforcement ratio in HSC beams with steel fibres

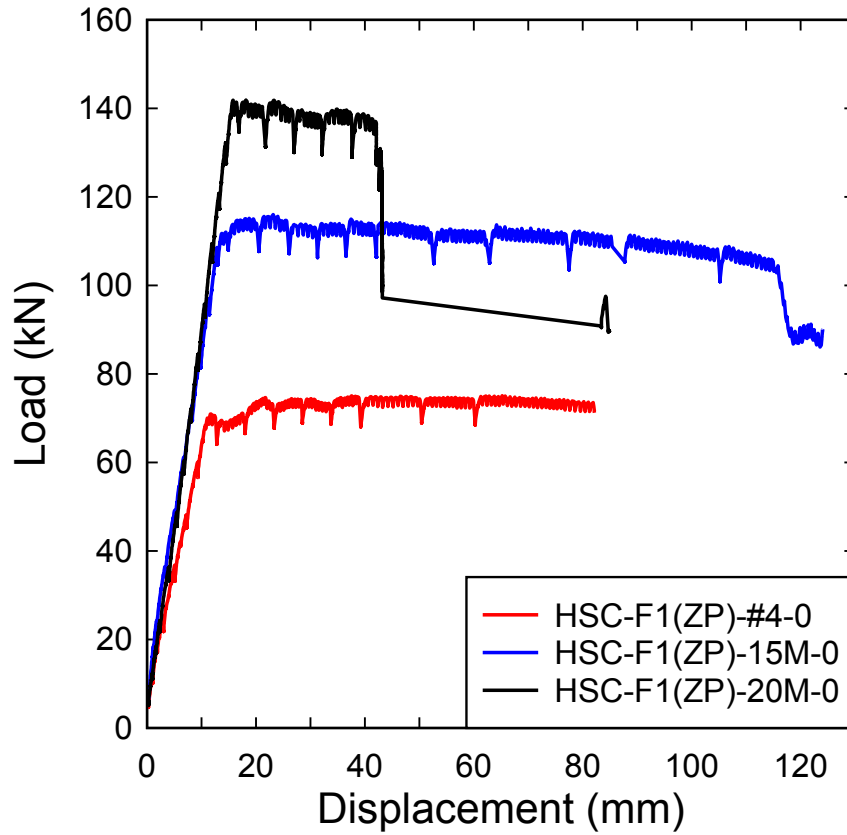


Figure 5-9 Load-displacement curve; effects of reinforcement ratio in HSC beams with steel fibres

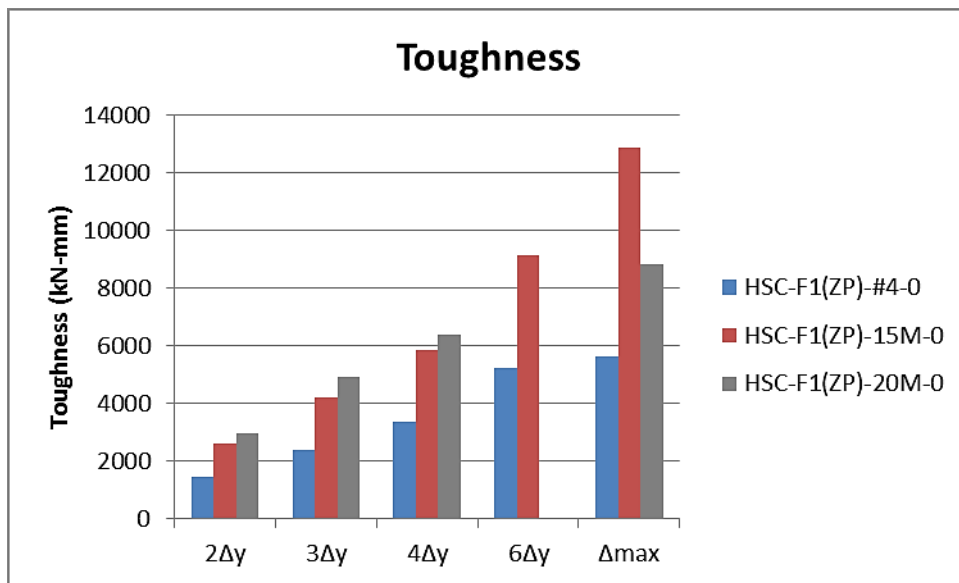


Figure 5-10 Toughness at $2\Delta y$, $3\Delta y$, $4\Delta y$, $6\Delta y$, and Δ_{max} ; effects of reinforcement ratio in HSC beams with steel fibres

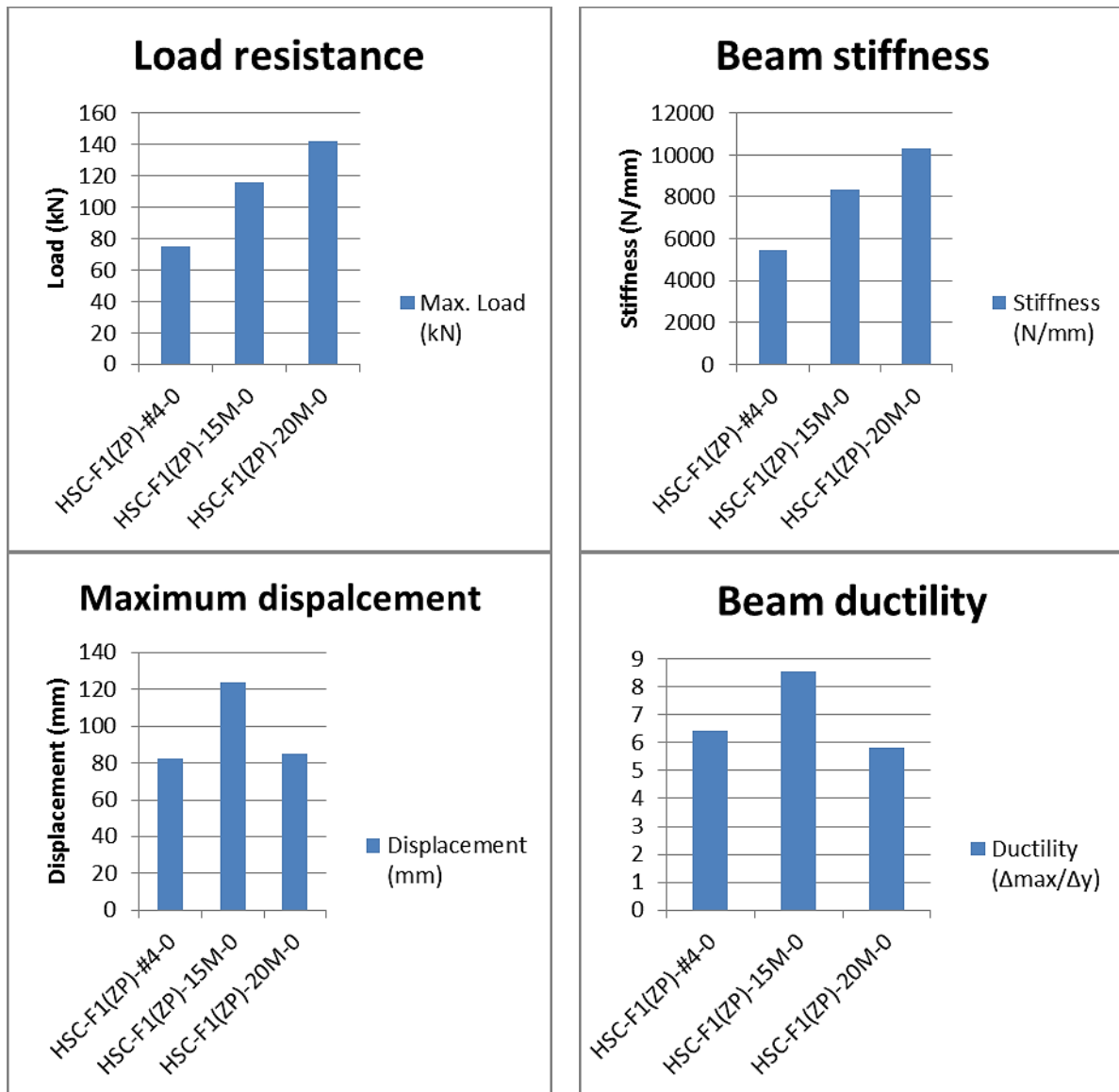


Figure 5-11 Load resistance, stiffness, ductility, and displacement; effects of reinforcement ratio in HSC beams with steel fibres

5.4 Effects of Stirrups in HSC beams

Specimens HSC-F0-20M-S and HSC-F0-20M-0 were designed with high-strength concrete and 20M bars but were designed with and without shear reinforcement, respectively. Photographs showing the failures of the specimens are included in Figure 5-12. Comparative load-displacement curves and bar charts of stiffness, load resistance, maximum displacement, ductility, and toughness are provided in Figure 5-13 to Figure 5-15.

Regarding failure mode, as expected the lack of shear reinforcement in the HSC-F0-20M-0 specimen resulted in a brittle shear failure while the provision of sufficient transverse reinforcement in HSC-F0-20M-S allowed this specimen to fail in a ductile flexure mode, with yielding of the longitudinal reinforcement and crushing of concrete in the compression region as seen in Figure 5-12. As a result, the HSC-F0-20M-S specimen showed a significant improvement in all aspects of load-deflection response when compared to the companion beam without stirrups.

The addition of transverse reinforcement improved load resistance due to the contribution of the stirrups to shear capacity. The HSC-F0-20M-S specimen sustained a load of $P_{max} = 137.5$ kN ($V = 68.8$ kN), an increase of 64% when compared to HSC-F0-20M-0 ($P_{max} = 83.8$ kN and $V = 41.9$ kN). The beam with transverse shear reinforcement also exhibited an increase in beam stiffness by 9% when compared to the companion beam which failed in shear (stiffness = 7911 N/mm vs. 7274 N/mm for HSC-F0-20M-S vs. HSC-F0-20M-0).

Owing to the change in failure mode, the beam with transverse reinforcement showed superior ductility and increased displacement before failure. The HSC-F0-20M-S sustained a displacement of $\Delta_{max} = 31.0$ mm, an increase of 191% when compared to HSC-F0-20M-0 ($\Delta_{max} = 10.7$ mm). The HSC-F0-20M-0 specimen lacked ductility due to shear failure while the HSC-F0-20M-S specimen showed ductility of $\Delta_{max}/\Delta_y = 2.07$. In terms of toughness measured upto Δ_{max} , the HSC-F0-20M-S specimen showed an increase in toughness of 537% when compared to HSC-F0-20M-0 (toughness = 2998 vs. 470).

In summary, the results confirm the importance of providing minimum web reinforcement in high-strength concrete beams in order to prevent shear failure and improve flexural response.

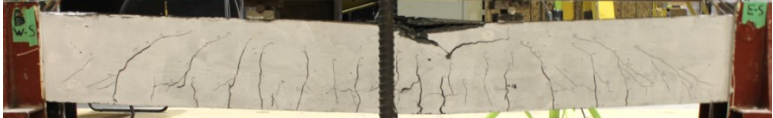
Specimen	Comments	
	Failure mode	Mid-span Δ (mm)
HSC-F0-20M-0	Shear	10.7
		
HSC-F0-20M-S	Concrete crushing	31.0
		

Figure 5-12 Photographs; effects of stirrups in HSC beams

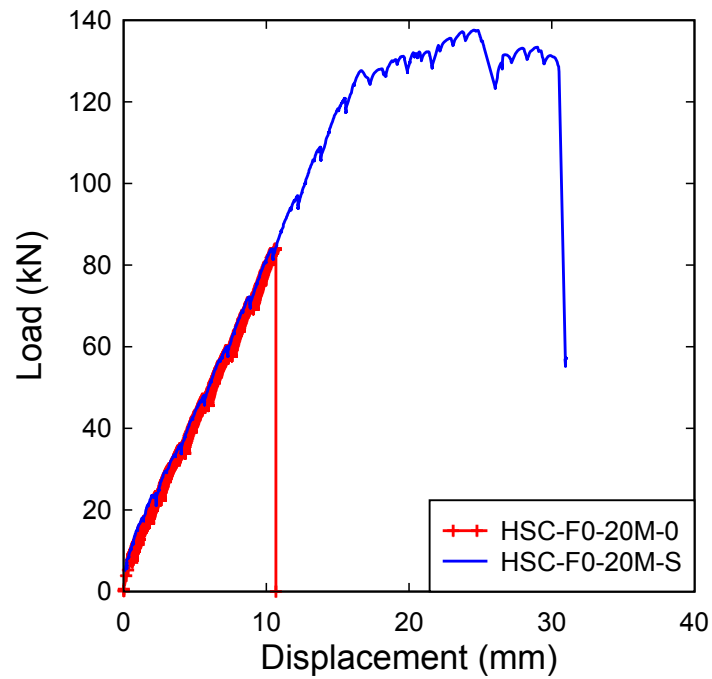


Figure 5-13 Load-displacement curve; effects of stirrups in HSC beams

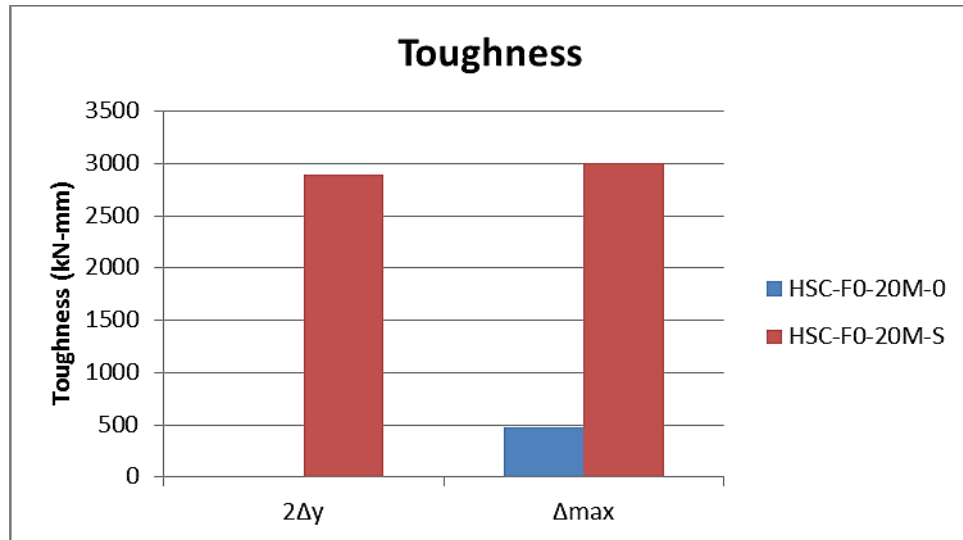


Figure 5-14 Toughness at $2\Delta_y$, $3\Delta_y$, $4\Delta_y$, and Δ_{max} ; effects of stirrups in HSC beams

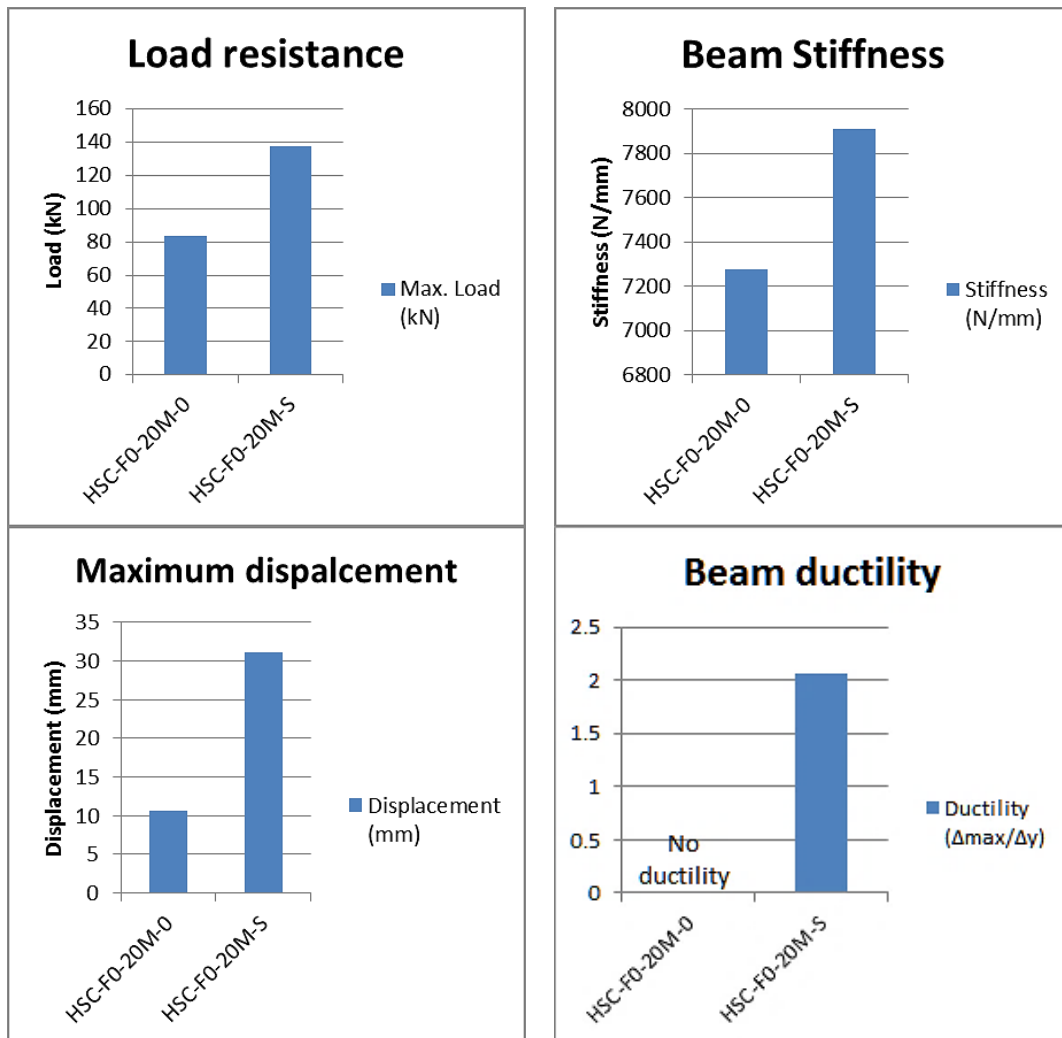


Figure 5-15 Load resistance, stiffness, ductility, and displacement; effects of stirrups in HSC beams

5.5 Effects of the Steel Fibres

The inclusion of steel fibres has shown to be beneficial with regards to the behaviour of the high-strength reinforced concrete beams. The provision of fibres reduced cracking and improved damage control. Specimens containing steel fibres also showed a boost in ductility when compared to companion beams without fibres. Finally the results show that the use of fibres increases shear resistance and allows for a replacement of transverse reinforcement. The following sub-sections provide observations with regards to the effects of steel fibres in HSC beams with and without transverse reinforcement.

5.5.1 Effects of the Steel Fibres vs. HSC Beams with Transverse Reinforcement

The high strength specimens, HSC-F0-#4-S, HSC-F0-15M-S, and HSC-F0-20M-S, were all designed with shear reinforcement arranged at a spacing of 100 mm, while companion specimens HSC-F1(ZP)-#4-0, HSC-F1(ZP)-15M-0, and HSC-F1(ZP)-20M-0 were designed without shear reinforcement and a volumetric ratio of 1.0% of steel fibres (ZP). Photographs showing the beams at failure are included in Figure 5-16. Comparative load-displacement curves and bar charts for the various load-deflection response indicators are provided in Figure 5-17 through Figure 5-19.

In regards to member response and failure mode, all specimens with and without fibres failed in flexure. The results demonstrate that fibres are able to replace transverse reinforcement in beams with varying shear demands. All HSC specimens without fibres showed concrete crushing failures in the compression zone at the mid-span, while provision of fibres delayed and controlled crushing as seen in Figure 5-16. Generally the addition of fibres resulted in a reduction of crack spacing with reduced crack widths at equivalent displacements when compared to companion HSC beams. Nonetheless, major cracking in the tension zone of the HSFRC specimens tended to concentrate at a single crack as the displacements increased. The benefit of steel fibres on flexural response is evident when comparing the load-deflection curves for companion HSC and HSFRC beams, with the fibres significantly improving ductility and toughness, with moderate effects on load carrying capacity.

Specimens containing steel fibres showed higher stiffness when compared to companion HSC beams with lateral steel reinforcement. For example Beam HSC-F1(ZP)-15M-0 had a stiffness of 8318 N/mm, representing an increase of 49% when compared to the HSC-F0-15M-S specimen (stiffness = 5573 N/mm). Similarly the provision of fibres in Beam HSC-F1(ZP)-20M-0 specimen resulted in a stiffness of 10314 N/mm; an increase of 30% when compared to HSC-F0-20M-S (stiffness = 7911 N/mm). The effect was more modest for Beam HSC-F1(ZP)-#4-0 which only showed a 3% increase in stiffness when compared to Beam HSC-F0-#4-S (stiffness = 5448 vs. 5287 N/mm). The addition of fibres had a moderate effect on maximum load resistance, with HSFRC specimens showing

increases in load resistance by factors of 11% and 3% for beams with 15M and 20M reinforcement ($P_{max} = 115.9$ kN and 104.6 kN for HSC-F1(ZP)-15M-0 and HSC-F0-15M-S; $P_{max} = 141.8$ kN and 137.5 kN for HSC-F1(ZP)-20M-0 and HSC-F0-20M-S, respectively). Countering this trend, the maximum load sustained by HSC-F0-#4-S ($P_{max} = 75.5$ kN) was slightly higher by 1% when compared to HSC-F1(ZP)-#4-0 which had steel fibres ($P_{max} = 75.1$ kN).

HSFRC specimens sustained larger displacements before failure when compared to companion HSC specimens owing to the improved compressive and tensile behaviour of HSFRC. When comparing to the HSC specimens, beams with steel fibres showed increases in maximum displacement by 40% for HSC-F1(ZP)-#4-0 ($\Delta_{max} = 82.2$ mm vs. 58.6 mm), by 205% for HSC-F1(ZP)-15M-0 ($\Delta_{max} = 124.0$ mm vs. 40.7 mm) and by 174% for HSC-F1(ZP)-20M-0 ($\Delta_{max} = 84.9$ mm vs. 31.0 mm) when compared to beams HSC-F0-#4-S, HSC-F0-15M-S and HSC-F0-20M-S, respectively. Because the HSFRC beams had higher maximum displacements, ductility was also significantly higher when compared to companion HSC beams. For the #4 series, Beam HSC-F1(ZP)-#4-0 ($\Delta_{max}/\Delta_y = 6.42$) performed better when compared to HSC-F0-#4-S ($\Delta_{max}/\Delta_y = 5.35$) with ductility increased by 20%. Similarly, the HSC-F1(ZP)-15M-0 specimen ($\Delta_{max}/\Delta_y = 8.55$) showed increase in ductility by 212% in comparison to HSC-F0-15M-S ($\Delta_{max}/\Delta_y = 2.74$). Finally, the HSC-F1(ZP)-20M-0 specimen ($\Delta_{max}/\Delta_y = 5.83$) had an increase in ductility by a factor 182% compared to HSC-F0-20M-S ($\Delta_{max}/\Delta_y = 2.07$).

Similarly, the HSFRC specimens showed superior toughness when compared to the HSC specimens. When toughness was calculated to maximum displacement Δ_{max} , specimens HSC-F1(ZP)-#4-0 (toughness = 5621 vs. 3180), HSC-F1(ZP)-15M-0 (toughness = 12888 vs. 3272), and HSC-F1(ZP)-20M-0 (toughness = 8827 vs. 2998) absorbed more energy when comparing to the companion beams with HSC, with improvements of 77%, 294% and 194% when comparing to specimens HSC-F0-#4-S, HSC-F0-15M-S and HSC-F0-20M-S. When comparing toughness upto equivalent displacement ratio of $2\Delta_y$, HSC-F1(ZP)-#4-0 (toughness = 1457 vs. 1056), HSC-F1(ZP)-15M-0 (toughness = 2592 vs. 2255), and HSC-F1(ZP)-20M-0 (toughness = 2987 vs. 2899) showed increases in toughness by 38%, 15% and 3% compared to specimens HSC-F0-#4-S, HSC-F0-15M-S and HSC-F0-20M-S. Beams HSC-F0-20M-S and HSC-F0-15M-S failed before reaching $3\Delta_y$ while the companion HSFRC beams continued to carry load beyond these displacement ratios. The HSC-F0-#4-S specimen which absorbed energy up to $4\Delta_y$, showed a reduction in toughness by 26% at $3\Delta_y$ and 24% at $4\Delta_y$ when compared with HSC-F1(ZP)-#4-0. The HSFRC specimens continued to carry load up to $6\Delta_y$, except for the HSC-F1(ZP)-20M-0 specimen which failed after $4\Delta_y$. Based on these results, it is evident that the steel fibres significantly enhanced the toughness and ductility of the high-strength reinforced concrete beams.

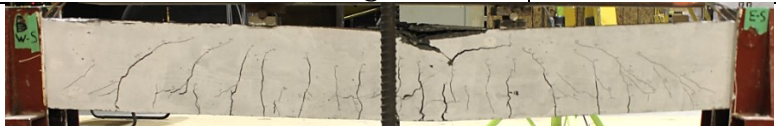
Specimen	Comments	
	Failure mode	Mid-span Δ (mm)
HSC-F0-#4-S	Concrete crushing and spalling	58.6
		
HSC-F1(ZP)-#4-0	Failure mode	Mid-span Δ (mm)
	Controlled concrete crushing with major crack in tension zone	82.2
		
HSC-F0-15M-S	Failure mode	Mid-span Δ (mm)
	Concrete crushing	40.7
		
HSC-F1(ZP)-15M-0	Failure mode	Mid-span Δ (mm)
	Controlled concrete crushing with major crack in tension zone	124.0
		
HSC-F0-20M-S	Failure mode	Mid-span Δ (mm)
	Concrete crushing	31.0
		
HSC-F1(ZP)-20M-0	Failure mode	Mid-span Δ (mm)
	Controlled concrete crushing with diagonal crack in moment region	84.9
		

Figure 5-16 Photographs; effects of steel fibres vs. stirrups in HSC beams

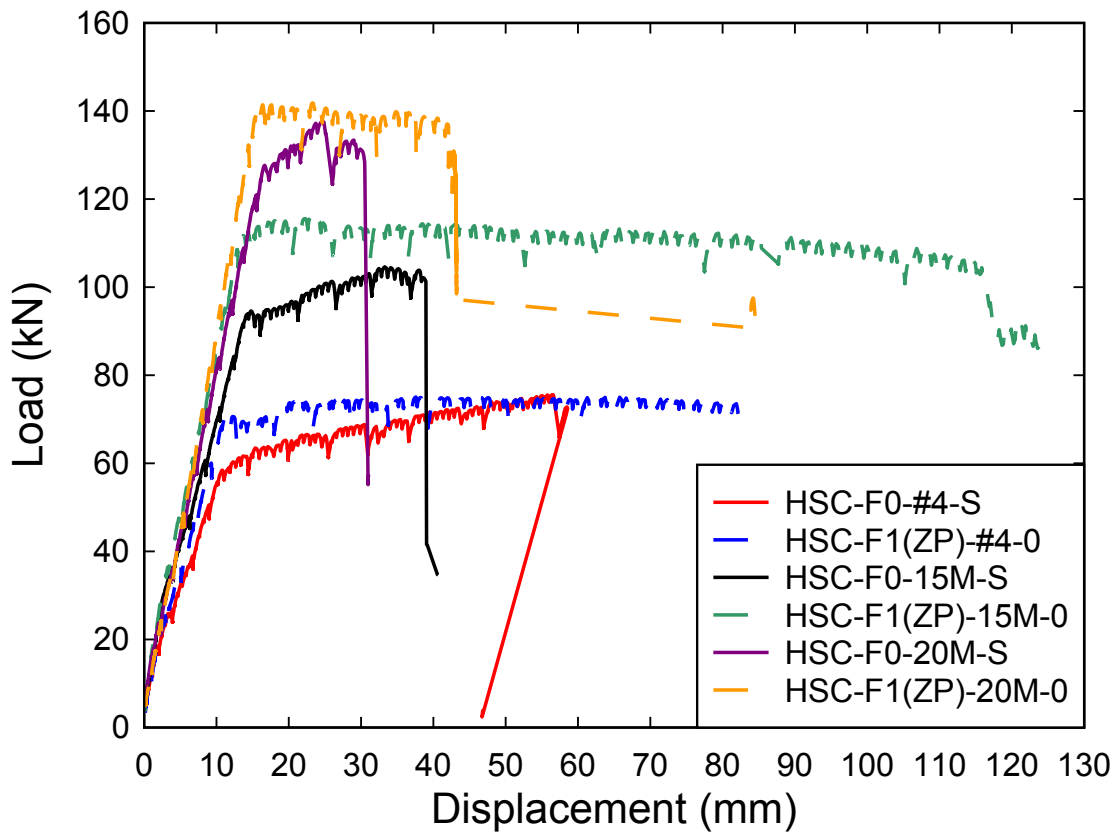


Figure 5-17 Load-displacement curve; effects of steel fibres vs. stirrups in HSC beams

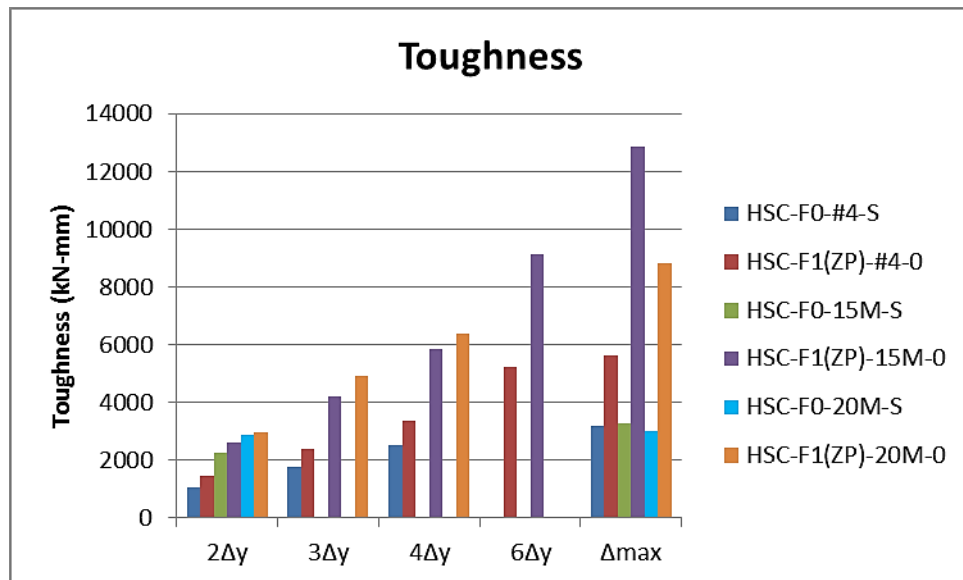


Figure 5-18 Toughness at $2\Delta_y$, $3\Delta_y$, $4\Delta_y$, $6\Delta_y$, and Δ_{max} ; effects of steel fibres vs. stirrups in HSC beams

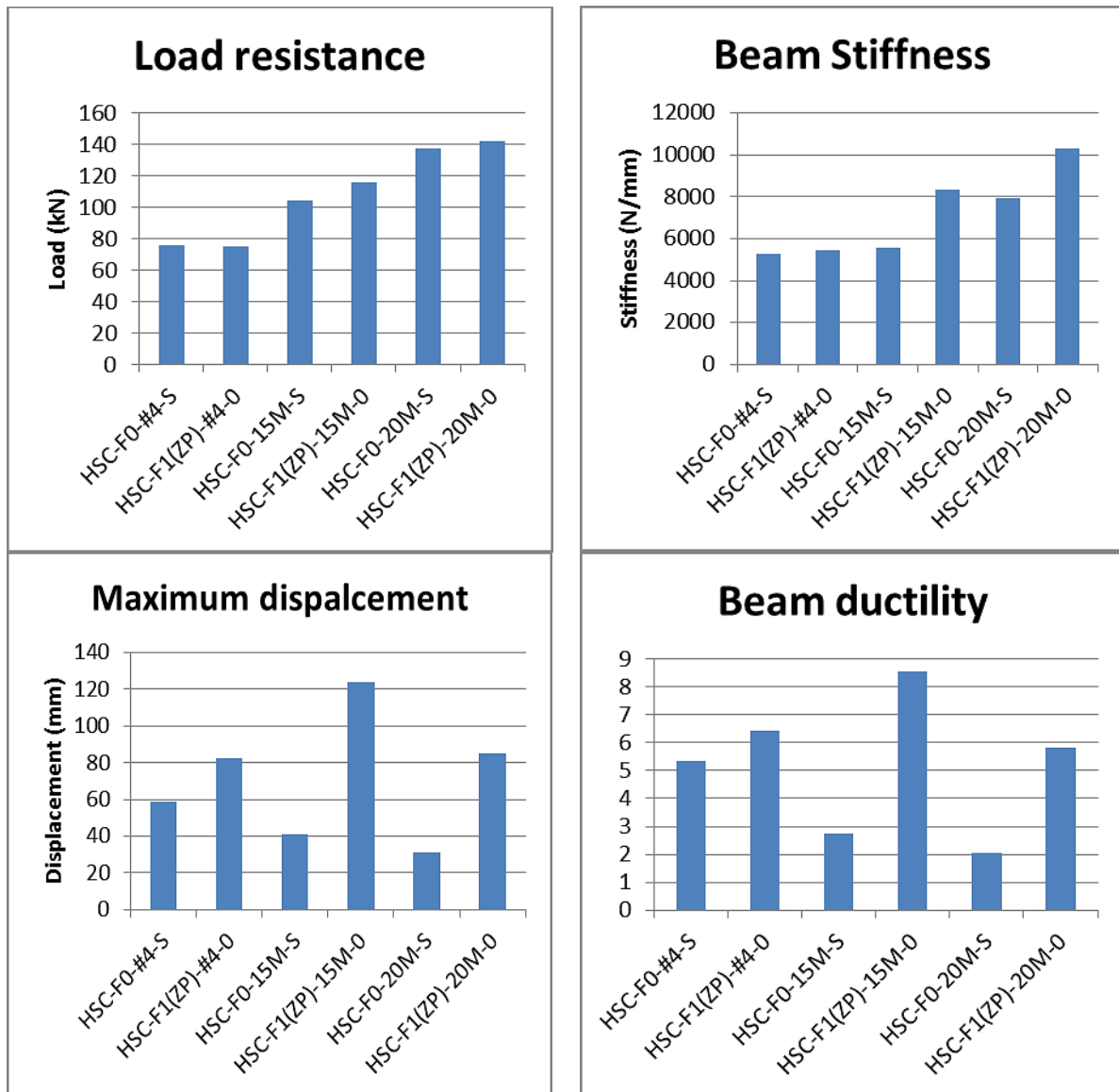


Figure 5-19 Load resistance, stiffness, ductility, and displacement; effects of steel fibres vs. stirrups in HSC beams

5.5.2 Effects of the Steel Fibres vs. HSC Beam without Transverse Reinforcement

The high strength specimens, HSC-F0-20M-0 and HSC-F1(ZP)-20M-0, were designed without transverse reinforcement, with steel fibres added at a volumetric ratio of 1.0% in the latter specimen. A comparison of the response of the specimens allows for an investigation on the effect of steel fibres in increasing shear resistance. Photographs showing the failure of the beams are included in Figure 5-20. Comparative load-displacement curves and bar charts of stiffness, load resistance, maximum displacement, ductility, and toughness, are provided in Figure 5-21 to Figure 5-23.

As expected the lack of shear reinforcement in the plain high-strength concrete HSC-F0-20M-0 specimen resulted in a brittle shear failure. In contrast, the provision of steel fibres in specimen HSC-F1(ZP)-20M-0 allowed the beam to fail in a ductile flexure failure mode as seen in Figure 5-20.

Load resistance and shear resistance in the HSC-F1(ZP)-20M-0 specimen was higher when compared to the HSC-F0-20M-0 specimen. The HSC-F1(ZP)-20M-0 specimen sustained a maximum Load of 141.8 kN ($V = 71$ kN), an increase in capacity of 69% when compared to HSC-F0-20M-0 ($P_{\max} = 83.8$ kN and $V = 42$ kN). Moreover, the HSC-F1(ZP)-20M-0 specimen (stiffness = 10314 N/mm) showed an increase in stiffness by 42% when compared to the HSC-F0-20M-0 specimen (stiffness = 7274 N/mm). More importantly, the increased shear resistance allowed the beam to reach its full flexural capacity allowing the development of flexural yielding in the longitudinal steel reinforcement.

Moreover, the addition steel fibres greatly increased maximum displacement with $\Delta_{\max} = 84.9$ mm for Beam HSC-F1(ZP)-20M-0, an improvement of 695% when compared to the HSC-F0-20M-0 specimen ($\Delta_{\max} = 10.7$ mm). As a result, the HSC-F1(ZP)-20M-0 specimen recorded significantly larger ductility ($\Delta_{\max}/\Delta_y = 5.83$) when compared to HSC-F0-20M-0 which showed no ductility due to brittle shear failure.

The addition of steel fibre reinforcement also boosted beam toughness when compared to the HSC beam. At $\Delta_{\max}$, the HSC-F1(ZP)-20M-0 specimen had toughness = 8827, an increase by a factor of 17.76 times when compared to HSC-F0-20M-0 (toughness = 470).

In summary, the results show that the addition of a sufficient quantity of steel fibres increases shear resistance of high-strength concrete beams, effectively allowing for the replacement of transverse steel reinforcement.

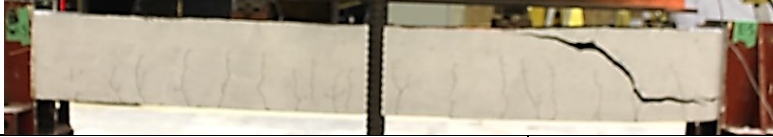
Specimen	Comments	
HSC-F0-20M-0	Failure mode	Mid-span Δ (mm)
	Shear	10.7
		
HSC-F1(ZP)-20M-0	Failure mode	Mid-span Δ (mm)
	Controlled concrete crushing with diagonal crack in moment region	84.9
		

Figure 5-20 Photographs; effect of steel fibres vs. HSC beam without stirrups

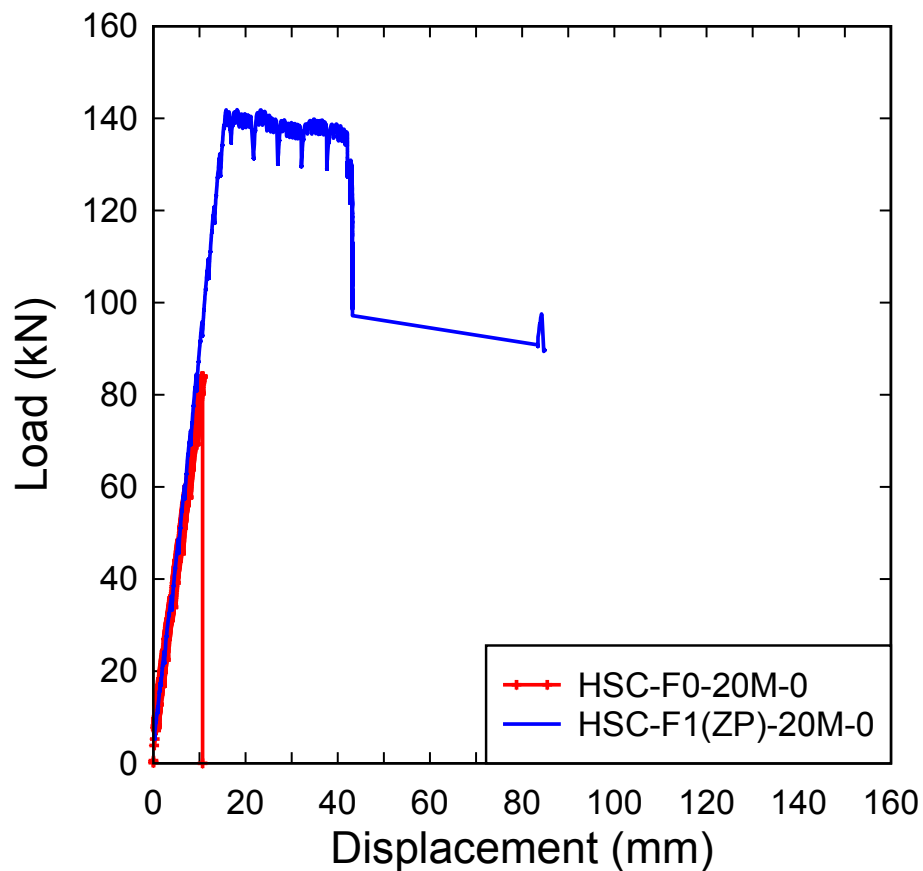


Figure 5-21 Load-displacement curves; effect of steel fibres vs. HSC beam without stirrups

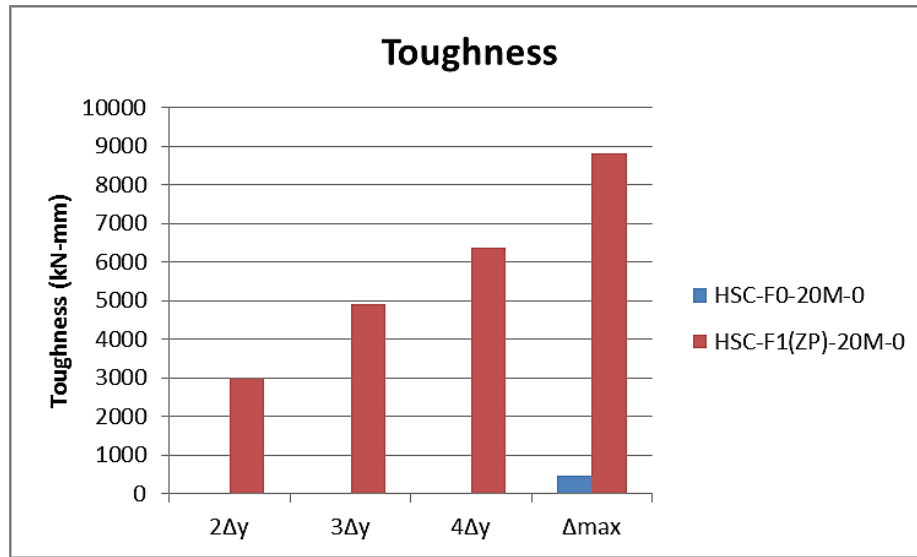


Figure 5-22 Ductility at $2\Delta_y$, $3\Delta_y$, $4\Delta_y$, and Δ_{max} ; effects of steel fibres vs. HSC beam without stirrups

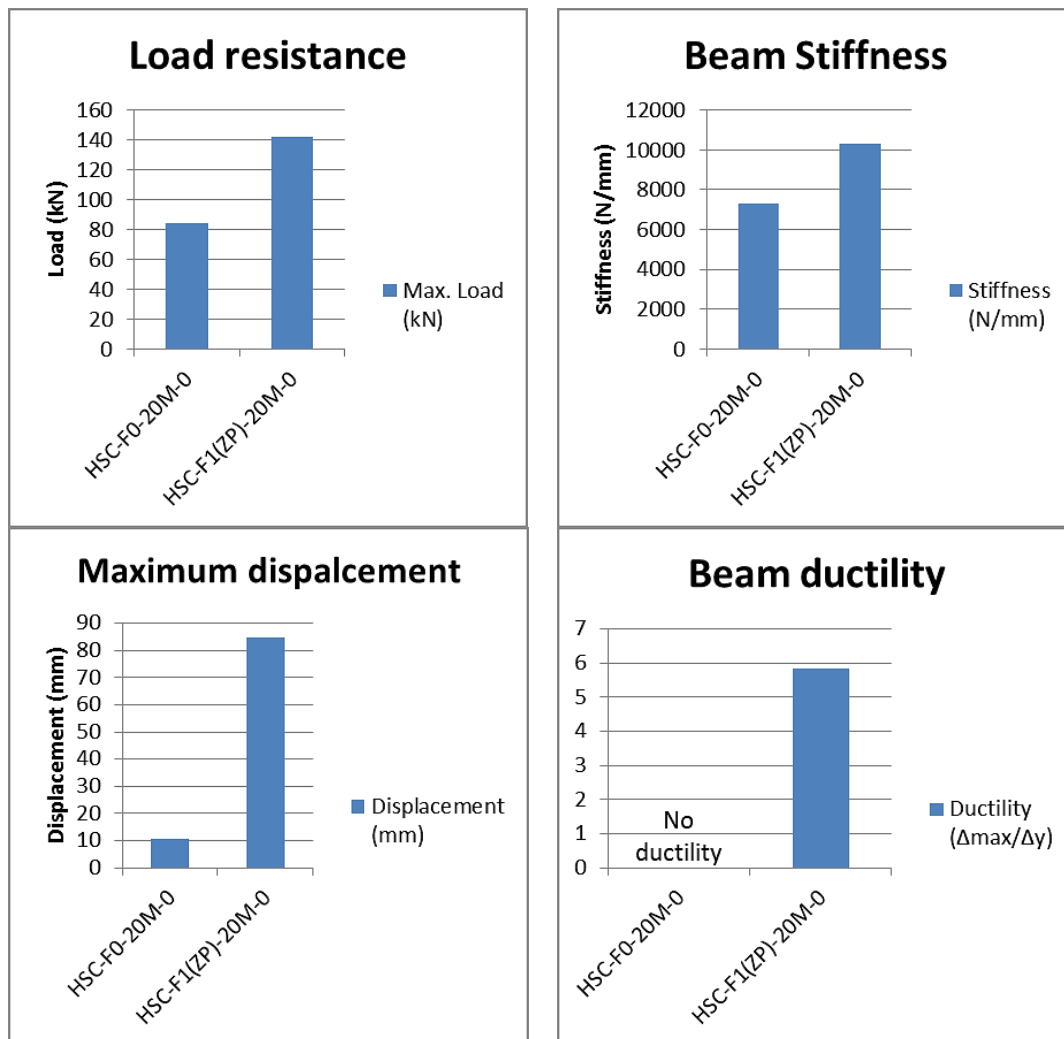


Figure 5-23 Load resistance, stiffness, ductility, and displacement; effects of steel fibres vs. HSC beam without stirrups

5.6 Effect of Combined Use of Stirrups and Fibres in HSC Beams

The previous sections have shown that the provision of steel fibres in high-strength concrete increases shear capacity and improves beam ductility and toughness. This section investigates the effect of combining steel fibres and transverse reinforcement in high-strength concrete beams. Specimens HSC-F1(ZP)-20M-S and HSC-F1(ZP)-20M-0 were both constructed with HSFRC with 1.0% steel fibres, 20M reinforcement but were detailed with and without stirrups spaced at 100 mm. Photographs showing the failures of the specimens are included in Figure 5-24. Comparative load-displacement curves and bar charts showing the stiffness, load resistance, maximum displacement, ductility, and toughness of the beams are provided in Figure 5-25 to Figure 5-27.

In terms of failure mode, both specimens failed in flexure with yielding of longitudinal steel and showed an ability to sustain significantly large displacements as can be clearly seen in Figure 5-24. The condition of the beams at failure in terms of crack distribution is similar. When examining overall response, both beams showed similar behaviour upto a displacement of ~ 45 mm, however, the combined use of fibres and lateral reinforcement resulted in an enhanced response for specimen HSC-F1(ZP)-20M-S beyond this point, with an improvement in load carrying capacity and ductility when compared to HSC-F1(ZP)-20M-0.

Specimen HSC-F1(ZP)-20M-S showed an increase in load resistance by 5% when compared to HSC-F1(ZP)-20M-0 ($P_{max} = 148.4$ vs. 141.8 kN). However, the stiffness for Beam HSC-F1(ZP)-20M-S dropped slightly by 3% compared to the HSC-F(ZP)-20M-0 specimen (stiffness = 10032 vs. 10314 N/mm).

The presence of transverse shear reinforcement significantly improved the ductility of the fibre-reinforced concrete beam. Specimen HSC-F1(ZP)-20M-S sustained maximum displacement of $\Delta_{max} = 125.9$ mm with ductility $\Delta_{max}/\Delta_y = 7.46$ increases of 48% and 28% when compared to HSC-F1(ZP)-20M-0 ($\Delta_{max} = 84.9$ mm and $\Delta_{max}/\Delta_y = 5.83$).

Similarly, the provision of shear reinforcement improved the toughness. When computed upto Δ_{max} , the HSC-F1(ZP)-20M-S specimen showed an increase in toughness by 55% compared to the HSC-F1(ZP)-20M-0 specimen (toughness = 13676 vs. 8827). Similarly, when toughness is computed upto $2\Delta_y$, the HSC-F1(ZP)-20M-S specimen showed an improvement in toughness by 23% compared to HSC-F1(ZP)-20M-0 (toughness = 3668 vs. 2987) and 18% for $3\Delta_y$ (toughness = 5786 vs. 4910) and 21% for $4\Delta_y$ (toughness 7743 vs. 6382).

In summary, the results demonstrate some improvements in the response of high-strength concrete beams when steel fibres are combined with transverse reinforcement. These improvements are more obvious at larger beam displacements.

Specimen	Comments	
	Failure mode	Mid-span Δ (mm)
HSC-F1(ZP)-20M-0	Controlled concrete crushing with diagonal crack in moment region	84.9
		
HSC-F1(ZP)-20M-S	Failure mode	Mid-span Δ (mm)
	Controlled concrete crushing with diagonal crack in moment region	125.9
		

Figure 5-24 Photographs; effects of combined use of stirrups and fibres in HSC beams

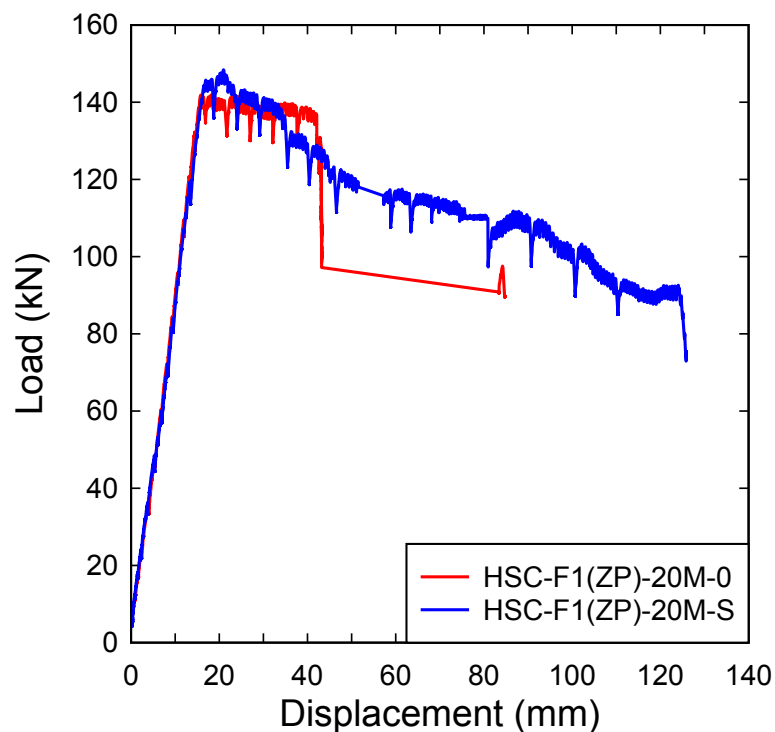


Figure 5-25 Load-displacement curve; effects of combined use of stirrups and fibres in HSC beams

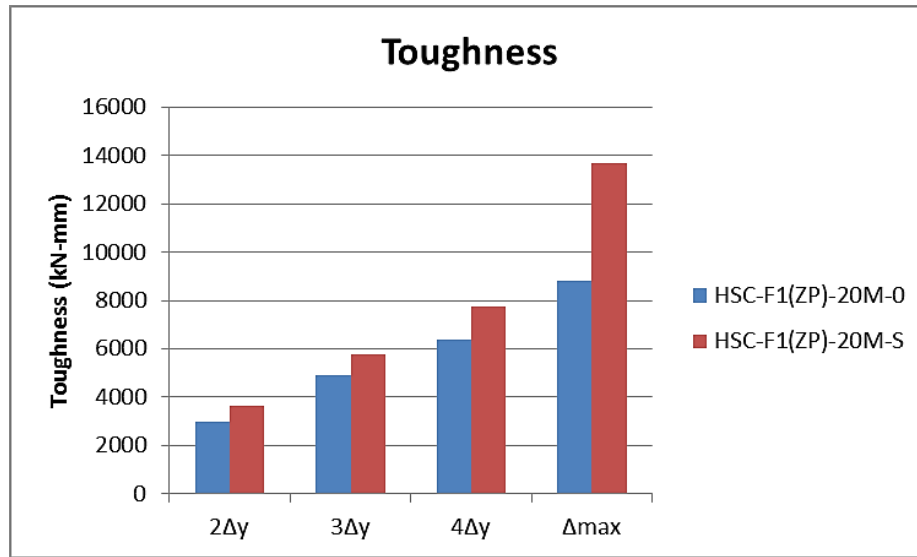


Figure 5-26 Toughness at $2\Delta y$, $3\Delta y$, $4\Delta y$, and Δ_{max} ; effects of combined use of stirrups and fibres in HSC beams

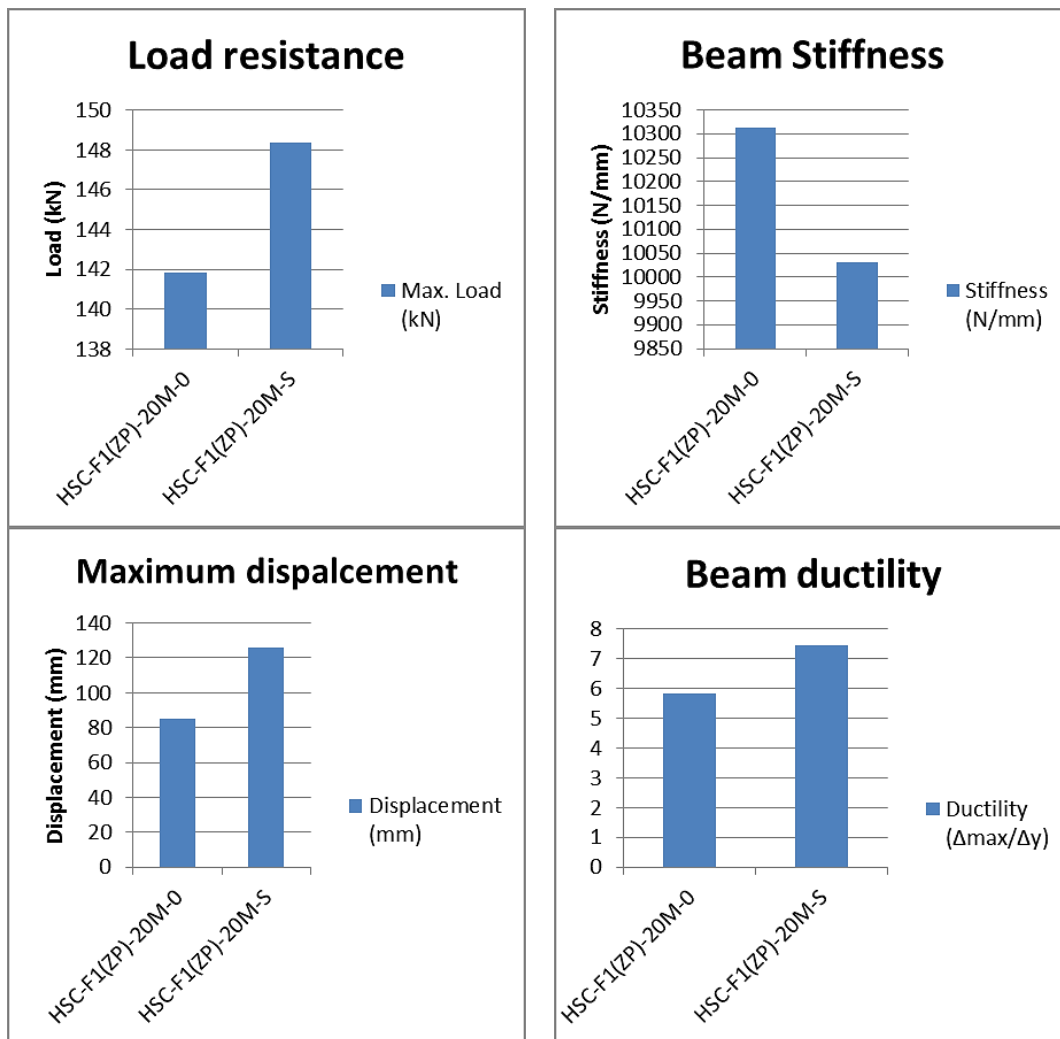


Figure 5-27 Load resistance, stiffness, ductility, and displacement; effects of combined use of stirrups and fibres in HSC beams

5.7 Effects of Concrete Strength

This section examines the effect of concrete strength on the behaviour of the plain reinforced concrete specimens tested in this research.

Specimens NSC-F0-#4-S and HSC-F0-#4-S had identical properties (2-#4 bars, stirrups at 100 mm) but were constructed with normal-strength self-consolidated concrete and high-strength concrete, respectively. Photographs showing the failures of the beams are included in Figure 5-28. Comparative load-displacement curves and bar charts comparing the stiffness, load resistance, maximum displacement, ductility and toughness, of the beams are provided in Figure 5-29 to Figure 5-31.

In terms of member response and failure mode, both specimens failed in flexure with concrete crushing in the compression region and spalling of concrete in the tension region at mid-span as seen in Figure 5-28. When comparing the load-deflection response, it is observed that high-strength concrete increased the toughness and overall beam performance.

The use of high strength concrete also resulted in appreciable increase in load resistance and stiffness. The HSC-F0-#4-S specimen sustained a maximum load $P_{max} = 75.5$ kN, which represents an increase in load capacity of 18% when compared to the NSC-F0-#4-S specimen ($P_{max} = 64.2$ kN). In terms of stiffness, the HSC-F0-#4-S specimen showed a stiffness which was 17% higher than that of Beam NSC-F0-#4-S specimen (stiffness = 5287 vs. 4529 N/mm).

Counter to what would be expected, the HSC beam showed higher maximum displacement and ductility. The HSC-F0-#4-S specimen sustained a maximum displacement of $\Delta_{max} = 58.6$ mm, an improvement of 30% when compared to the NSC specimen ($\Delta_{max} = 45.1$ mm). Similarly, the HSC-F0-#4-S specimen had an increase of 71% in ductility when compared to NSC-F0-#4-S specimen ($\Delta_{max}/\Delta_y = 5.35$ vs. 3.13).

The beam cast with normal-strength concrete showed higher toughness at $2\Delta_y$ and $3\Delta_y$, but it failed before it reached $4\Delta_y$, whereas the high strength concrete was still carrying load and able to absorb energy up to $4\Delta_y$. When comparing toughness upto Δ_{max} , the HSC-F0-#4-S specimen showed an increase of 39% when compared to NSC-F0-#4-S specimen (toughness = 3180 vs. 2286).

In summary, the results of this comparison show that the use of high-strength concrete improved the load carrying capacity, the initial stiffness and the overall toughness of the beams tested in this research program.

Specimen	Comments	
	Failure mode	Mid-span Δ (mm)
NSC-F0-#4-S	Concrete crushing and spalling	45.1
		
HSC-F0-#4-S	Failure mode	Mid-span Δ (mm)
	Concrete crushing and spalling	58.6
		

Figure 5-28 Photographs; effects of concrete strength

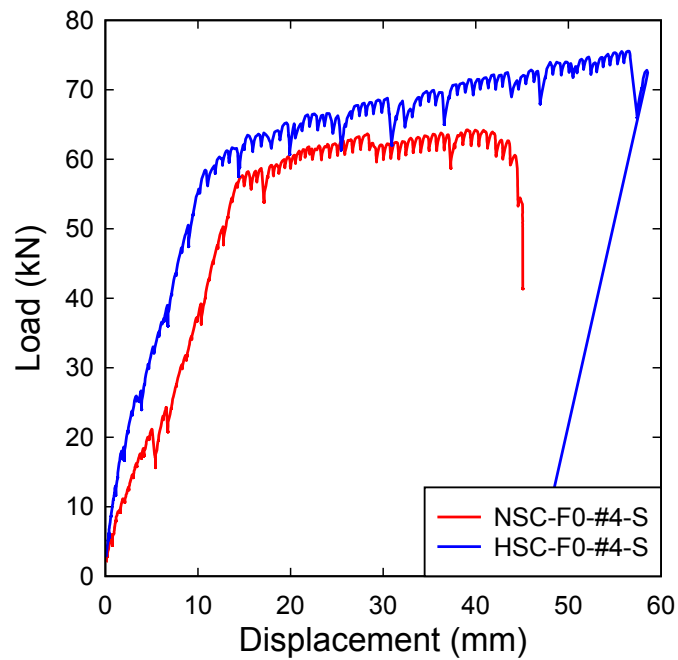


Figure 5-29 Load-displacement curve; effects of concrete strength

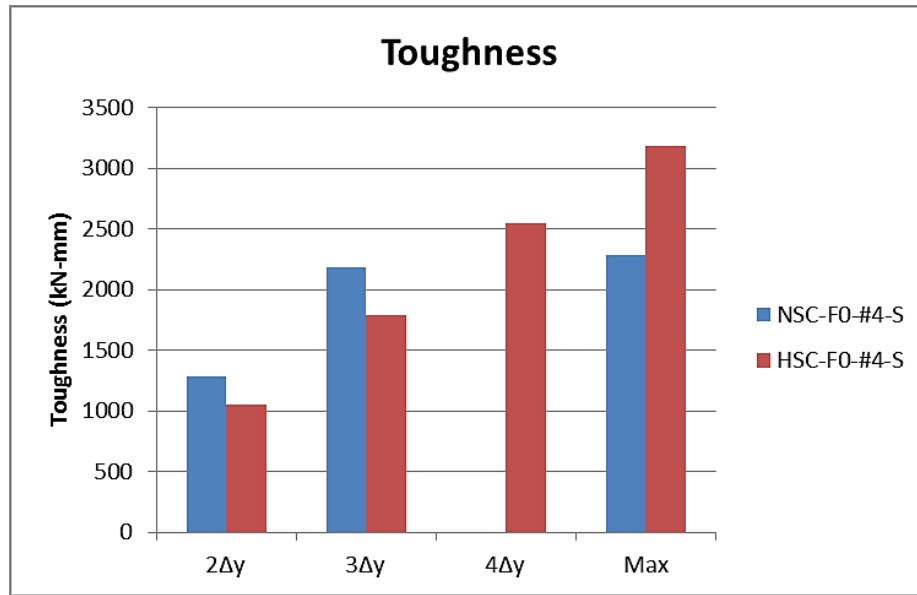


Figure 5-30 Toughness at $2\Delta_y$, $3\Delta_y$, $4\Delta_y$, and Δ_{max} ; effects of concrete strength

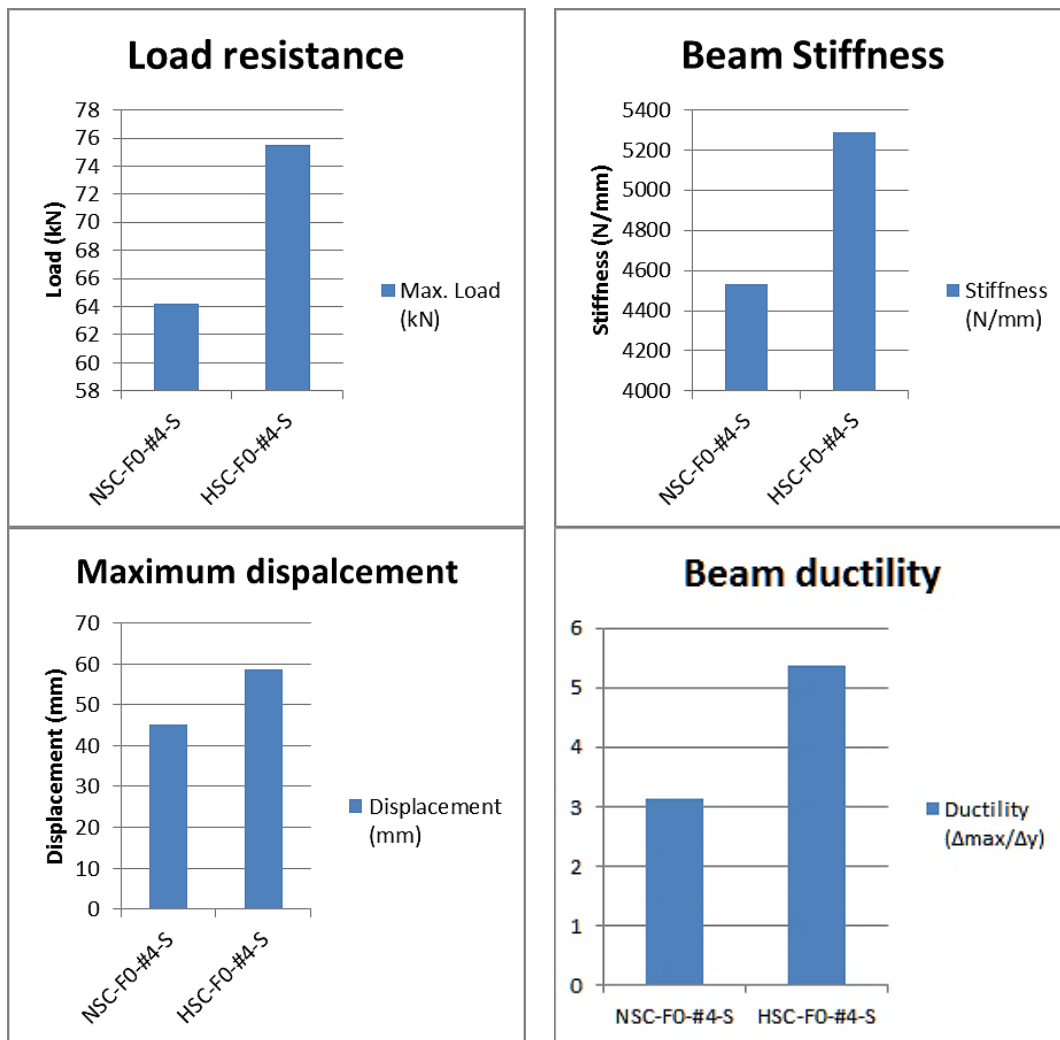


Figure 5-31 Load resistance, stiffness, ductility, and displacement; effects of concrete strength

Chapter 6: Experimental Results of the Shock-Tube Tests

6.1 Chapter Overview

This chapter outlines the experimental results of the dynamic testing of eleven reinforced concrete beams in the shock-tube. The results for each specimen are presented in terms of the blast test parameters, maximum and residual mid-span displacements, blast and impulse time-histories, displacement time histories and strains in the steel reinforcement. The damage progression and failure during testing are described along with photographs of the specimens after each test. Overall, the chapter is divided into four sections. Section 6.2 provides a summary of the experimental results in tabular form. Sections 6.3, 6.4 and 6.5 describe the experimental results for series #4, series 15M and series 20M specimens, respectively. Section 6.6 presents the pressure-impulse and displacement time histories in graphs along with photographs showing the damage progression and failure of the specimens at each blast load level.

6.2 Summary of Experimental Results

The experimental results for series #4, 15M and 20M are summarized in Table 6-1 and Table 6-2. The tables summarize the shockwave properties of each test, including the approximate driver pressure in psi, the reflected pressure (P_r), positive phase duration (t_p) and reflected impulse (I_r). Also included are the response of the specimens in terms of maximum and residual mid-span displacements (δ_{max} and δ_{res}), and support rotations (θ_{max}) after each test.

The reflected pressure values were recorded from a pressure transducer located at the bottom of the shock-tube end test frame. This transducer accurately measured the pressure acting on the load transfer device (LTD). For reflected impulse, the pressure-time history curve for the reflected pressure was integrated over the positive phase using the software *DPlot* (HydeSoft Computing LLC) to obtain the reflected impulse. It should be noted that the mid-span displacements and support rotation values in Table 6-1 and Table 6-2 are not cumulative.

Table 6-1 Blast data summary (1/2)

Beams	Shockwave properties				Displacement		$\theta_{\max}$	Failure Mode
ID	Shot	I_r	P_r	t_p	$\delta_{\max}$	δ_{res}		
	(psi)	(kPa-ms)	(kPa)	(ms)	(mm)	(mm)	degree	
NSC-F0-#4-S	17	245.00	23.31	21.02	12.65	1.54	0.65	concrete spalling
	30	373.00	40.82	18.28	29.53	13.04	1.52	
	40	448.50	45.24	19.83	47.96	25.11	2.46	
HSC-F0-#4-S	17	229.00	24.42	18.76	13.15	3.62	0.64	concrete spalling
	30	348.00	38.24	18.20	30.44	14.16	1.56	
	40	421.00	44.99	18.72	44.53	6.59	2.29	
HSC-F1(ZP)-#4-0	17	216.60	24.29	17.83	10.63	3.71	0.55	concrete splitting/ fibre pullout
	30	344.00	41.50	16.58	18.47	4.10	0.95	
	40	428.95	48.21	17.80	35.18	15.61	1.81	
	50	512.00	55.50	18.45	55.10	32.26	2.83	
	60	562.62	68.75	16.37	61.0	38.89	3.13	
HSC-F0-15M-S	17	223.00	23.89	18.67	11.50	2.66	0.59	Concrete Crushing
	30	340.70	42.88	15.89	21.40	4.71	1.10	
	50	516.00	58.64	17.60	124.00	22.04	6.41	
HSC-F1(ZP)-15M-0	17	213.70	24.49	17.45	10.05	3.91	0.52	Concrete splitting/ fibre pullout
	30	347.60	40.28	17.26	15.02	2.66	0.77	
	50	512.00	56.51	18.12	33.31	10.08	1.71	
	70	657.00	69.48	18.91	78.91	54.68	4.05	
HSC-F0-20M-0	17	239.53	23.93	20.02	12.55	3.37	0.64	Shear
	30	370.80	38.70	19.16	19.31	9.21	0.99	
HSC-F0-20M-S	17	244.28	23.58	20.72	10.44	1.98	0.54	Concrete Crushing
	30	360.00	39.22	18.36	15.12	0.15	0.78	
	50	538.18	57.40	18.75	32.91	12.43	1.69	
	70	702.59	68.83	20.42	118.06	71.69	6.06	
HSC-F1(ZP)-20M-0	17	244.0	23.4	20.85	8.44	1.4	0.43	Concrete splitting / fibre pullout
	30	387.0	41.8	18.53	14.69	3.43	0.75	
	50	571.2	57.1	20.01	20.82	4.07	1.07	
	70	734.5	69.6	21.11	42.06	19.27	2.16	
	80	738.0	75.3	19.61	62.4	37.36	3.20	
HSC-F1(ZP)-20M-S	17	260.4	23.8	21.87	9.21	2.29	0.47	Concrete Crushing
	30	358.0	41.4	17.30	14.91	2.2	0.77	
	50	552.0	57.4	19.24	27.64	4.04	1.42	
	70	732.7	71.1	20.60	67.03	41.84	3.44	
	80	774.0	79.1	19.56	153.01	79.22	7.86	

Table 6-2 Blast data summary (2/2)

Beams	Shockwave properties				Displacement		$\theta_{\max}$	Failure Mode
ID	Shot	I_r	P_r	t_p	$\delta_{\max}$	δ_{res}		
	(psi)	(kPa-ms)	(kPa)	(ms)	(mm)	(mm)	degree	
HSC-F0.5 (ZP)-20M-S	17	248.0	25.8	19.26	8.54	2.88	0.44	Concrete Crushing
	30	370.0	44.5	16.62	13.96	1.68	0.72	
	50	564.0	59.2	19.05	24.09	7.13	1.24	
	70	750.0	78.2	19.18	48.55	29.59	2.49	
	80	792.3	90.8	17.45	119.52	77.69	6.14	
HSC-F0.5 (5D)-20M-S	17	278.0	25.7	21.65	8.45	1.59	0.43	Concrete Crushing
	30	416.4	44.2	18.84	14.57	1.47	0.75	
	50	565.0	63.5	17.79	25.37	4.74	1.30	
	70	740.9	74.5	19.90	62.66	35.90	3.22	
	80	792.3	90.8	17.45	148.58	84.55	7.63	

6.3 Description of Experimental Results – Series #4

This section details the experimental results for the #4 series beams. As discussed in Chapter 3, the specimens in this series were tested at driver pressures of 17 psi (Blast 1), 30 psi (Blast 2) and then increased in increments of 10 psi until failure (Blasts 2A, 3, 3A). Blast 1 (17 psi) was intended to keep the beams within the elastic range; blast 2 (30 psi) and all other blasts to the end of the repeated blast loading testing were calibrated to bring the reinforcement within the plastic range.

6.3.1 Specimen NSC – F0-#4-S

The NSC-F0-#4-S beam was constructed using self-consolidated concrete (SCC) as a normal-strength concrete to investigate the effects of concrete strength. The specimen was reinforced longitudinally with #4 bars and transversely with 6.3 mm stirrups spaced at 100 mm. The experimental results for the shockwaves and displacements for each blast are shown in Table 6-1. Graphs showing reflected pressure, impulse and mid-span displacement histories are illustrated in Figure 6-1. Photographs showing the damage on the beam after each blast are included in Figure 6-2.

Blast 1 (17 psi) resulted in the development of hairline cracks over the beam specimen. The maximum and residual displacements after this blast were measured to be 12.65 mm and 1.54 mm respectively. The tensile strain gauge recorded a maximum strain of 0.00256 mm/mm ($\epsilon_{\text{res}} = 0.00019$ mm/mm), resulting in the yield of reinforcement. Hairline cracks were distributed in the mid-span region, and a few hairline cracks were observed in the shear span.

Blast 2 (30 psi) resulted in a major crack in the moment region. The beam reached a maximum displacement of 29.53 mm and settled at a residual displacement of 13.04 mm. A major crack appeared in the mid-span which was measured to be 1.70 mm wide at 1130 mm from the bottom support.

Blast 2A (40 psi) resulted in the failure of the specimen at mid-span. The LVDT measured the maximum and residual displacements of the beam to be 47.96 mm and 25.11 mm respectively. Concrete spalling occurred at the mid-span of the specimen on the tension zone exposing the steel bars, as seen in Figure 6-2.

6.3.2 Specimen HSC-F0-#4-S

The HSC-F0-#4-S beam was composed of high-strength concrete, #4 longitudinal steel bars, and 6.3 mm stirrups spaced at 100 mm. The experimental results in terms of shockwaves and displacements properties for each blast are shown in Table 6-1. Graphs showing reflected pressure, impulse and mid-span displacement histories are illustrated in Figure 6-3. Photographs showing the damage on the beam after each blast are included in Figure 6-4.

Blast 1 (17 psi) resulted in the appearance of hairline cracks in the beam specimen. The maximum and residual displacements were measured to be 13.15 mm and 3.62 mm respectively. The tensile strain gauge recorded a maximum tensile strain of 0.0029 mm/mm ($\epsilon_{res} = 0.0005$ mm/mm) resulting in the yield of the steel bars. The hairline cracks appeared in the mid-span region of the beam.

Blast 2 (30 psi) resulted in a major crack in the moment region. The HSC-F0-#4-S beam reached a maximum displacement of 30.44 mm and settled at a residual displacement of 14.16 mm. The hairline crack widened and became a major crack with a width of 1.65 mm.

Blast 2A (40 psi) caused the failure of the specimen at mid-span. The LVDT measured the maximum and residual displacement of the beam to be 44.53 mm and 6.59 mm, respectively. Concrete in tension zone in the mid-span region spalled, exposing the steel bars as seen in Figure 6-4(e). This created two large (fistful size) concrete fragments flew which landed on the floor at a distance of 2000 mm and 2500 mm from the beam.

6.3.3 Specimen HSC-F1(ZP)-#4-0

The HSC-F1(ZP)-#4-0 beam was constructed with the same components as the HSC-F0-#4-S specimen, with the addition of 1.0% steel fibres by volume of concrete. This addition resulted in an enhancement in the beam behaviour in terms of maximum and residual displacements when compared to the other two companion specimens. The experimental results for the shockwaves and displacements for each blast are shown in

Table 6-1. Graphs showing reflected pressure, impulse and mid-span displacement histories are illustrated in Figure 6-5 and Figure 6-6. Photographs showing the damage in the beam after each blast are included in Figure 6-7.

Blast 1 (17 psi) resulted in the development of hairline cracks in the mid-span of the beam specimen. The maximum and residual displacements were measured at 10.63 mm and 3.71 mm respectively. The tensile strain gauge recorded an average maximum strain of 0.00526 mm/mm ($\epsilon_{res} = 0.00038$ mm/mm) resulting in the yielding of the steel bars. Hairline cracks were distributed over the mid-span region.

Blast 2 (30 psi) resulted in the first major crack at the mid-span of the beam. The beam reached a maximum displacement of 18.47 mm and settled at a residual displacement of 4.10 mm. Three major cracks emerged at mid-span with widths of 0.41–1.14 mm located at 780 mm, 975 mm, and 1350 mm from the bottom support.

Blast 2A (40 psi) caused an increase in the width of the major cracks but did not result in failure of the specimen. The LVDT measured the maximum and residual displacement of the beam at 35.18 mm and 15.61 mm respectively. The three major cracks expanded to crack widths which ranged from 2.21 mm–5.10 mm.

Blast 3 (50 psi) resulted in further development as widening of the major cracks. The maximum and residual displacements were measured at 55.10 mm and 32.26 mm, respectively. The three major cracks widened to widths in the range of 4.18 mm–12.0 mm, with the initiation of the pullout of the steel fibres.

Blast 3A (60 psi) resulted in the failure of the specimen at mid-span. The maximum and residual displacements were measured at 61.0 mm and 38.89 mm, respectively. Concrete splitting occurred at the mid-span of the specimen and the steel fibres pulled out at the major crack location which was 975 mm away from the bottom support.

6.4 Description of Experimental Results – Series 15M

This section details the experimental results for series 15M beams. As with the previous series the specimens were initially tested at driver pressures of 17 psi (Blast 1) and 30 psi (Blast 2) which were intended to bring the specimens to yield and into the plastic range respectively. The remaining two blasts were applied by increasing the driver pressure in 20 psi increments (Blast 3 – 50 psi; Blast 4 - 70 psi) and were intended to increase damage and cause failure of the specimens. .

6.4.1 Specimen HSC-F0-15M-S

The HSC-F0-15M-S beam was composed of high-strength concrete, normal-strength 15M longitudinal steel bars, and shear reinforcement consisting of 6.3 mm stirrups spaced at 100 mm. The experimental results for the shockwaves and displacements for each blast are shown in Table 6-1. Graphs showing reflected pressure, impulse and mid-span displacement histories are illustrated in Figure 6-8. Photographs showing the damage in the beam after each blast are included in Figure 6-9.

Blast 1 (17 psi) resulted in the formation of hairline cracks in the moment region. The maximum and residual displacements were measured at 11.50 mm and 2.66 mm respectively. Hairline cracks were distributed over the mid-span with a spacing varied between 150 mm and 200 mm in the constant moment region. Unfortunately, the strain gauge did not work in this specimen due to a technical issue.

Blast 2 (30 psi) resulted in the formation of a major crack in the flexural region. The beam reached a maximum displacement of 21.40 mm and settled at a residual displacement of 4.71 mm. The major crack which appeared in the mid-span region had a 1.10 mm crack width and appeared at 1140 mm from the bottom support.

Blast 3 (50 psi) resulted in the failure of the specimen at mid-span. The LVDT measured the maximum and residual displacement of the beam at 124.95 mm and 22.04 mm respectively. Concrete crushing occurred in the compression zone in the moment region, with complete disintegration of the ejection of debris and the formation of secondary blast fragments. In addition, damage was observed in the tension zone with a small chunk of concrete held in the tension zone with multiple cracks around it, as seen in Figure 6-9.

6.4.2 Specimen HSC-F1(ZP)-15M-0

The composition of the HSC-F1(ZP)-15M-0 beam was the same as that of the HSC-F0-15M-S, except for the addition of a 1.0% steel fibres and the omission of stirrups. This addition of steel fibres resulted in an improvement in the beam behaviour compared to that of the previous specimen. The experimental results for the shockwaves and displacements for each blast are shown in Table 6-1. Graphs showing reflected pressure history with either impulse or mid-span displacement are illustrated in Figure 6-10 and Figure 6-11. Photographs showing the progression of the damage for the beam after each blast are included in Figure 6-12.

Blast 1 (17 psi) resulted in the development of hairline cracks in the mid-span of the beam. The maximum and residual displacements were measured at 10.05 mm and 3.91 mm respectively. It was observed that a significant number of hairline cracks appeared at the

mid-span. Unfortunately, the strain gauge did not work in this specimen due to a technical issue.

Blast 2 (30 psi) resulted in the development of further hairline cracks which spread into the shear spans. After this test, the beam reached a maximum displacement of 15.02 mm and settled at a residual displacement of 2.66 mm. Hairline cracks appeared in different parts of the beam span as shown in Figure 6-12.

Blast 3 (50 psi) resulted in the development of a major crack in the mid-span but did not cause failure of the specimen. The LVDT measured the maximum and residual displacement of the beam at 33.31 mm and 10.08 mm respectively. A major crack appeared with a crack width of 3.90 mm at 1135 mm from the bottom support.

Blast 4 (70 psi) resulted in the failure of the specimen at mid-span. The maximum and residual displacements were measured at 78.91 mm and 54.68 mm, respectively. The major crack expanded to a very large width of 19.80 mm, resulting in a concrete splitting failure at mid-span and the pullout of the steel fibres at this location, as seen in Figure 6-12.

6.5 Description of Experimental Results — Series 20M

This section details the experiment results for the series 20M beams. The 6 specimens in this series investigated the effect of transverse reinforcement, steel fibres, steel fibre content, steel fibre properties and combined use of fibres and stirrups. The specimens in this series were tested under gradually increasing pressures with Blasts 1-4 matching the intensities used in the 15M series (Blast 1 - 17 psi; Blast 2 - 30 psi; Blast 3 - 50 psi; Blast 4 - 70 psi). For some beams a 5th blast at a driver pressure of 80 psi was required to cause failure in the 20M specimens.

6.5.1 Specimen HSC-F0-20M-0

The HSC-F0-20M-0 beam was composed of high-strength concrete and 20 M longitudinal reinforcement. This beam was expected to fail in shear due to the lack of transverse steel reinforcement. The experimental results for the shockwaves and displacements for each blast are shown in Table 6-1. Graphs showing reflected pressure history with either impulse or mid-span displacement are illustrated in Figure 6-13. Photographs showing the damage and failure of the beam are included in Figure 6-14.

Blast 1 (17 psi) resulted in the appearance of hairline cracks over the beam span. The maximum and residual displacements after this blast were measured to be 12.55 mm and 3.37 mm respectively. The appearance of the first hairline cracks were observed in the shear spans and longer cracks over the mid-span.

Blast 2 (30 psi) resulted in the failure of the specimen. The specimen suffered a brittle shear failure mode with the diagonal crack forming in the top shear span. The beam reached a maximum displacement of 19.31 mm and settled at a residual displacement of 9.21 mm. The diagonal shear crack had a width of 11.25 mm, as seen in Figure 6-14.

6.5.2 Specimen HSC-F0-20M-S

The HSC-F0-20M-S beam had the same composition as the previous specimen, except for the addition of transverse reinforcement spaced at 100 mm in the shear spans. The experimental results for the shockwaves and displacements for each blast are shown in Table 6-1. Graphs showing reflected pressure history with either impulse or mid-span displacement are illustrated in Figure 6-15 and Figure 6-16. Photographs showing the damage progression in the beam are included in Figure 6-17.

Blast 1 (17 psi) resulted in the emergence of hairline cracks in the moment region. The maximum and residual displacements were measured at 10.44 mm and 1.98 mm respectively. With this level of blast load, hairline cracks were seen over the mid-span. Unfortunately, the strain gauge did not work in this specimen due a technical issue.

Blast 2 (30 psi) resulted in the formation of further hairline cracks distributed over the beam span but did not result in failure of the specimen. The beam reached a maximum displacement of 15.12 mm and then settled at a residual displacement of 0.15 mm. It was observed that more hairline cracks appeared throughout the beam span.

Blast 3 (50 psi) resulted in the development of a major crack at mid-span. The LVDT measured the maximum and residual displacement of the beam at 32.91 mm and 12.43 mm respectively. The major crack had a width of 2.0 mm at 1020 mm from the bottom support.

Blast 4 (70 psi) resulted in the failure of the specimen at mid-span. The maximum and residual displacements were measured at 118.06 mm and 71.69 mm, respectively. The failure occurred in the compression zone due to concrete crushing at mid-span, with disintegration of the concrete and the ejection of small concrete fragments which were spread over the floor within a distance of 2000 mm from the beam, as seen in Figure 6-17.

6.5.3 Specimen HSC-F1(ZP)-20M-0

The composition of the HSC-F1(ZP)-20M-0 beam matched that of Beam HSC-F0-20M-0, except for the addition of steel fibres at a ratio 1.0% by volume of concrete. It is noted that this HSFRC beam was constructed without shear reinforcement. The experimental results for the shockwaves and displacements for this beam after each blast are shown in Table 6-1. Graphs showing reflected pressure, impulse and mid-span

displacement histories are illustrated in Figure 6-18 and Figure 6-19. Photographs showing the damage progression in the beam after each blast are included in Figure 6-20.

Unlike previous specimens, Blast 1 (17 psi) did not result in the formation of cracks over the beam specimen span. The maximum and residual displacements were measured at 8.44 mm and 1.40 mm respectively. Unfortunately, the strain gauge did not work in this specimen due to a technical issue.

Blast 2 (30 psi) resulted in the appearance of hairline cracks in the flexural span but did not result in shear failure of the specimen. The beam reached a maximum displacement of 14.69 mm, and settled at a residual displacement of 3.43 mm. This shot resulted in the first hairline cracks that were distributed over the mid-span.

Blast 3 (50 psi) resulted in the formation of a major crack at mid-span. The LVDT measured the maximum and residual displacement of the beam to be 20.82 mm and 4.07 mm, respectively. The major crack had a width of 0.95 mm, approximately 890 mm away from the bottom support.

Blast 4 (70 psi) resulted in further damages to the specimen, and more cracks at mid-span but did not cause failure of the specimen. The maximum and residual displacements were measured at 42.06 mm and 19.27 mm respectively. The major crack widened to 9.25 mm, and more cracks appeared over the beam span. Since damage was relatively minor it was decided to test the beam under one more time.

Blast 5 (80 psi) resulted in the failure of the specimen at mid-span. The maximum and residual displacements were measured at 62.4 mm and 37.36 mm respectively. The concrete in the tension region at mid-span showed splitting, and the steel fibres were observed to have pulled out at the major crack, causing a crack width of 20.90 mm.

6.5.4 Specimen HSC-F1(ZP)-20M-S

The HSC-F1(ZP)-20M-S beam had the same material composition as the previous HSC-F1-20M-0 specimen, except for the addition of shear reinforcement (this test therefore allowed for an investigation of combined use of stirrups and steel fibres). The experimental results for the shockwaves and displacements after each blast are shown in Table 6-1. Graphs showing reflected pressure, impulse and mid-span displacement histories are illustrated in Figure 6-21 and Figure 6-22. Photographs showing the damage progression in the beam are included in Figure 6-23.

Blast 1 (17 psi) resulted in the formation of hairline cracks in the moment region of the beam. The maximum and residual displacements were measured at 9.21 mm and 2.29 mm

respectively. This specimen showed hairline cracks over the mid-span at this level of blast loading, while the previous specimen remained without cracks. Unfortunately, the strain gauge did not work in this specimen due to a technical issue.

Blast 2 (30 psi) exhibited further hairline cracks in the beam span. The beam reached a maximum displacement of 14.91 mm and settled at a residual displacement of 2.20 mm. This specimen showed no major cracks at blast 2, but had more hairline cracks which appeared in the mid-span region.

Blast 3 (50 psi) resulted in the formation of two major cracks at mid-span. The LVDT measured the maximum and residual displacement of the beam at 27.64 mm and 4.04 mm respectively. The two major cracks were 1.70 mm and 1.90 mm wide, occurring 1200 mm and 1095 mm away from the bottom support.

Blast 4 (70 psi) resulted in more damage and more cracks at mid-span. The maximum and residual displacements were measured at 67.03 mm and 41.84 mm respectively. The two major cracks expanded to widths of 3.90 mm and 9.85 mm. Also, more hairline cracks developed in other regions in the beam span.

Blast 5 (80 psi) resulted in the failure of the specimen at mid-span. The maximum and residual displacements were measured at 153.01 mm and 79.22 mm respectively. Concrete crushing occurred at the mid-span of the specimen. The steel fibre were also observed to have pulled out at the major crack that was 1095 mm away from the bottom support, as shown in Figure 6-23.

6.5.5 Specimen HSC-F0.5(ZP)-20M-S

The HSC-F0.5(ZP)-20M-S beam had the same material composition as that of the HSC-F1(ZP)-20M-S specimen, except for a reduced content of steel fibres to 0.5%. The experimental results in terms of shockwaves and displacements for each blast are shown in Table 6-2. Graphs showing reflected pressure, impulse and mid-span displacement histories are illustrated in Figure 6-24 and Figure 6-25. Photographs showing the damage in the beam after each blast are included in Figure 6-26.

Blast 1 (17 psi) resulted in the appearance of hairline cracks distributed in the flexural span. The maximum and residual displacements were measured at 8.54 mm and 2.88 mm respectively. The tensile strain gauge recorded a maximum tensile strain of 0.00131 mm/mm ($\epsilon_{res} = 0.00026\text{mm/mm}$), leading to the conclusion that the longitudinal reinforcement was still in the elastic region. Hairline cracks were distributed over the mid-span.

Blast 2 (30 psi) exhibited more hairline cracks which developed in the beam span. The beam reached a maximum displacement of 13.96 mm and settled at a residual displacement of 1.68 mm. The tensile strain gauge recorded a maximum tensile strain of 0.0046 mm/mm ($\epsilon_{res} = 0.00076$ mm/mm), resulting in the yielding of longitudinal reinforcement. Moreover, further hairline cracks appeared in the mid-span.

Blast 3 (50 psi) resulted in the formation of one major crack at mid-span. The LVDT measured the maximum and residual displacements of the beam to be 24.09 mm and 7.13 mm respectively. The major crack had a width of 1.80 mm and was located at 1230 mm from the bottom support.

Blast 4 (70 psi) resulted in more damage and more cracks at mid-span. The maximum and residual displacements were measured to be 48.55 mm and 29.59 mm respectively. The major crack widened to 9.90 mm with partial pullout of the steel fibres at this location. Additional hairline cracks developed throughout the beam as a result of this blast.

Blast 5 (80 psi) resulted in the failure of the specimen at mid-span. The maximum and residual displacements were measured to be 119.52 mm and 77.69 mm respectively. The failure was almost the same as for the previous specimen with concrete crushing in the compression zone and steel fibre pullout at the major crack location, except that this specimen showed a lower maximum displacement.

6.5.6 Specimen HSC-F0.5 (5D)-20M-S

The HSC-F0.5 (5D)-20M-S beam had the same material composition as the previous specimen except for the type of fibre which was used. For this beam, the longer 5D steel fibres were used. These fibres also had a greater aspect-ratio, tensile strength and improved anchorage properties (double-hooks). The experimental results for the beam in the form of shockwaves and displacements properties after each blast are shown in Table 6-2. Graphs showing reflected pressure, impulse and mid-span displacement histories are illustrated in Figure 6-27 and Figure 6-28. Photographs showing the damage in the beam after each blast are included in Figure 6-29.

Blast 1 (17 psi) resulted the appearance of hairline cracks in the beam. The maximum and residual displacements were measured at 8.45 mm and 1.59 mm respectively. The tensile strain gauge recorded a maximum tensile strain of 0.015 mm/mm ($\epsilon_{res} = 1.3 \times 10^{-5}$ mm/mm), which would seem to indicate the yielding of the steel bars. Hairline cracks were distributed throughout the beam span.

Blast 2 (30 psi) exhibited further hairline cracks which developed in the beam span. The beam reached a maximum displacement of 14.57 mm and settled at a residual displacement of 1.47 mm. Additional hairline cracks appeared in the mid-span region.

Blast 3 (50 psi) resulted in the formation of two major cracks at mid-span. The LVDT measured the maximum and residual displacement of the beam at 25.37 mm and 4.74 mm respectively. The two major cracks had widths of 0.40 mm and 0.50 mm and appeared at a distances of 1340 mm and 1210 mm from the bottom support.

Blast 4 (70 psi) resulted in the formation of cracking which extended all the way into the compression zone at mid-span. The maximum and residual displacements were measured at 62.66 mm and 35.90 mm respectively. The two major cracks in tension zone widened to 4.50 mm and 5.20 mm and small fragments of concrete fell from the compression zone of the beam.

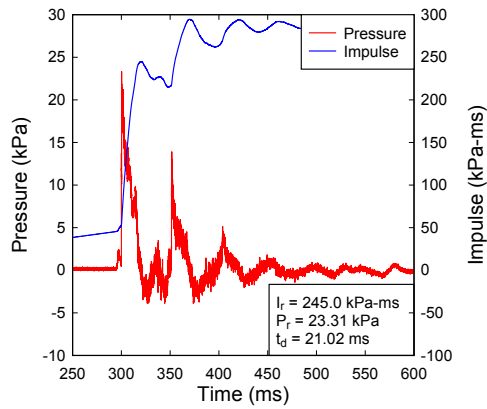
Blast 5 (80 psi) resulted in the failure of the specimen at mid-span. The maximum and residual displacements were measured to be 148.58 mm and 84.55 mm respectively. Concrete crushing at the mid-span of the specimen occurred and the steel fibres were observed to have pulled out at the major crack location which was 1210 mm away from the bottom support. It is observed that the failure of this beam was similar to that of the previous specimen, except that this one exhibited a higher maximum displacement.

6.6 Pressure, Impulse, and Displacement Time Histories & Selected Photographs

6.6.1 Specimen NSC-F0-#4-S

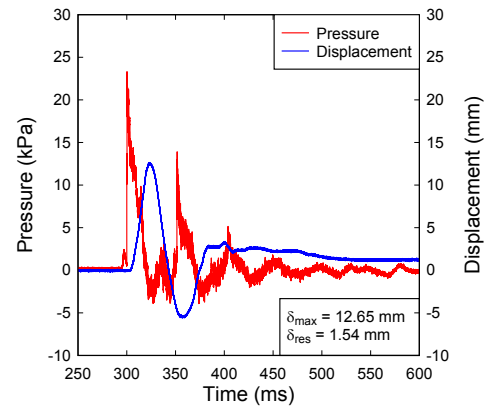
Pressure and Impulse Time History

SCC-F0-#4-S - Blast 1: 9ft Driver, $P_d = 17$ psi



Pressure and Displacement Time History

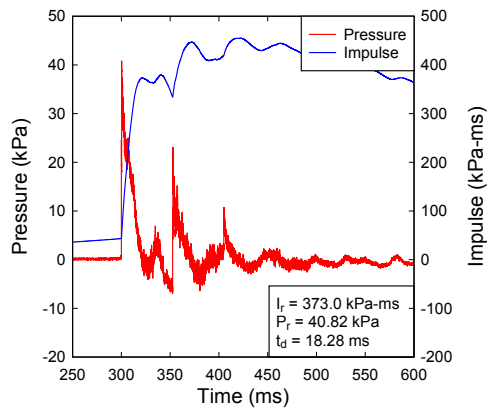
SCC-F0-#4-S - Blast 1: 9ft Driver, $P_d = 17$ psi



a) Blast 1: Reflected pressure, impulse, and displacement time histories.

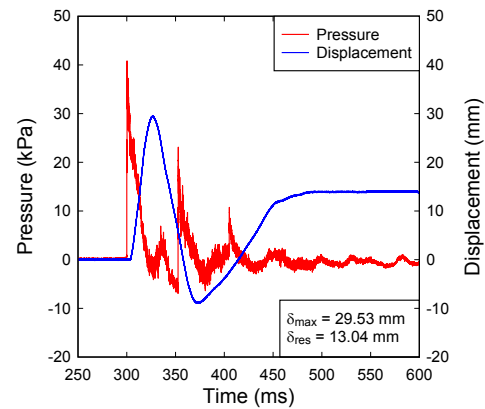
Pressure and Impulse Time History

SCC-F0-#4-S - Blast 2: 9ft Driver, $P_d = 30$ psi



Pressure and Displacement Time History

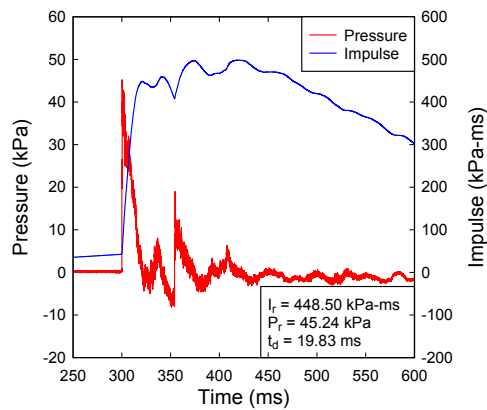
SCC-F0-#4-S - Blast 2: 9ft Driver, $P_d = 30$ psi



b) Blast 2: Reflected pressure, impulse, and displacement time histories.

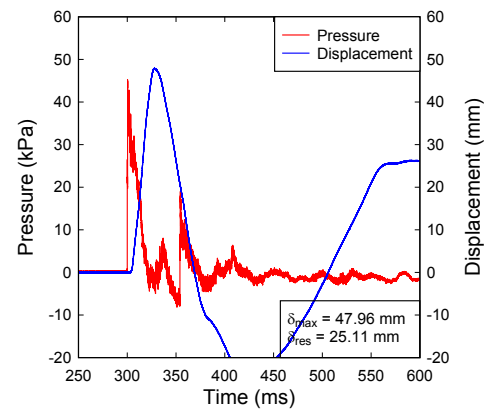
Pressure and Impulse Time History

SCC-F0-#4-S - Blast 2(A): 9ft Driver, $P_d = 40$ psi



Pressure and Displacement Time History

SCC-F0-#4-S - Blast 2(A): 9ft Driver, $P_d = 40$ psi



a) Blast 2A: Reflected pressure, impulse, and displacement time histories.

Figure 6-1 NSC-F0-#4-S, recorded reflected pressure, impulse and displacement for Blast 1-2A



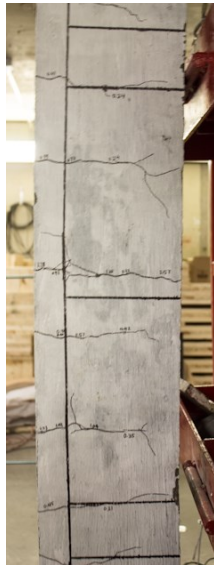
a) Blast 1



b) Blast 2



c) Blast 2A



d) Mid-span cracking
at Blast 2



e) Mid-span spalling
at Blast 2A



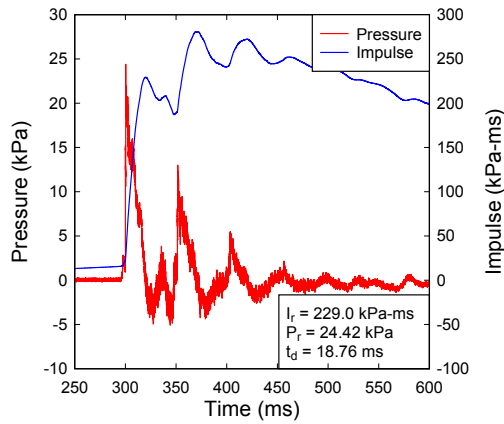
f) Mid-span spalling
at Blast 2A

Figure 6-2 NSC-F0-#4-S, photographs at the end of Blast 1-2A

6.6.2 Specimen HSC-F0-#4-S

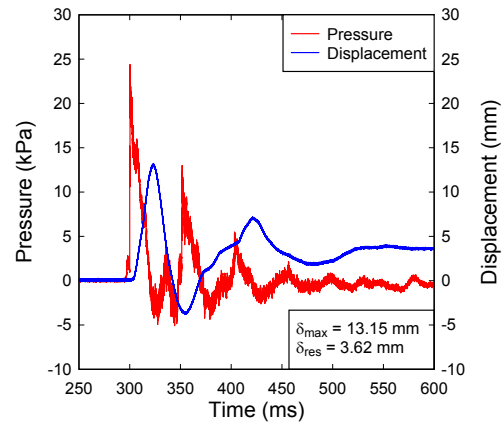
Pressure and Impulse Time History

HSC-F0-#4-S - Blast 1: 9ft Driver, $P_d = 17$ psi



Pressure and Displacement Time History

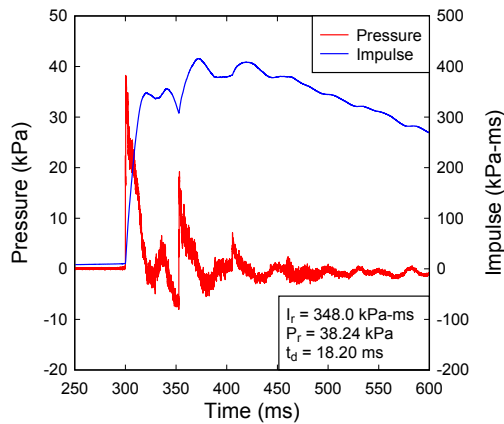
HSC-F0-#4-S - Blast 1: 9ft Driver, $P_d = 17$ psi



a) Blast 1: Reflected pressure, impulse, and displacement time histories.

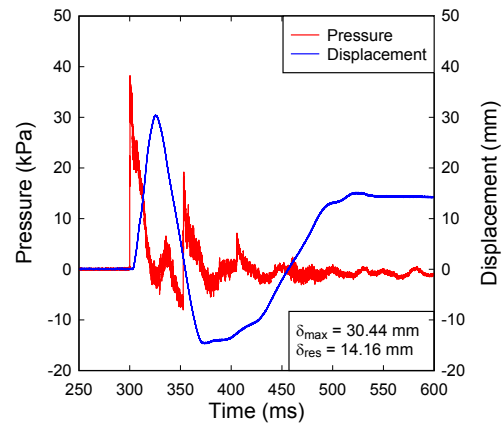
Pressure and Impulse Time History

HSC-F0-#4-S - Blast 2: 9ft Driver, $P_d = 30$ psi



Pressure and Displacement Time History

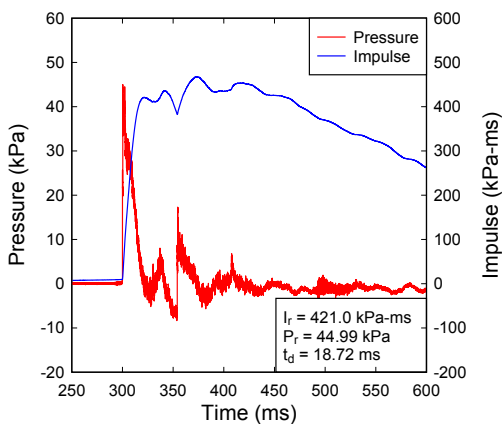
HSC-F0-#4-S - Blast 2: 9ft Driver, $P_d = 30$ psi



b) Blast 2: Reflected pressure, impulse, and displacement time histories.

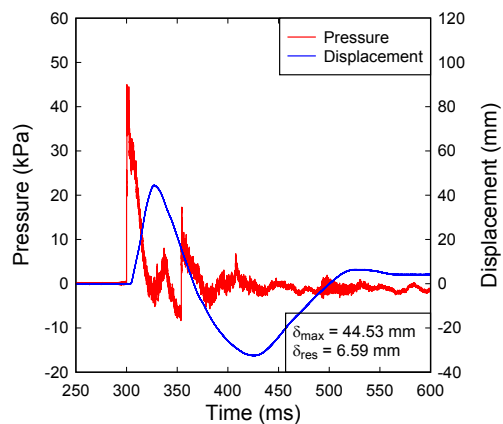
Pressure and Impulse Time History

HSC-F0-#4-S - Blast 2(A): 9ft Driver, $P_d = 40$ psi



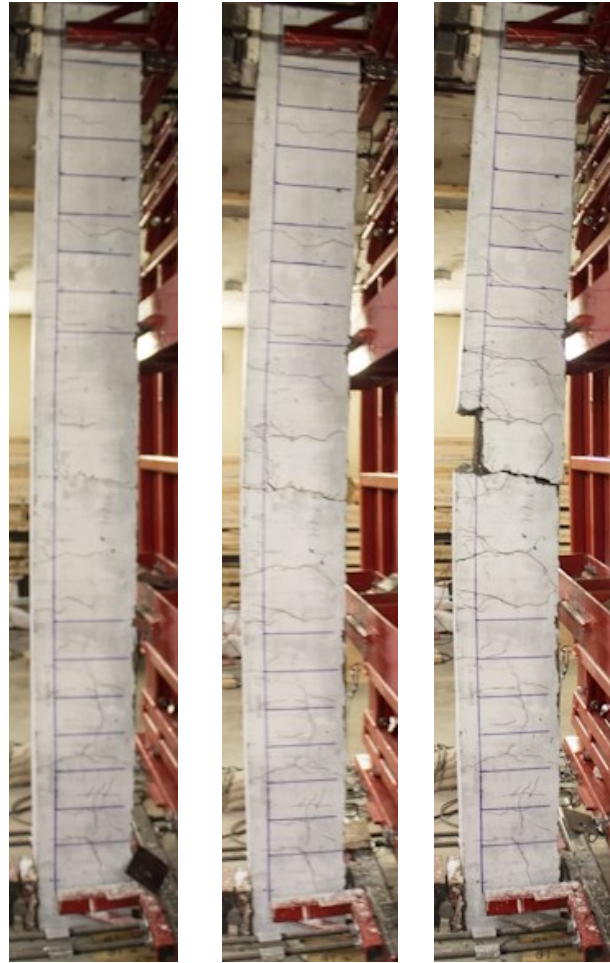
Pressure and Displacement Time History

HSC-F0-#4-S - Blast 2(A): 9ft Driver, $P_d = 40$ psi



c) Blast 2A: Reflected pressure, impulse, and displacement time histories.

Figure 6-3 HSC-F0-#4-S, recorded reflected pressure, impulse and displacement for Blast 1-2A



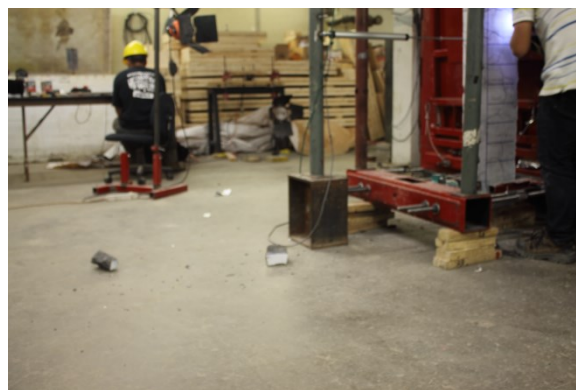
a) Blast 1

b) Blast 2

c) Blast 2A



d) Mid-span cracking at Blast 2



e) Mid-span concrete chunks at Blast 2A



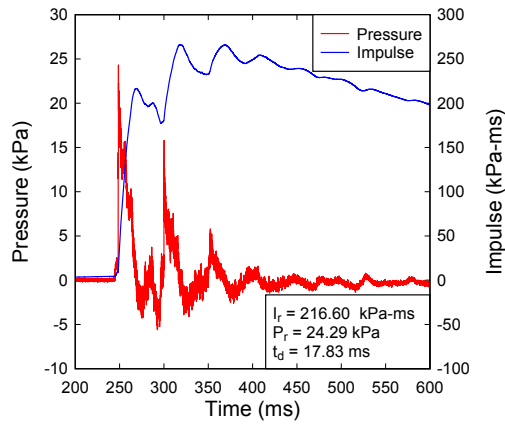
f) Mid-span spalling at Blast 2A

Figure 6-4 HSC-F0-#4-S, photographs at the end of Blast 1-2A

6.6.3 Specimen HSC-F1(ZP)-#4-0

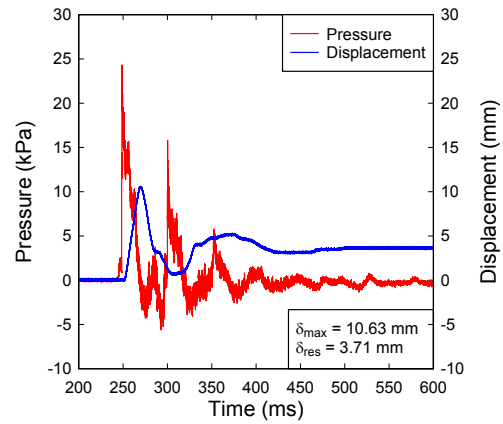
Pressure and Impulse Time History

HSC-F1-#4-0 - Blast 1: 9ft Driver, $P_d = 17$ psi



Pressure and Displacement Time History

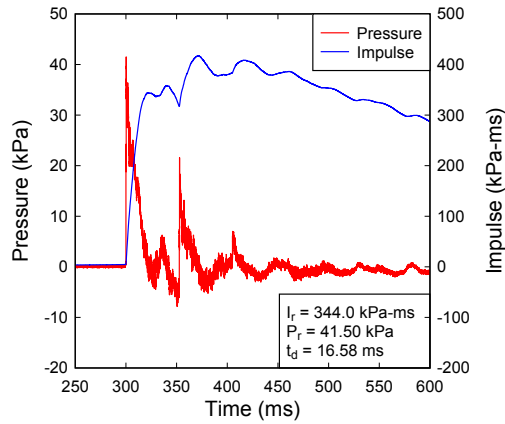
HSC-F1-#4-0 - Blast 1: 9ft Driver, $P_d = 17$ psi



a) Blast 1: Reflected pressure, impulse, and displacement time histories.

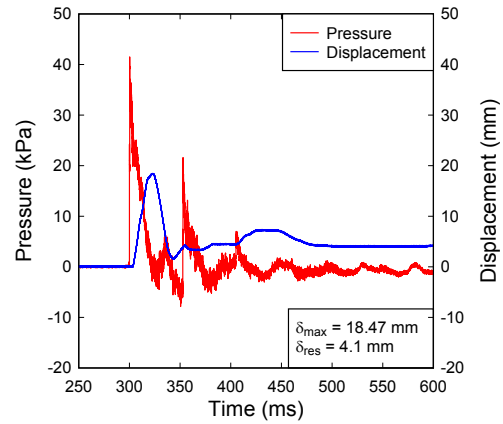
Pressure and Impulse Time History

HSC-F1-#4-0 - Blast 2: 9ft Driver, $P_d = 30$ psi



Pressure and Displacement Time History

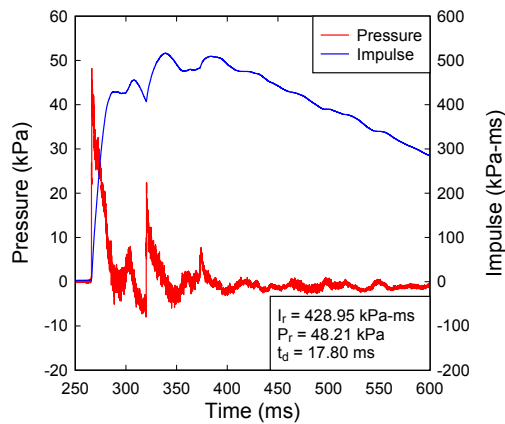
HSC-F1-#4-0 - Blast 2: 9ft Driver, $P_d = 30$ psi



b) Blast 2: Reflected pressure, impulse, and displacement time histories.

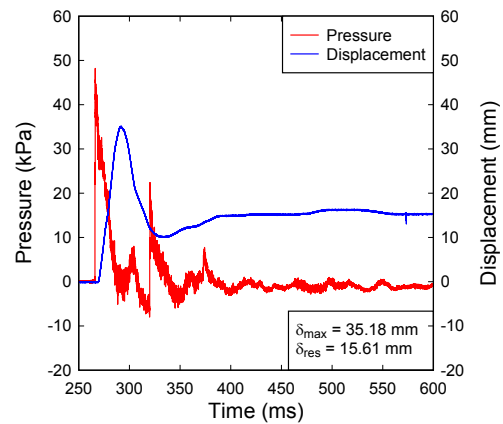
Pressure and Impulse Time History

HSC-F1-#4-0 - Blast 2(A): 9ft Driver, $P_d = 40$ psi



Pressure and Displacement Time History

HSC-F1-#4-0 - Blast 2(A): 9ft Driver, $P_d = 40$ psi

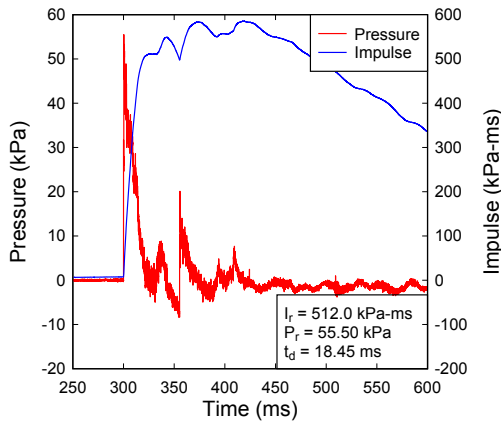


c) Blast 2A: Reflected pressure, impulse, and displacement time histories.

Figure 6-5 HSC-F1(ZP)-#4-0, recorded reflected pressure, impulse and displacement for Blast 1-2A

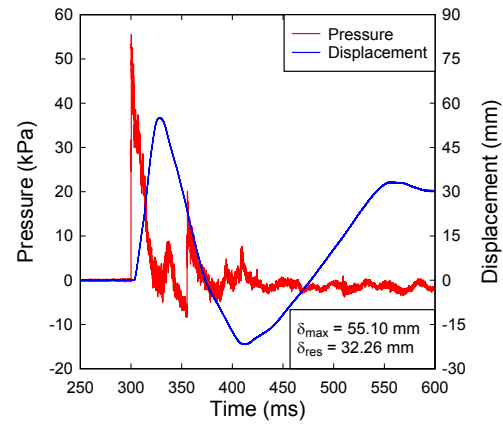
Pressure and Impulse Time History

HSC-F1-#4-0 - Blast 3: 9ft Driver, $P_d = 50$ psi



Pressure and Displacement Time History

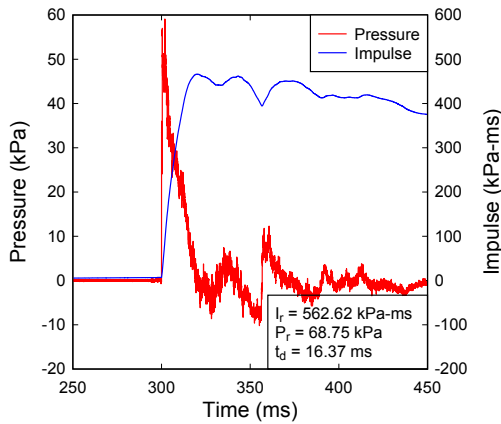
HSC-F1-#4-0 - Blast 3: 9ft Driver, $P_d = 50$ psi



a) Blast 3 : Reflected pressure, impulse, and displacement time histories.

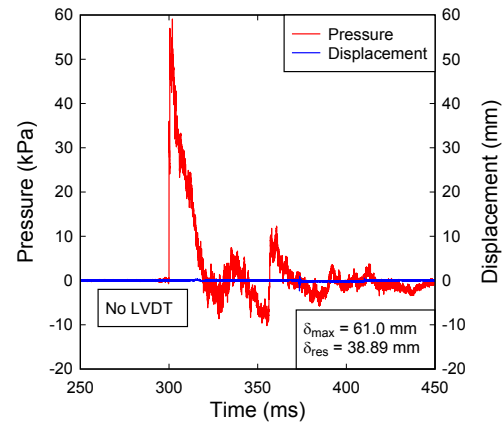
Pressure and Impulse Time History

HSC-F1-#4-0 - Blast 3(A): 9ft Driver, $P_d = 60$ psi



Pressure and Displacement Time History

HSC-F1-#4-0 - Blast 3(A): 9ft Driver, $P_d = 60$ psi



b) Blast 3A: Reflected pressure, impulse, and displacement time histories.

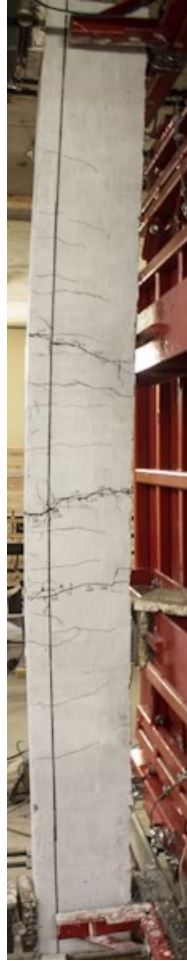
Figure 6-6 HSC-F1(ZP)-#4-0, recorded reflected pressure, impulse and displacement for Blast 3-3A



a) Blast 1



b) Blast 2



c) Blast 2A



d) Blast 3



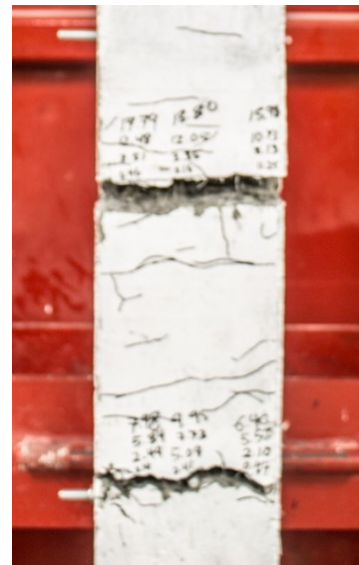
e) Blast 3A



f) Mid-span cracking
at Blast 3



g) Mid-span splitting
at Blast 3A

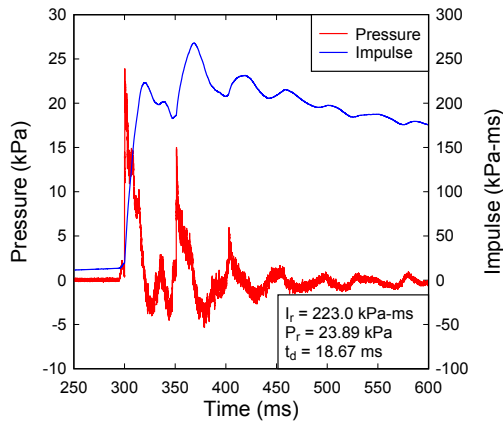


h) Mid-span fibre pulled out
at Blast 3A

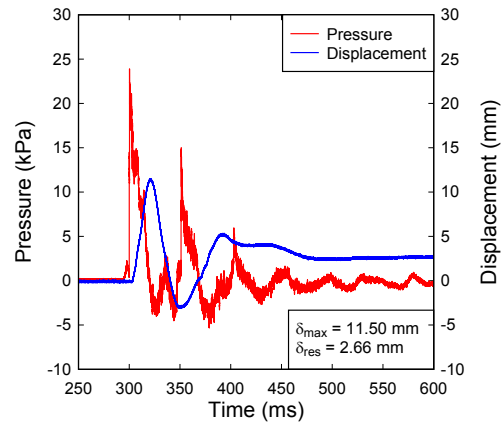
Figure 6-7 HSC-F1(ZP)-#4-0, photographs at the end of Blast 1-4

6.6.4 Specimen HSC-F0-15M-S

Pressure and Impulse Time History
HSC-F0-15M-S - Blast 1: 9ft Driver, $P_d = 17$ psi

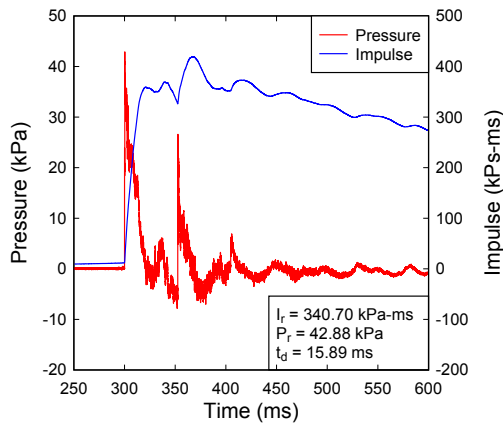


Pressure and Displacement Time History
HSC-F0-15M-S - Blast 1: 9ft Driver, $P_d = 17$ psi

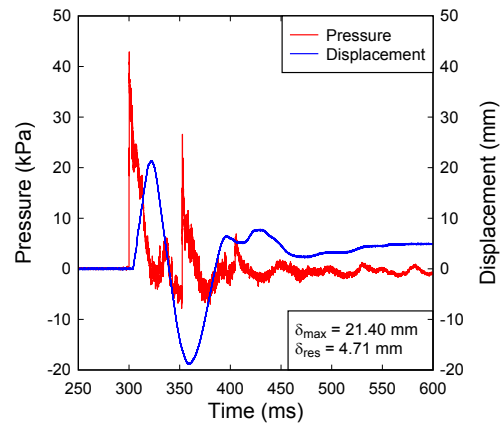


a) Blast 1: Reflected pressure, impulse, and displacement time histories.

Pressure and Impulse Time History
HSC-F0-15M-S - Blast 2: 9ft Driver, $P_d = 30$ psi

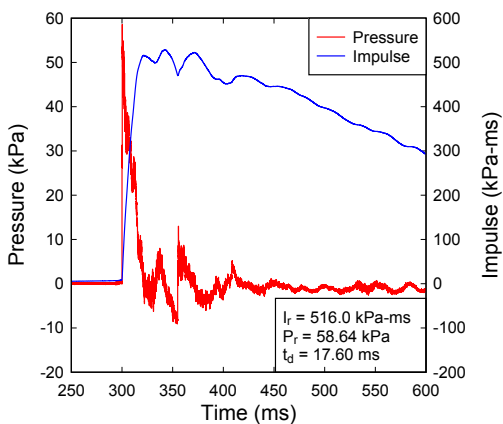


Pressure and Displacement Time History
HSC-F0-15M-S - Blast 2: 9ft Driver, $P_d = 30$ psi

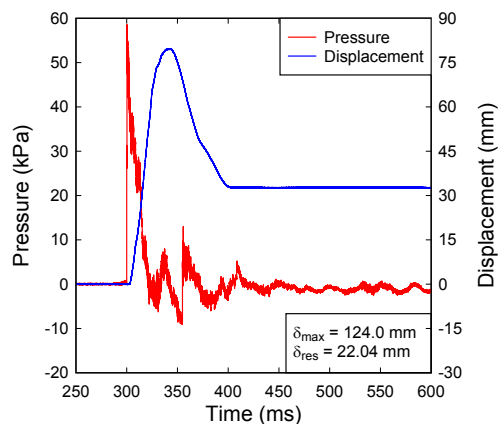


b) Blast 2: Reflected pressure, impulse, and displacement time histories.

Pressure and Impulse Time History
HSC-F0-15M-S - Blast 3: 9ft Driver, $P_d = 50$ psi

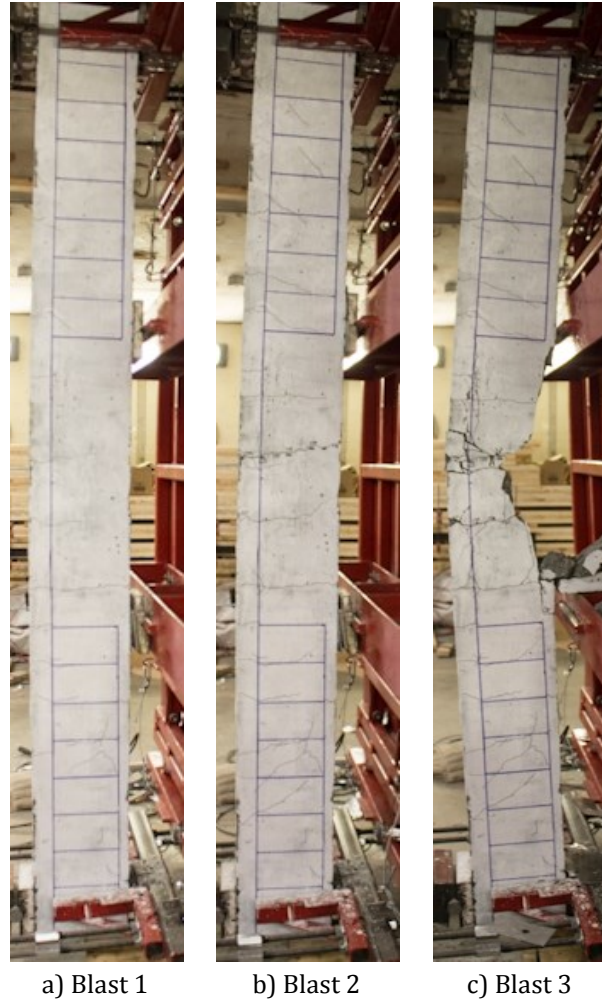


Pressure and Displacement Time History
HSC-F0-15M-S - Blast 3: 9ft Driver, $P_d = 50$ psi



c) Blast 3: Reflected pressure, impulse, and displacement time histories.

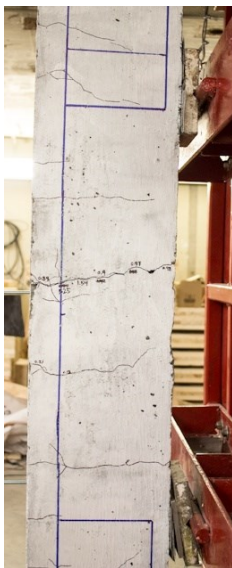
Figure 6-8 HSC-F0-15M-S, recorded reflected pressure, impulse and displacement for Blast 1-3



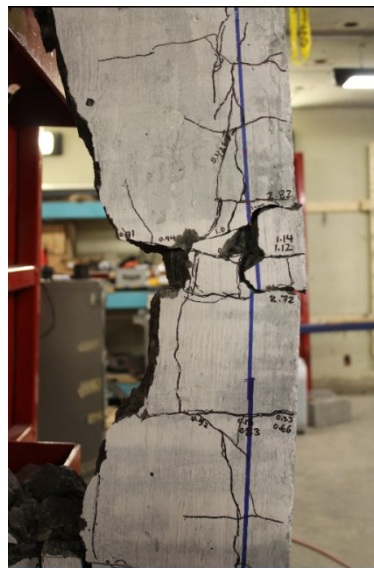
a) Blast 1

b) Blast 2

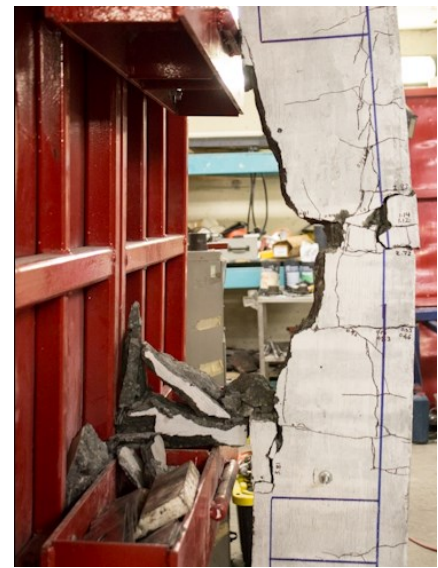
c) Blast 3



d) Mid-span cracking
at Blast 2



e) Mid-span small chunk hanging
at Blast 3



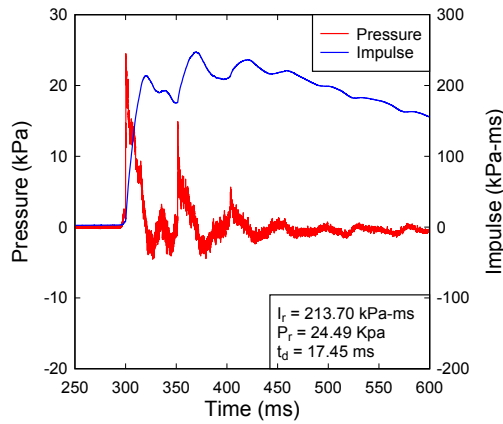
f) Mid-span crushing
at Blast 3

Figure 6-9 HSC-F0-15M-S, photographs at the end of Blast 1-3

6.6.5 Specimen HSC-F1(ZP)-15M-0

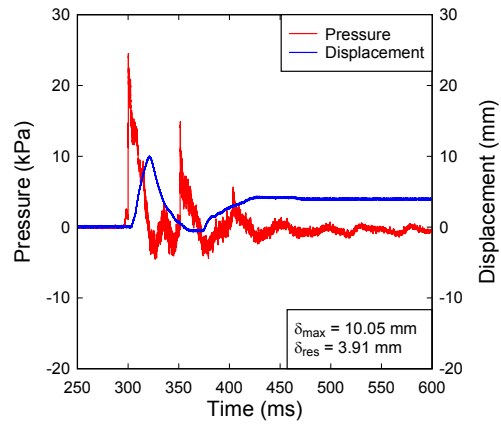
Pressure and Impulse Time History

HSC-F1-15M-0 - Blast 1: 9ft Driver, $P_d = 17$ psi



Pressure and Displacement Time History

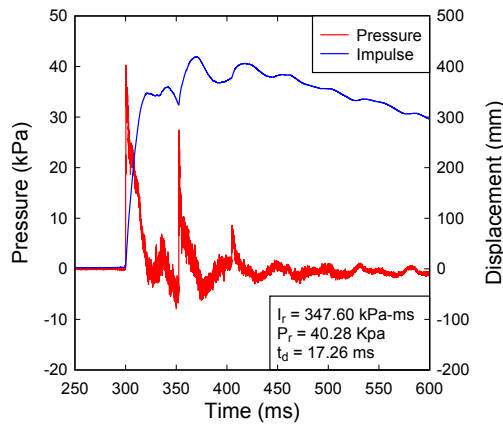
HSC-F1-15M-0 - Blast 1: 9ft Driver, $P_d = 17$ psi



a) Blast 1: Reflected pressure, impulse, and displacement time histories.

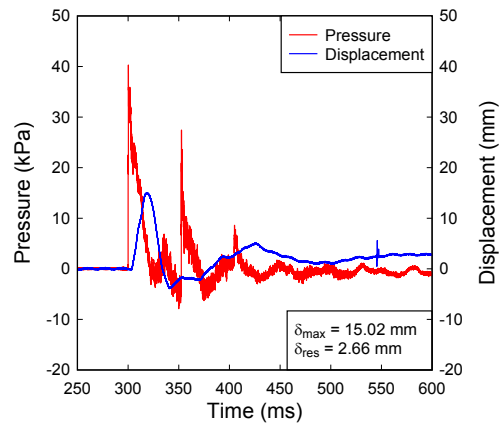
Pressure and Impulse Time History

HSC-F1-15M-0 - Blast 2: 9ft Driver, $P_d = 30$ psi



Pressure and Displacement Time History

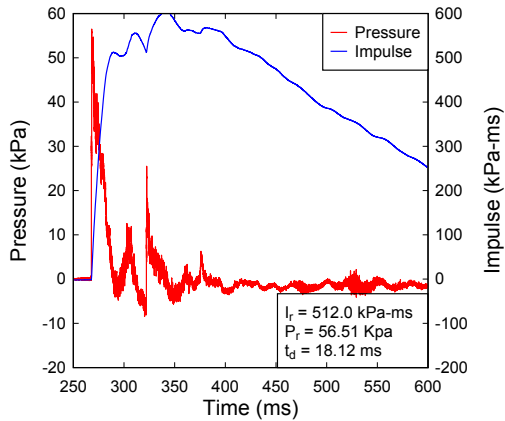
HSC-F1-15M-0 - Blast 2: 9ft Driver, $P_d = 30$ psi



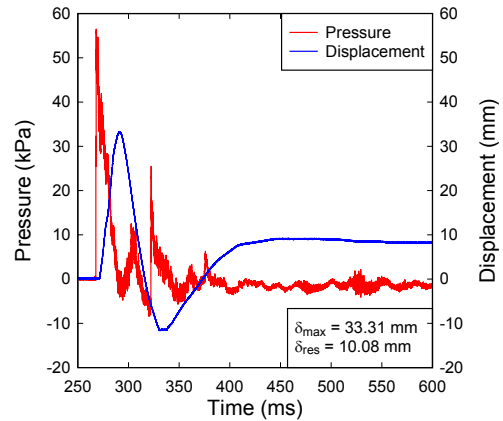
b) Blast 2: Reflected pressure, impulse, and displacement time histories.

Figure 6-10 HSC-F1(ZP)-15M-0, recorded reflected pressure, impulse and displacement for Blast 1-2

Pressure and Impulse Time History
HSC-F1-15M-0 - Blast 3: 9ft Driver, $P_d = 50$ psi

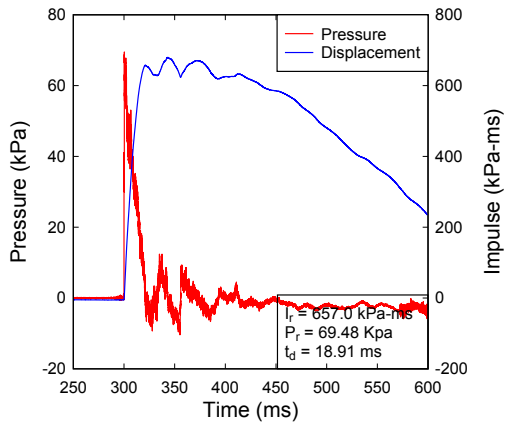


Pressure and Displacement Time History
HSC-F1-15M-0 - Blast 3: 9ft Driver, $P_d = 50$ psi

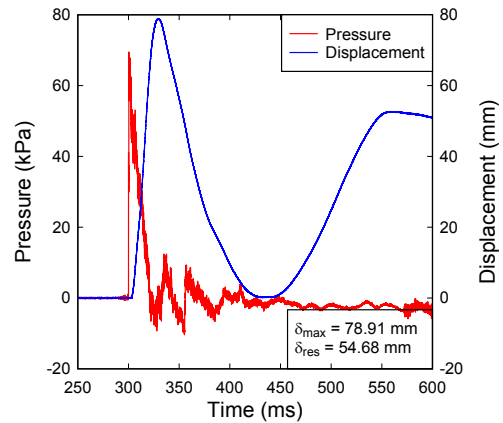


a) Blast 3: Reflected pressure, impulse, and displacement time histories.

Pressure and Impulse Time History
HSC-F1-15M-0 - Blast 4: 9ft Driver, $P_d = 70$ psi



Pressure and Displacement Time History
HSC-F1-15M-0 - Blast 4: 9ft Driver, $P_d = 70$ psi



b) Blast 4: Reflected pressure, impulse, and displacement time histories.

Figure 6-11 HSC-F1(ZP)-15M-0, recorded reflected pressure, impulse and displacement for Blast 3-4



a) Blast 1



b) Blast 2



c) Blast 3



d) Blast 4



e) Mid-span cracking
at Blast 3



f) Mid-span splitting
at Blast 4



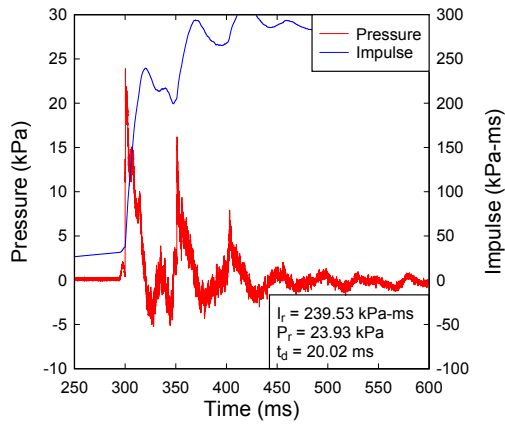
g) Mid-span fibre pulled out
at Blast 4

Figure 6-12 HSC-F1(ZP)-15M-0, photographs at the end of Blast 1-4

6.6.6 Specimen HSC-F0-20M-0

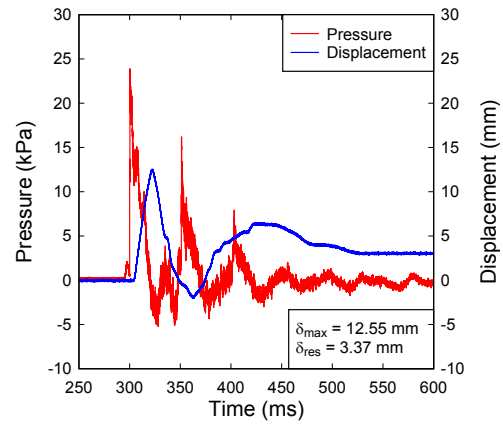
Pressure and Impulse Time History

HSC-F0-20M-0 - Blast 1: 9ft Driver, $P_d = 17$ psi



Pressure and Displacement Time History

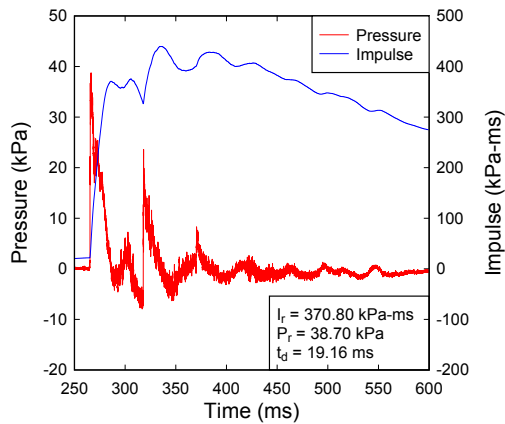
HSC-F0-20M-0 - Blast 1: 9ft Driver, $P_d = 17$ psi



a) Blast 1: Reflected pressure, impulse, and displacement time histories.

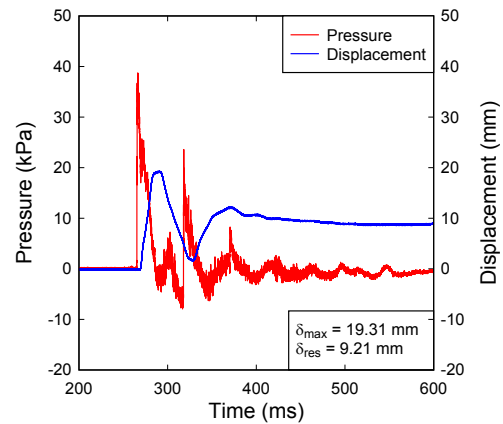
Pressure and Impulse Time History

HSC-F0-20M-0 - Blast 2: 9ft Driver, $P_d = 30$ psi



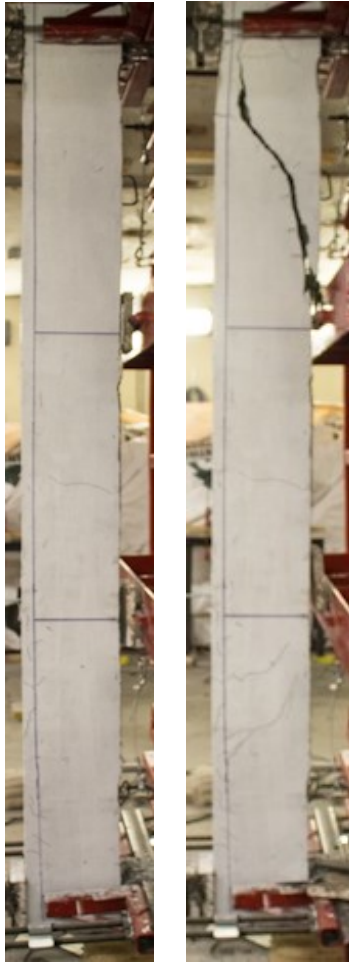
Pressure and Displacement Time History

HSC-F0-20M-0 - Blast 2: 9ft Driver, $P_d = 30$ psi



b) Blast 2: Reflected pressure, impulse, and displacement time histories.

Figure 6-13 HSC-F0-20M-0, recorded reflected pressure, impulse and displacement for Blast 1-2



a) Blast 1

b) Blast 2



c) Mid-span hairline cracks
at Blast 1



d) shear-span hairline cracks
(bottom) at Blast 2



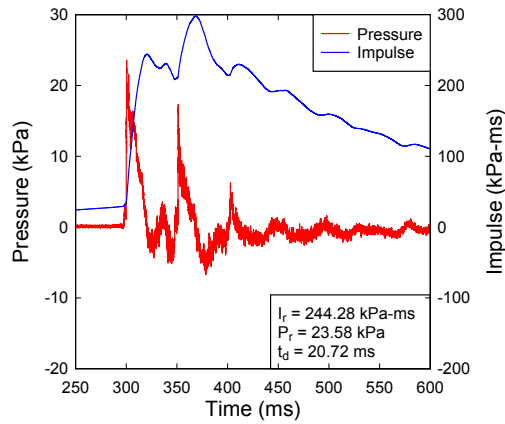
e) Shear-crack (top)
at Blast 2

Figure 6-14 HSC-F0-20M-0, photographs at the end of Blast 1-2

6.6.7 Specimen HSC-F0-20M-S

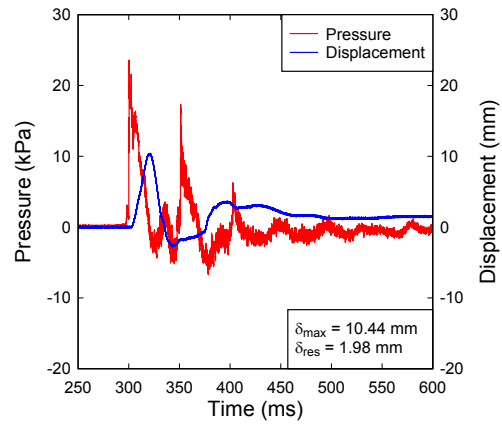
Pressure and Impulse Time History

HSC-F0-20M-S - Blast 1: 9ft Driver, $P_d = 17$ psi



Pressure and Displacement Time History

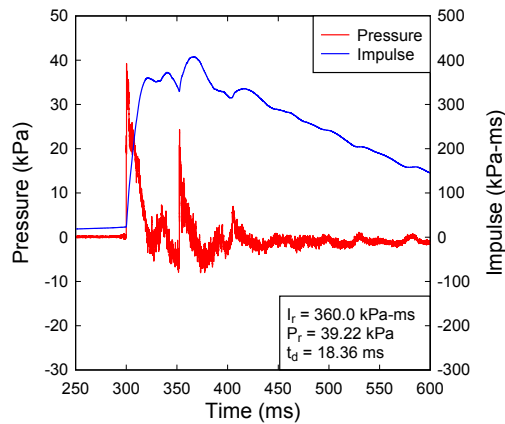
HSC-F0-20M-S - Blast 1: 9ft Driver, $P_d = 17$ psi



a) Blast 1: Reflected pressure, impulse, and displacement time histories.

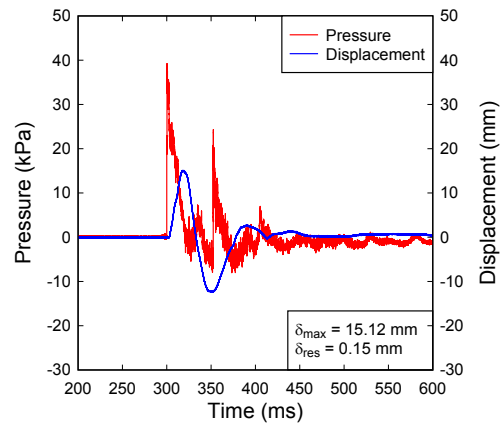
Pressure and Impulse Time History

HSC-F0-20M-S - Blast 2: 9ft Driver, $P_d = 30$ psi



Pressure and Displacement Time History

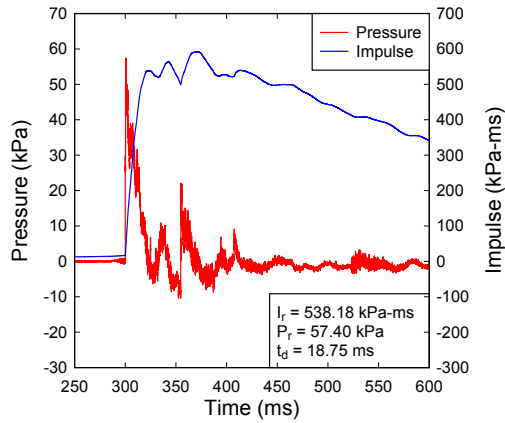
HSC-F0-20M-S - Blast 2: 9ft Driver, $P_d = 30$ psi



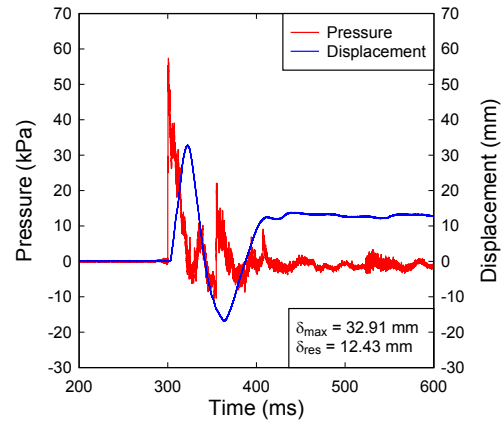
b) Blast 2: Reflected pressure, impulse, and displacement time histories.

Figure 6-15 HSC-F0-20M-S, recorded reflected pressure, impulse and displacement for Blast 1-2

Pressure and Impulse Time History
HSC-F0-20M-S - Blast 3: 9ft Driver, $P_d = 50$ psi

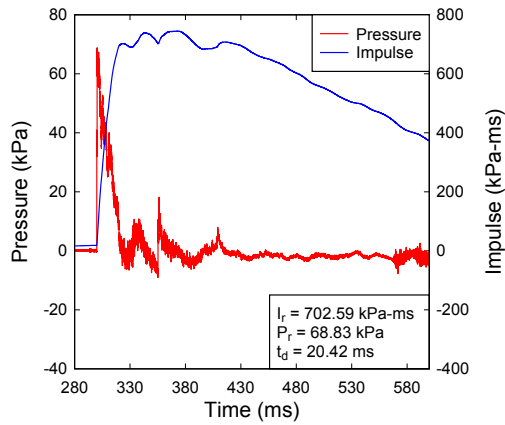


Pressure and Displacement Time History
HSC-F0-20M-S - Blast 3: 9ft Driver, $P_d = 50$ psi

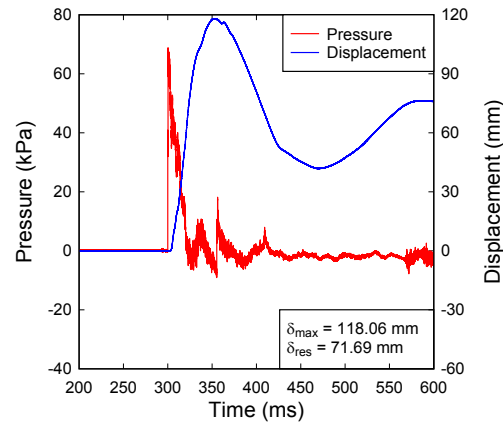


a) Blast 3: Reflected pressure, impulse, and displacement time histories.

Pressure and Impulse Time History
HSC-F0-20M-S - Blast 4: 9ft Driver, $P_d = 70$ psi



Pressure and Displacement Time History
HSC-F0-20M-S - Blast 4: 9ft Driver, $P_d = 70$ psi



b) Blast 4: Reflected pressure, impulse, and displacement time histories.

Figure 6-16 HSC-F0-20M-S, recorded reflected pressure, impulse and displacement for Blast 3-4



a) Blast 1



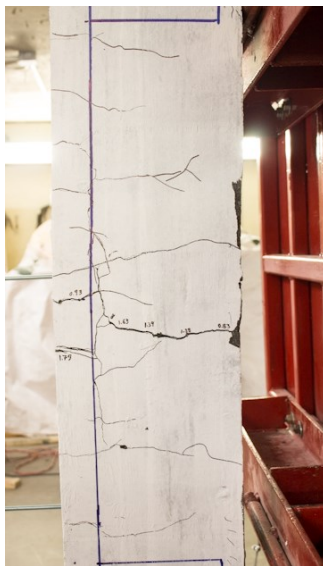
b) Blast 2



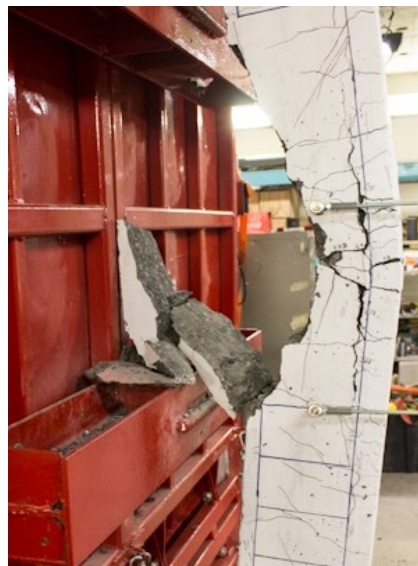
c) Blast 3



d) Blast 4



e) Mid-span cracking
at Blast 3



f) Mid-span chunks
at Blast 4

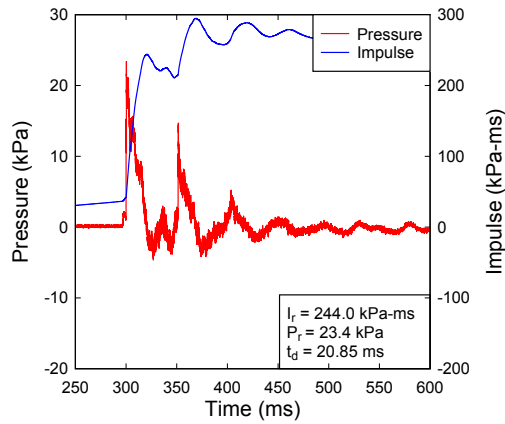


g) Mid-span crushing
at Blast 4

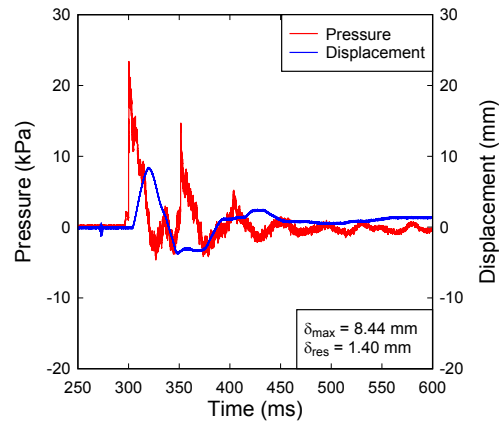
Figure 6-17 HSC-F0-20M-S, photographs at the end of Blast 1-4

6.6.8 Specimen HSC-F1(ZP)-20M-0

Pressure and Impulse Time History
HSC-F1-20M-0 - Blast 1: 9ft Driver, $P_d = 17$ psi

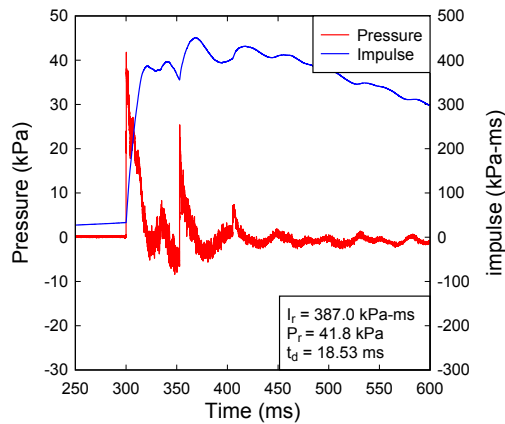


Pressure and Displacement Time History
HSC-F1-20M-0 - Blast 1: 9ft Driver, $P_d = 17$ psi

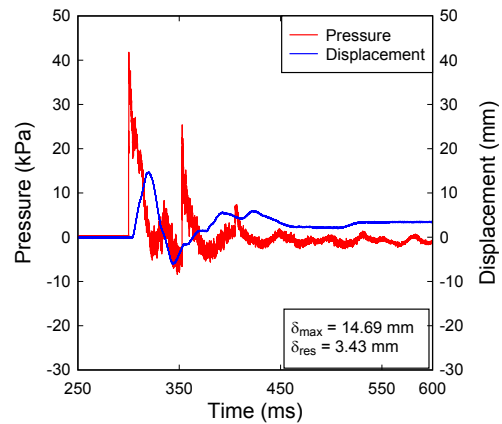


a) Blast 1: Reflected pressure, impulse, and displacement time histories.

Pressure and Impulse Time History
HSC-F1-20M-0 - Blast 2: 9ft Driver, $P_d = 30$ psi

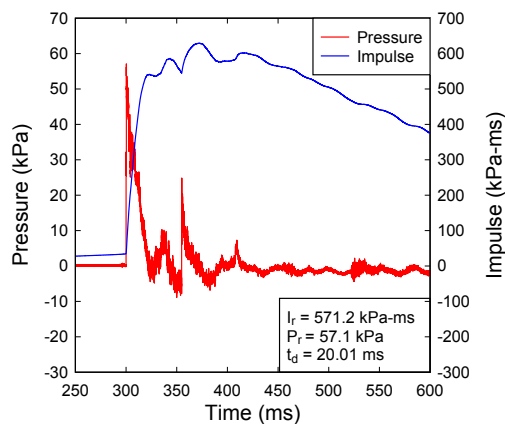


Pressure and Displacement Time History
HSC-F1-20M-0 - Blast 2: 9ft Driver, $P_d = 30$ psi

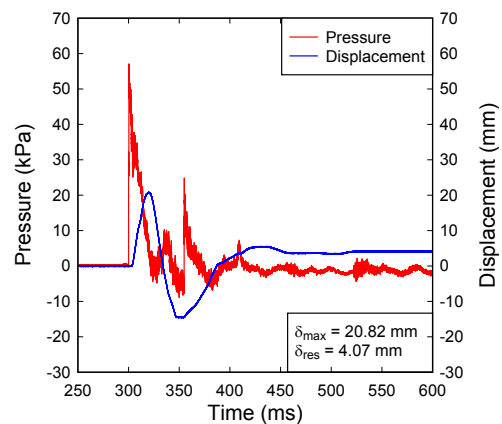


b) Blast 2: Reflected pressure, impulse, and displacement time histories.

Pressure and Impulse Time History
HSC-F1-20M-0 - Blast 3: 9ft Driver, $P_d = 50$ psi



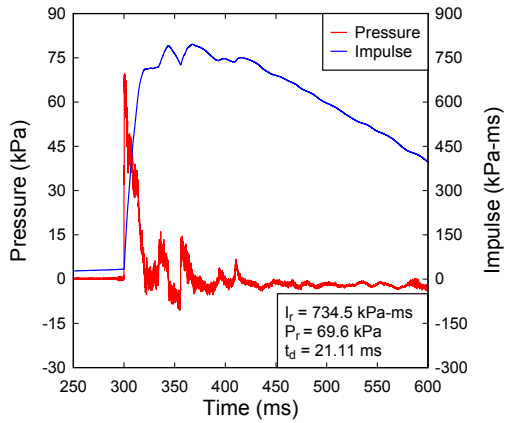
Pressure and Displacement Time History
HSC-F1-20M-0 - Blast 3: 9ft Driver, $P_d = 50$ psi



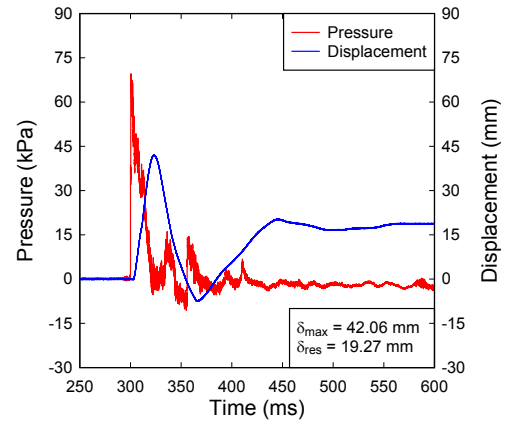
c) Blast 3: Reflected pressure, impulse, and displacement time histories.

Figure 6-18 HSC-F1(ZP)-20M-0, recorded reflected pressure, impulse and displacement for Blast 1-3

Pressure and Impulse Time History
HSC-F1-20M-0 - Blast 4: 9ft Driver, $P_d = 70$ psi

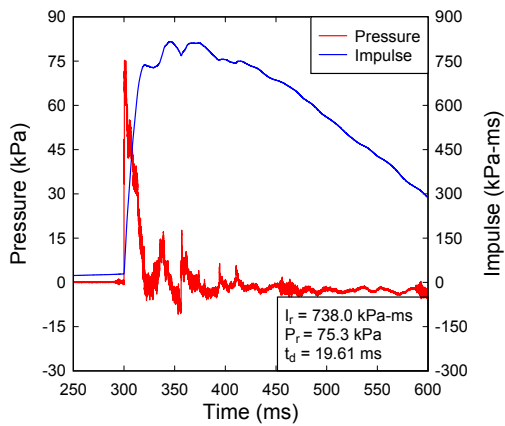


Pressure and Displacement Time History
HSC-F1-20M-0 - Blast 4: 9ft Driver, $P_d = 70$ psi

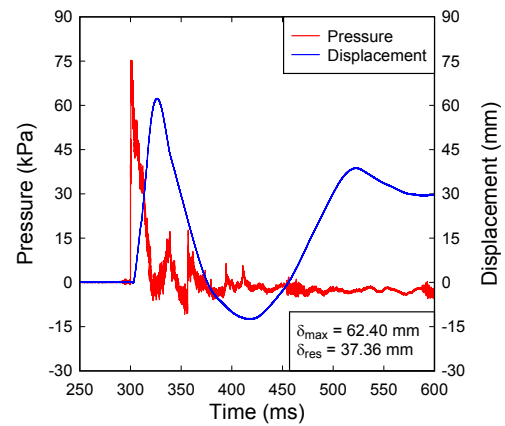


a) Blast 4: Reflected pressure, impulse, and displacement time histories.

Pressure and Impulse Time History
HSC-F1-20M-0 - Blast 5: 9ft Driver, $P_d = 80$ psi



Pressure and Displacement Time History
HSC-F1-20M-0 - Blast 5: 9ft Driver, $P_d = 80$ psi



b) Blast 5: Reflected pressure, impulse, and displacement time histories.

Figure 6-19 HSC-F1(ZP)-20M-0, recorded reflected pressure, impulse and displacement for Blast 4-5



a) Blast 1



b) Blast 2



c) Blast 3



d) Blast 4



e) Blast 5



f) Mid-span cracks
at Blast 4



g) Mid-span splitting
at Blast 5



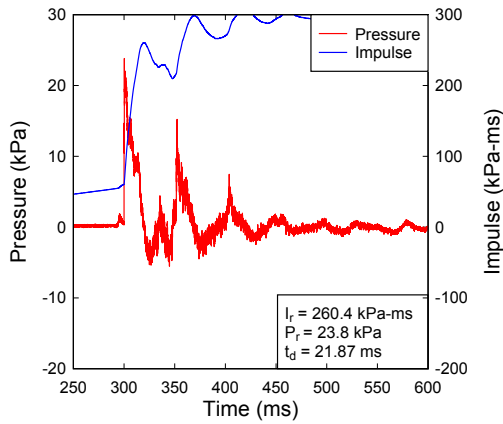
h) Mid-span fibre pulled out
at Blast 5

Figure 6-20 HSC-F1(ZP)-20M-0, photographs at the end of Blast 1-5

6.6.9 Specimen HSC-F1(ZP)-20M-S

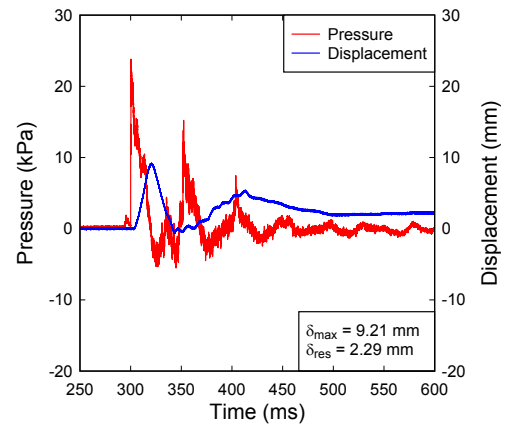
Pressure and Impulse Time History

HSC-F1-20M-S - Blast 1: 9ft Driver, $P_d = 17$ psi



Pressure and Displacement Time History

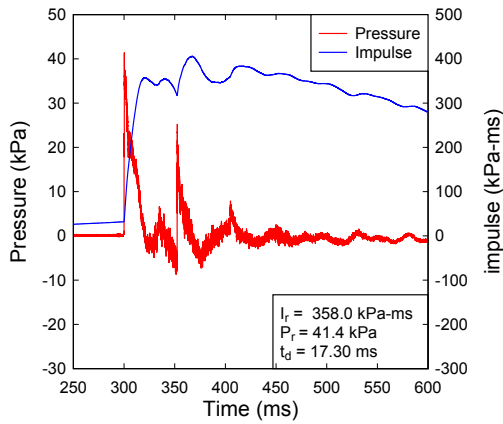
HSC-F1-20M-S - Blast 1: 9ft Driver, $P_d = 17$ psi



a) Blast 1: Reflected pressure, impulse, and displacement time histories.

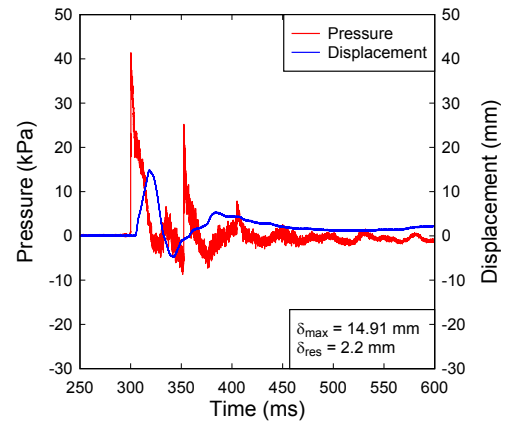
Pressure and Impulse Time History

HSC-F1-20M-S - Blast 2: 9ft Driver, $P_d = 30$ psi



Pressure and Displacement Time History

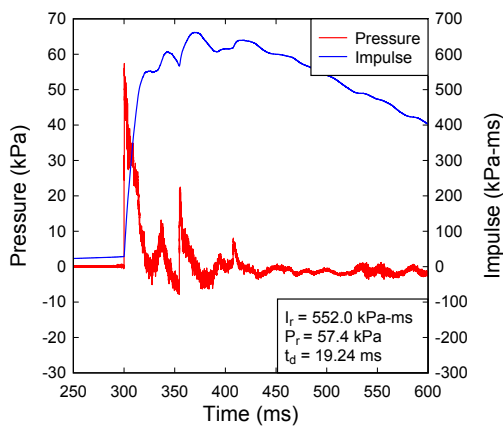
HSC-F1-20M-S - Blast 2: 9ft Driver, $P_d = 30$ psi



b) Blast 2: Reflected pressure, impulse, and displacement time histories.

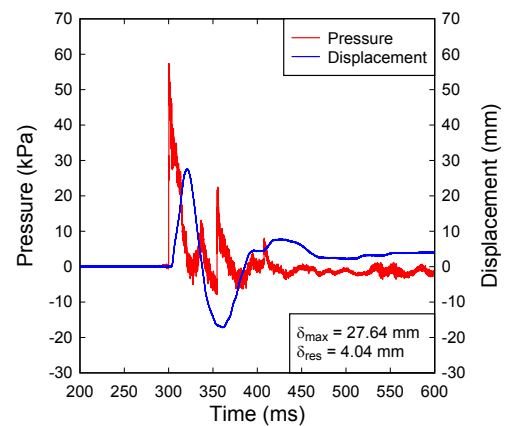
Pressure and Impulse Time History

HSC-F1-20M-S - Blast 3: 9ft Driver, $P_d = 50$ psi



Pressure and Displacement Time History

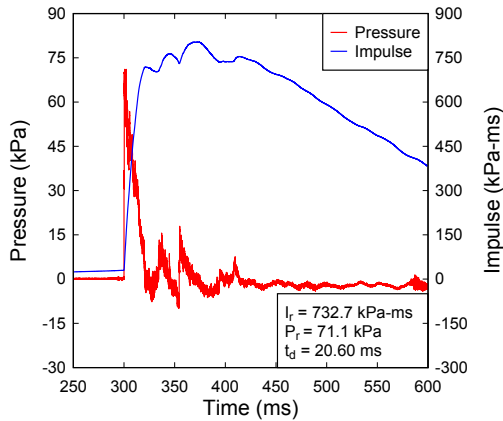
HSC-F1-20M-S - Blast 3: 9ft Driver, $P_d = 50$ psi



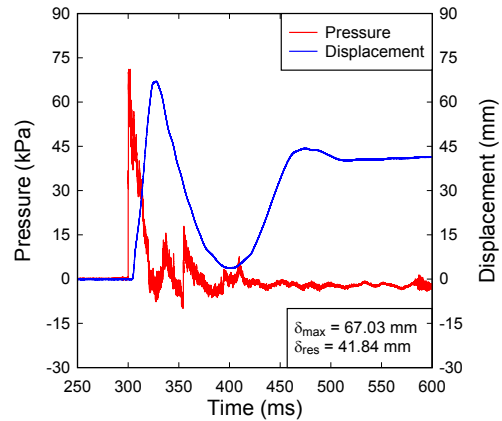
c) Blast 3: Reflected pressure, impulse, and displacement time histories.

Figure 6-21 HSC-F1(ZP)-20M-S, recorded reflected pressure, impulse and displacement for Blast 1-3

Pressure and Impulse Time History
HSC-F1-20M-S - Blast 4: 9ft Driver, $P_d = 70$ psi

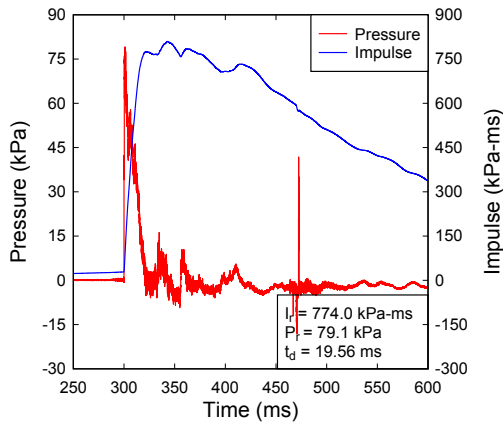


Pressure and Displacement Time History
HSC-F1-20M-S - Blast 4: 9ft Driver, $P_d = 70$ psi

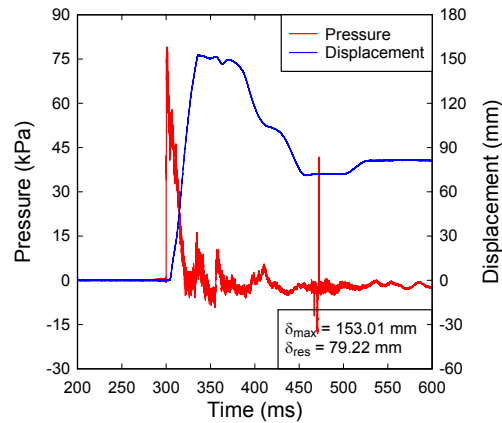


a) Blast 4: Reflected pressure, impulse, and displacement time histories.

Pressure and Impulse Time History
HSC-F1-20M-S - Blast 5: 9ft Driver, $P_d = 80$ psi



Pressure and Displacement Time History
HSC-F1-20M-S - Blast 5: 9ft Driver, $P_d = 80$ psi



b) Blast 5: Reflected pressure, impulse, and displacement time histories.

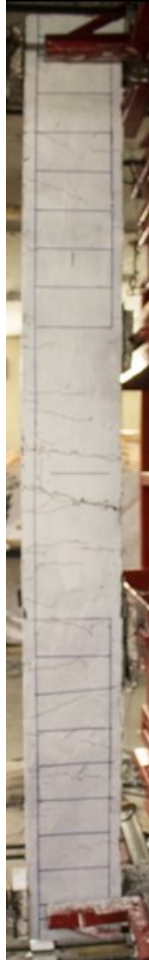
Figure 6-22 HSC-F1(ZP)-20M-S, recorded reflected pressure, impulse and displacement for Blast 4-5



a) Blast 1



b) Blast 2



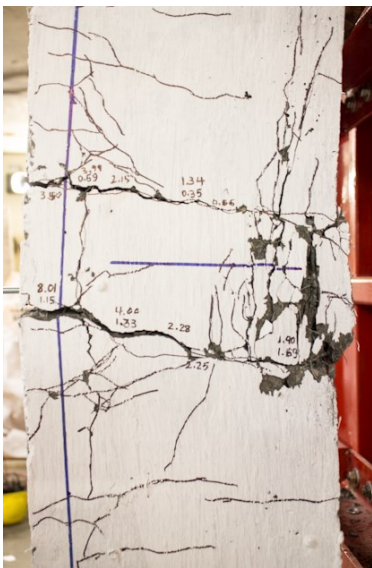
c) Blast 3



d) Blast 4



e) Blast 5



f) Mid-span cracking
at Blast 4



g) Mid-span crushing
at Blast 5



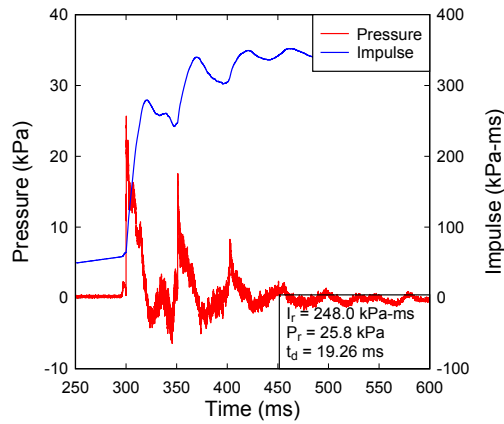
h) Mid-span fibre pulled out
at Blast 5

Figure 6-23 HSC-F1(ZP)-20M-S, photographs at the end of Blast 1-5

6.6.10 Specimen HSC-F0.5 (ZP)-20M-S

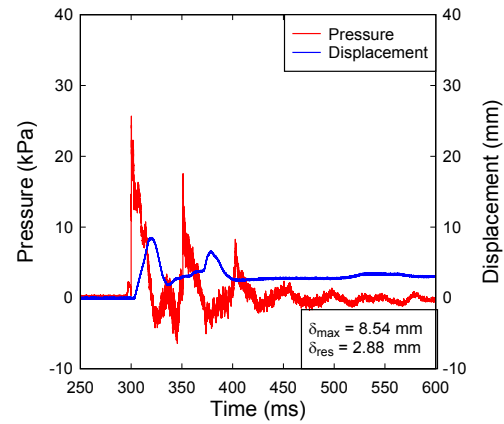
Pressure and Impulse Time History

HSC-F0.5(ZP)-20M-S - Blast 1: 9ft Driver, $P_d = 17$ psi



Pressure and Displacement Time History

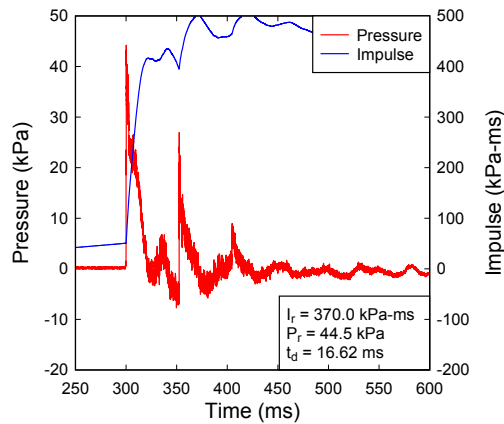
HSC-F0.5(ZP)-20M-S - Blast 1: 9ft Driver, $P_d = 17$ psi



a) Blast 1: Reflected pressure, impulse, and displacement time histories.

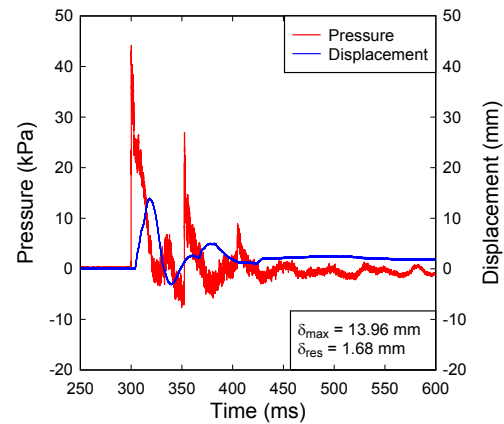
Pressure and Impulse Time History

HSC-F0.5(ZP)-20M-S - Blast 2: 9ft Driver, $P_d = 30$ psi



Pressure and Displacement Time History

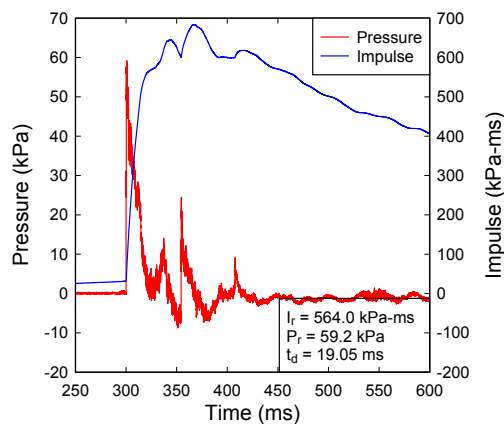
HSC-F0.5(ZP)-20M-S - Blast 2: 9ft Driver, $P_d = 30$ psi



b) Blast 2: Reflected pressure, impulse, and displacement time histories.

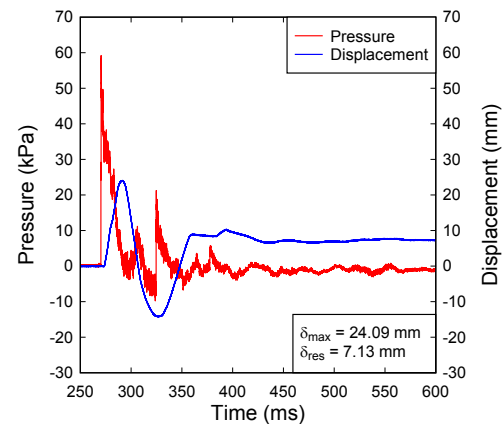
Pressure and Impulse Time History

HSC-F0.5(ZP)-20M-S - Blast 3: 9ft Driver, $P_d = 50$ psi



Pressure and Displacement Time History

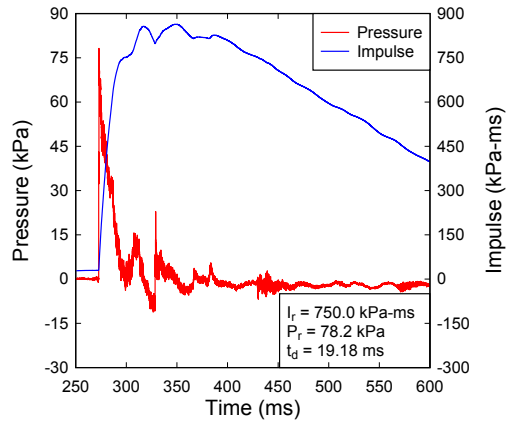
HSC-F0.5(ZP)-20M-S - Blast 3: 9ft Driver, $P_d = 50$ psi



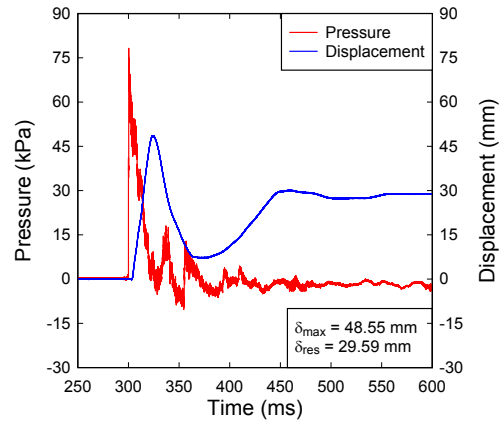
c) Blast 3: Reflected pressure, impulse, and displacement time histories.

Figure 6-24 HSC-F0.5 (ZP)-20M-S, recorded reflected pressure, impulse and displacement for Blast 1-3

Pressure and Impulse Time History
HSC-F0.5(ZP)-20M-S - Blast 4: 9ft Driver, $P_d = 70$ psi

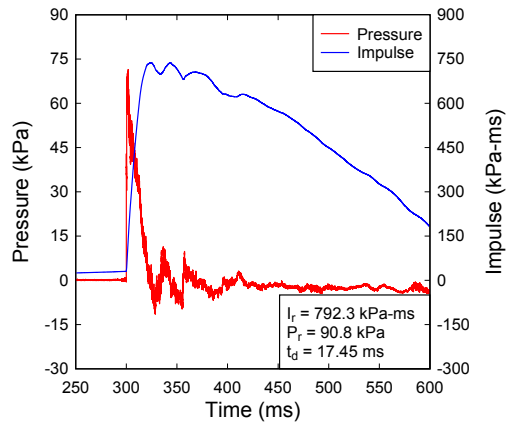


Pressure and Displacement Time History
HSC-F0.5(ZP)-20M-S - Blast 4: 9ft Driver, $P_d = 70$ psi

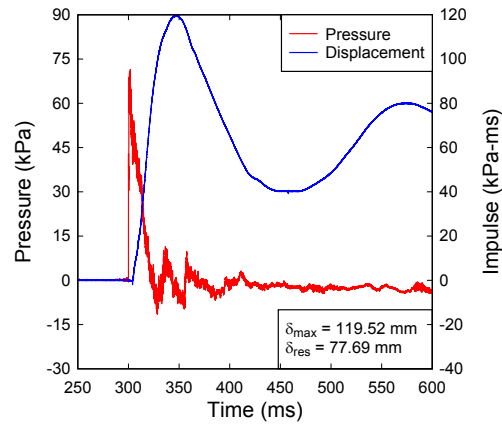


a) Blast 4: Reflected pressure, impulse, and displacement time histories.

Pressure and Impulse Time History
HSC-F0.5(ZP)-20M-S - Blast 5: 9ft Driver, $P_d = 80$ psi



Pressure and Displacement Time History
HSC-F0.5(ZP)-20M-S - Blast 5: 9ft Driver, $P_d = 80$ psi



b) Blast 5: Reflected pressure, impulse, and displacement time histories.

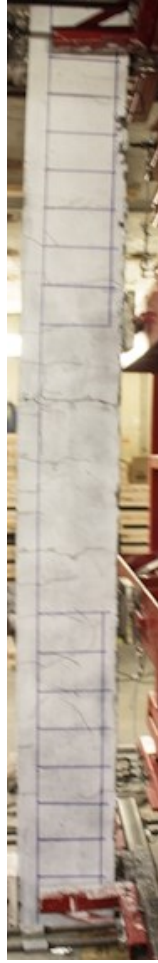
Figure 6-25 HSC-F0.5 (ZP)-20M-S, recorded reflected pressure, impulse and displacement for Blast 4-5



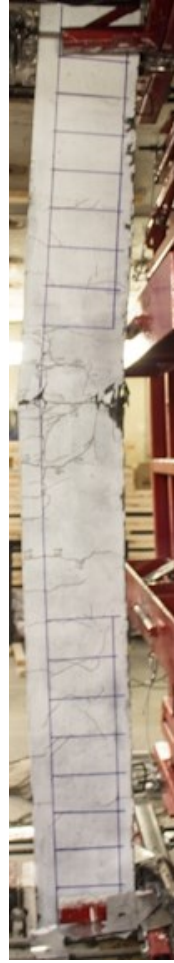
a) Blast 1



b) Blast 2



c) Blast 3



d) Blast 4



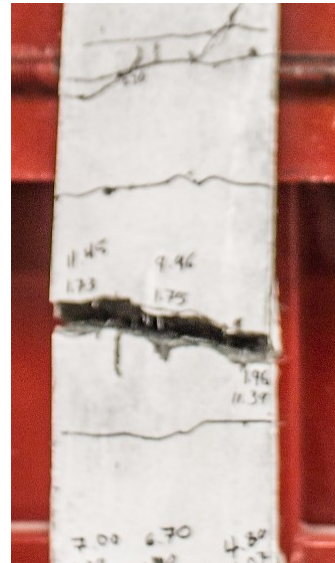
e) Blast 5



f) Mid-span cracking
at Blast 4



g) Mid-span crushing
at Blast 5

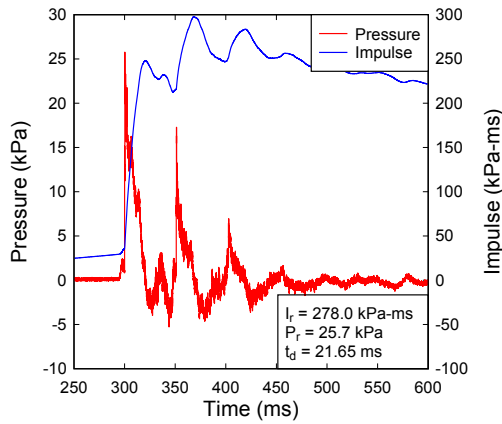


h) Mid-span fibre pulled out
at Blast 5

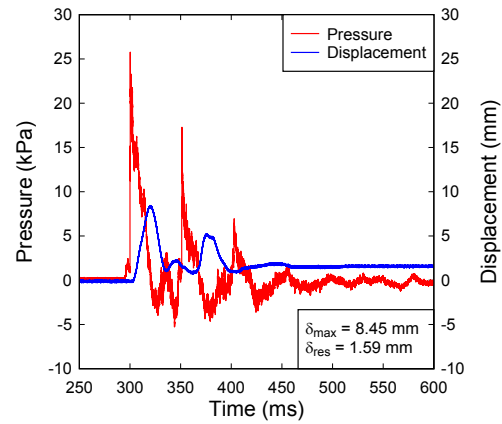
Figure 6-26 HSC-F0.5 (ZP)-20M-S, photographs at the end of Blast 1-5

6.6.11 Specimen HSC-F0.5 (5D)-20M-S

Pressure and Impulse Time History
HSC-F0.5(5D)-20M-S - Blast 1: 9ft Driver, $P_d = 17$ psi

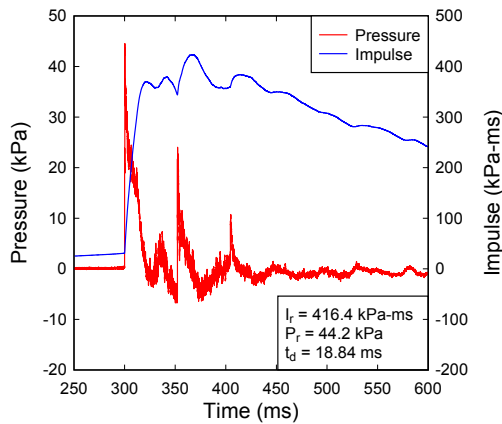


Pressure and Displacement Time History
HSC-F0.5(5D)-20M-S - Blast 1: 9ft Driver, $P_d = 17$ psi

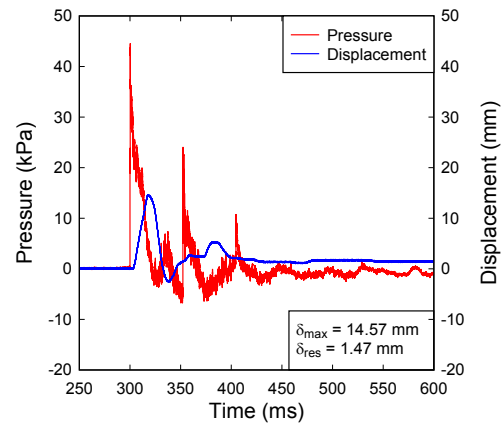


a) Blast 1: Reflected pressure, impulse, and displacement time histories.

Pressure and Impulse Time History
HSC-F0.5(5D)-20M-S - Blast 2: 9ft Driver, $P_d = 30$ psi

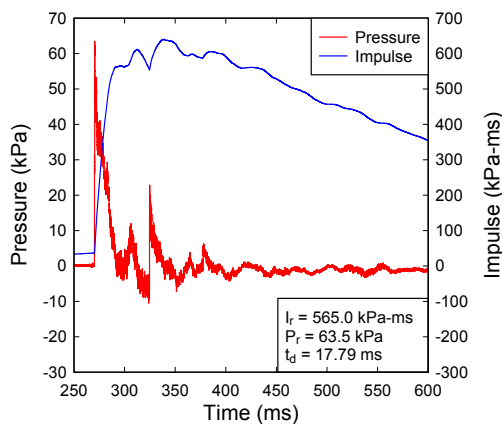


Pressure and Displacement Time History
HSC-F0.5(5D)-20M-S - Blast 2: 9ft Driver, $P_d = 30$ psi

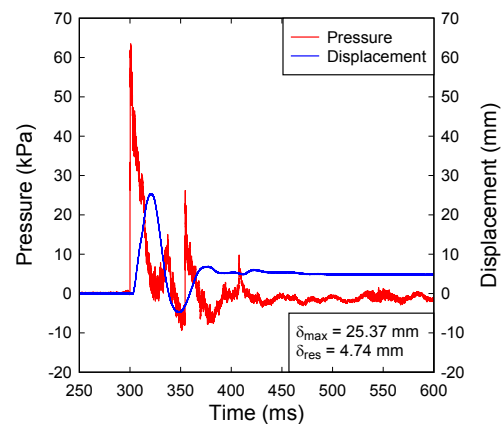


b) Blast 2: Reflected pressure, impulse, and displacement time histories.

Pressure and Impulse Time History
HSC-F0.5(5D)-20M-S - Blast 3: 9ft Driver, $P_d = 50$ psi



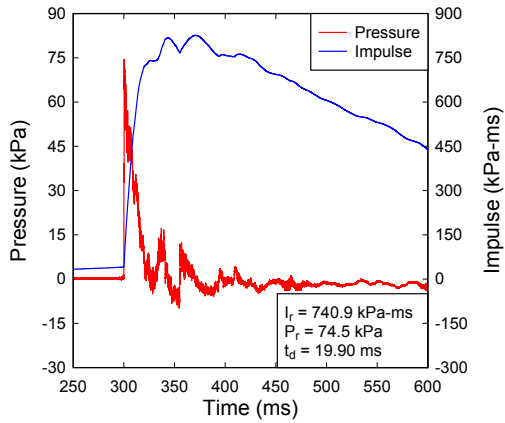
Pressure and Displacement Time History
HSC-F0.5(5D)-20M-S - Blast 3: 9ft Driver, $P_d = 50$ psi



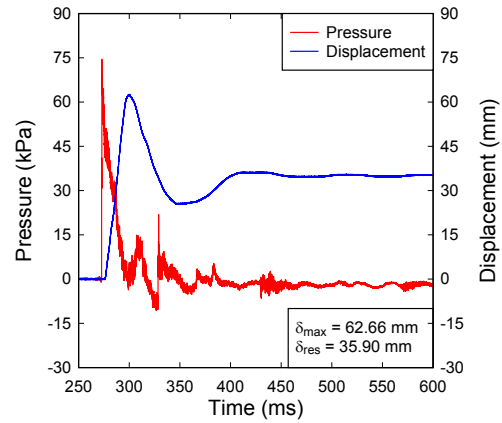
c) Blast 3: Reflected pressure, impulse, and displacement time histories.

Figure 6-27 HSC-F0.5 (5D)-20M-S, recorded reflected pressure, impulse and displacement for Blast 1-3

Pressure and Impulse Time History
HSC-F0.5(5D)-20M-S - Blast 4: 9ft Driver, $P_d = 70$ psi

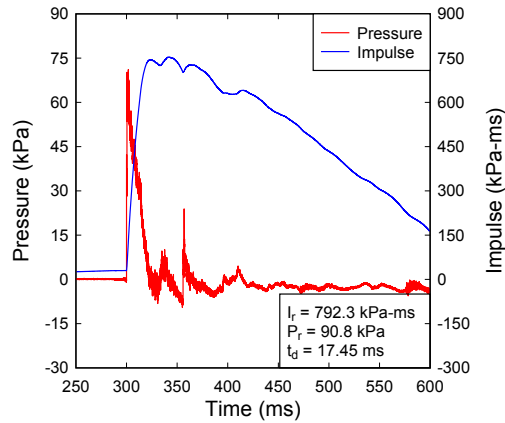


Pressure and Displacement Time History
HSC-F0.5(5D)-20M-S - Blast 4: 9ft Driver, $P_d = 70$ psi

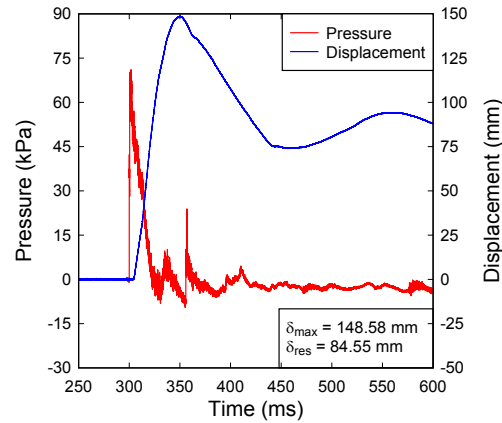


a) Blast 4: Reflected pressure, impulse, and displacement time histories.

Pressure and Impulse Time History
HSC-F0.5(5D)-20M-S - Blast 5: 9ft Driver, $P_d = 80$ psi



Pressure and Displacement Time History
HSC-F0.5(5D)-20M-S - Blast 5: 9ft Driver, $P_d = 80$ psi



b) Blast 5: Reflected pressure, impulse, and displacement time histories.

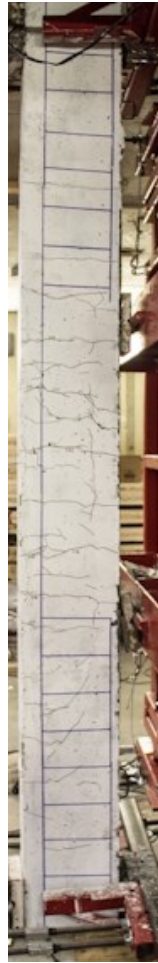
Figure 6-28 HSC-F0.5 (5D)-20M-S, recorded reflected pressure, impulse and displacement for Blast 4-5



a) Blast 1



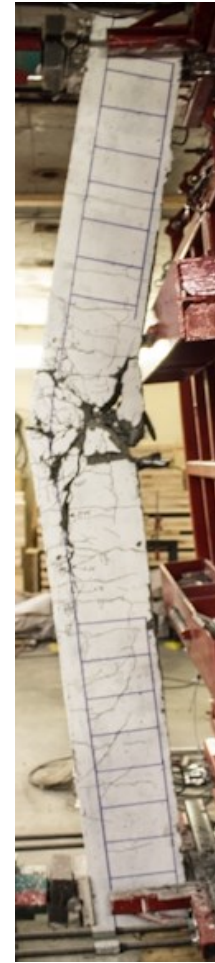
b) Blast 2



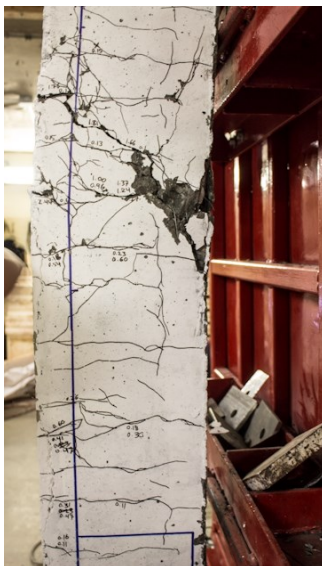
c) Blast 3



d) Blast 4



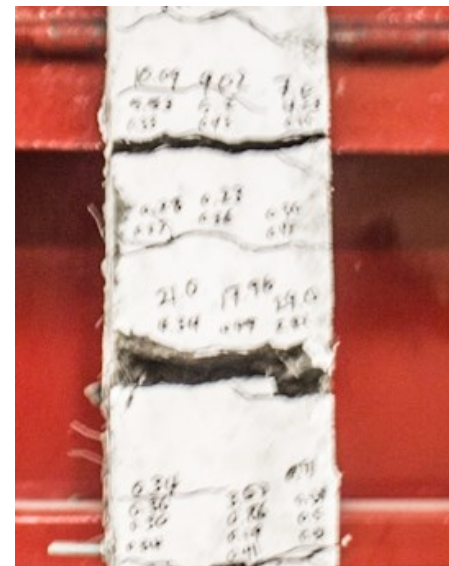
e) Blast 5



f) Mid-span cracking
at Blast 4



g) Mid-span crushing
at Blast 5



h) Mid-span fibre pulled out
at Blast 5

Figure 6-29 HSC-F0.5 (5D)-20M-S, photographs at the end of Blast 1-5

Chapter 7: Discussion of Dynamic Experimental Results

7.1 Chapter Overview

This chapter examines the differences in experimental behaviours between the eleven NSC, HSC, and HSFRC beams which were tested under dynamic loading. The effect of the test parameters are compared in terms of their effect on maximum and residual displacements, failure modes, failure blast load magnitudes, crack control, damage and secondary fragmentation. The chapter is divided into several sections discussing different aspects as follows:

The effects of the reinforcement ratio

- HSC with shear reinforcement
 - HSC-F0-#4-S vs. HSC-F0-15M-S vs. HSC-F0-20M-S
- HSC with a 1% volume of steel fibres
 - HSC-F1(ZP)-#4-0 vs. HSC-F1(ZP)-15M-0 vs. HSC-F1(ZP)-20M-0

The effects of the concrete strength and stirrups in HSC beams

- Series #4 beams with NSC vs. HSC
 - NSC-F0-#4-S vs. HSC-F0-#4-S
- Series 20M HSC beams with and without stirrups
 - HSC-F0-20M-0 vs. HSC-F0-20M-S

The effects of the steel fibres

- HSFRC vs. HSC beams with shear reinforcement
 - HSC-F0-#4-S vs. HSC-F1(ZP)-#4-0
 - HSC-F0-15M-S vs. HSC-F1(ZP)-15M-0
 - HSC-F0-20M-S vs. HSC-F1(ZP)-20M-0
- HSFRC vs. HSC beam without shear reinforcement
 - HSC-F0-20M-0 vs. HSC-F1(ZP)-20M-0

The effects of the steel fibre properties

- Steel fibre volumetric ratio
 - HSC-F0-20M-S vs. HSC-F0.5(ZP)-20M-S vs. HSC-F1(ZP)-20M-S
- Steel fibre type
 - HSC-F0.5(ZP)-20M-S vs. HSC-F0.5(5D)-20M-S

The effects of combined use of stirrups and fibres

- HSC beams with a 1% volume of steel fibres
 - HSC-F1(ZP)-20M-0 vs. HSC-F1(ZP)-20M-S

7.2 General Observations

The experimental parameters are described in general in this section. Plots showing the maximum and residual mid-span displacements for each blast test are summarized in Figure 7-1 through Figure 7-7, along with notes on particular failure modes. A brief examination of the findings in these tables and charts shows improvements in the behaviour of the beams with the increase in longitudinal reinforcement ratio, provision of stirrups and use of steel fibres. Other parameters such as concrete strength, fibre content and fibre type are found to have more moderate effect on beam response. Details on the effects of the various test variables are discussed in the subsequent sections.

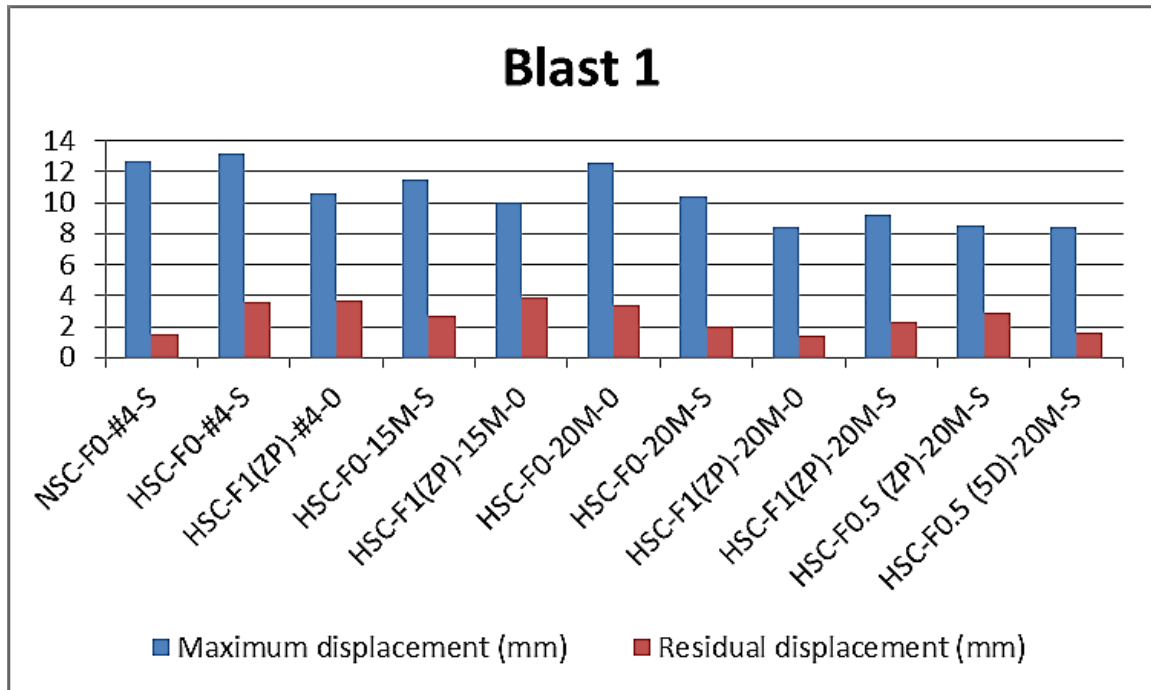


Figure 7-1 Maximum and residual displacements for blast 1

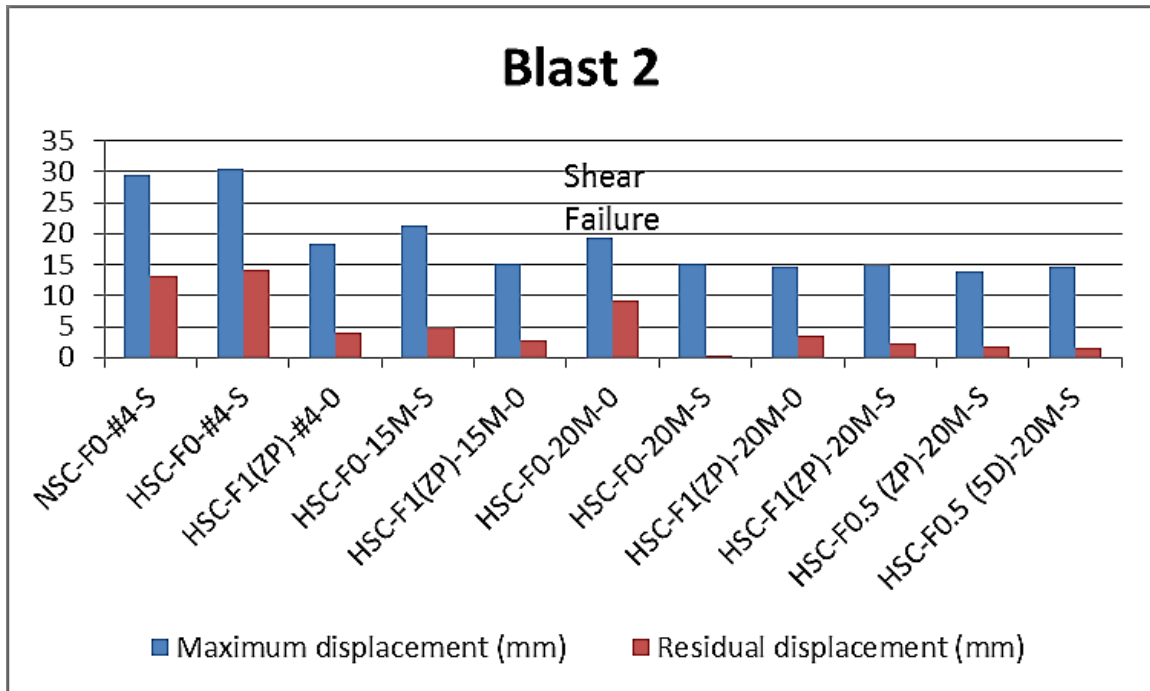


Figure 7-2 Maximum and residual displacements for blast 2

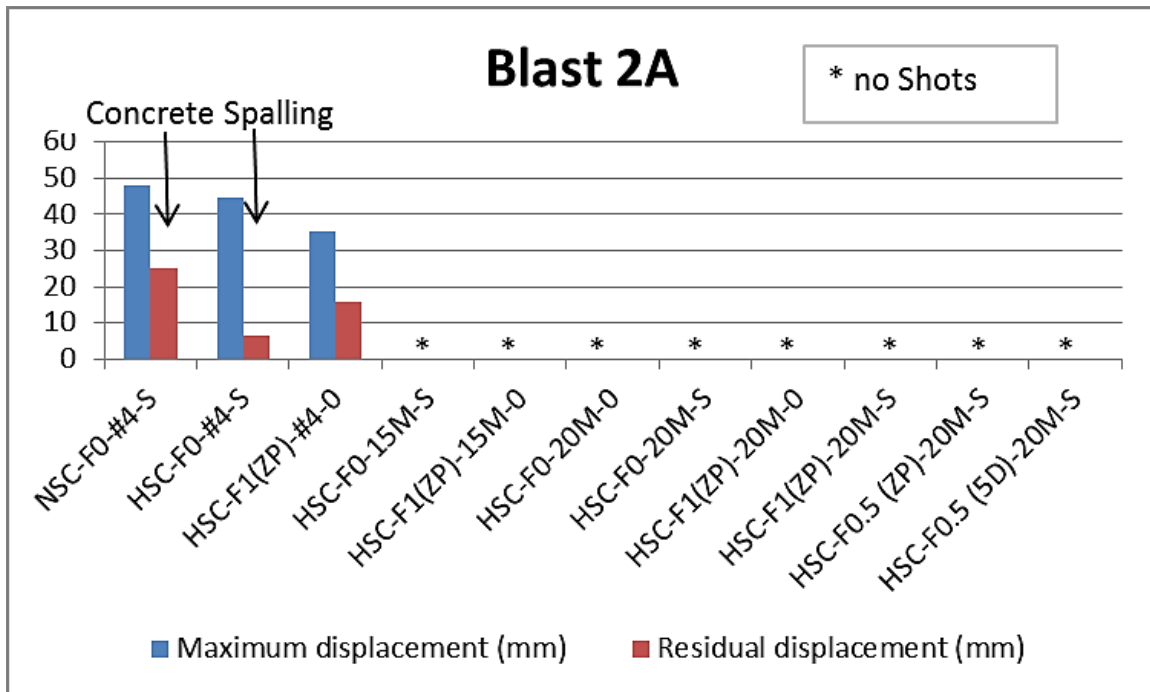


Figure 7-3 Maximum and residual displacements for blast 2A

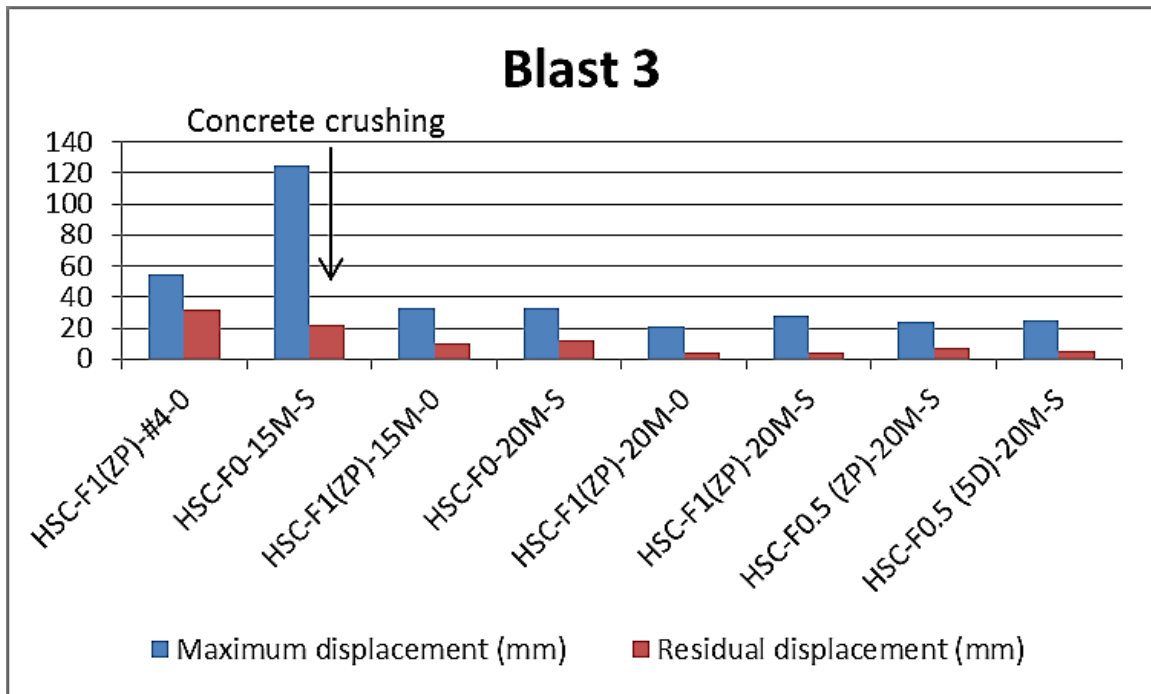


Figure 7-4 Maximum and residual displacements for blast 3

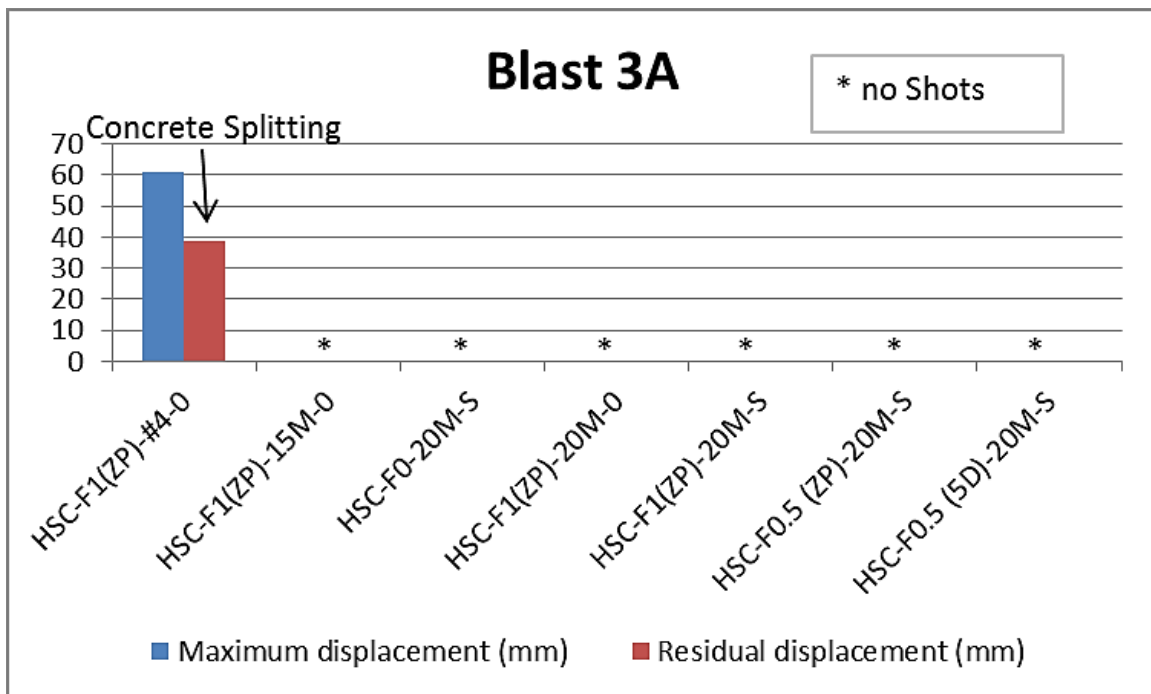


Figure 7-5 Maximum and residual displacements for blast 3A

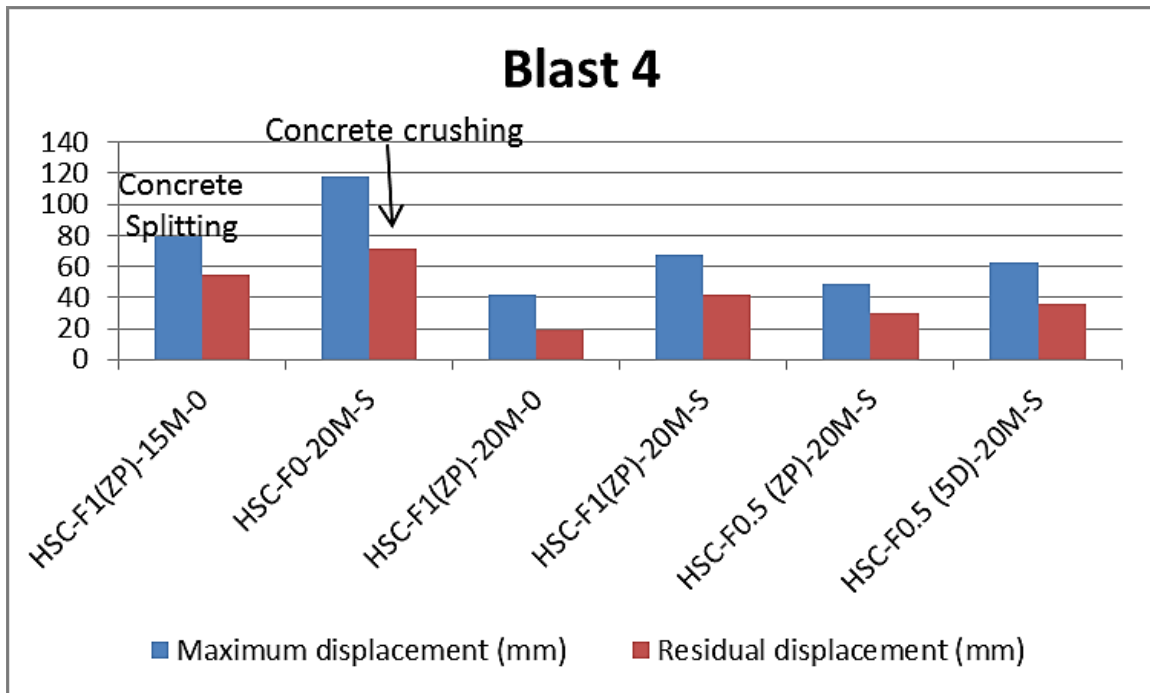


Figure 7-6 Maximum and residual displacements for blast 4

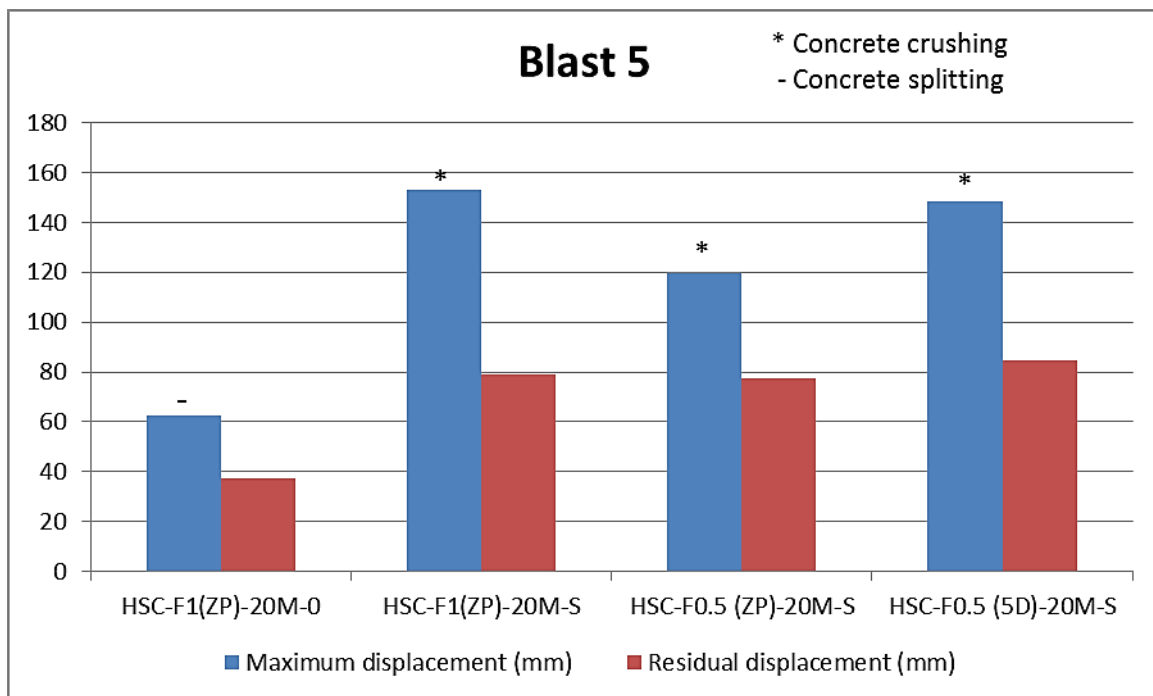


Figure 7-7: Maximum and residual displacements for blast 5

7.3 Effects of the Reinforcement Ratio

Based on the data obtained from the experimental program, the reinforced concrete beams (with and without fibres) showed improved blast performance as the reinforcement ratio was increased. The following sub-sections demonstrate this observation for each type of concrete mix.

7.3.1 Effects of the Reinforcement Ratio: HSC Beams without Steel Fibres

High strength concrete specimens HSC-F0-#4-S, HSC-F0-15M-S, and HSC-F0-20M-S were designed with varying amounts of longitudinal steel reinforcement which consisted of 2-#4, 2-15M and 2-20M bars respectively. All three beams contained shear reinforcement arranged at spacing of 100 mm in the shear spans. Photographs showing the condition of the beams at failure are included in Figure 7-8. Comparative displacement bar charts and time histories for blasts 1 and 2 are provided in Figure 7-9 and Figure 7-10.

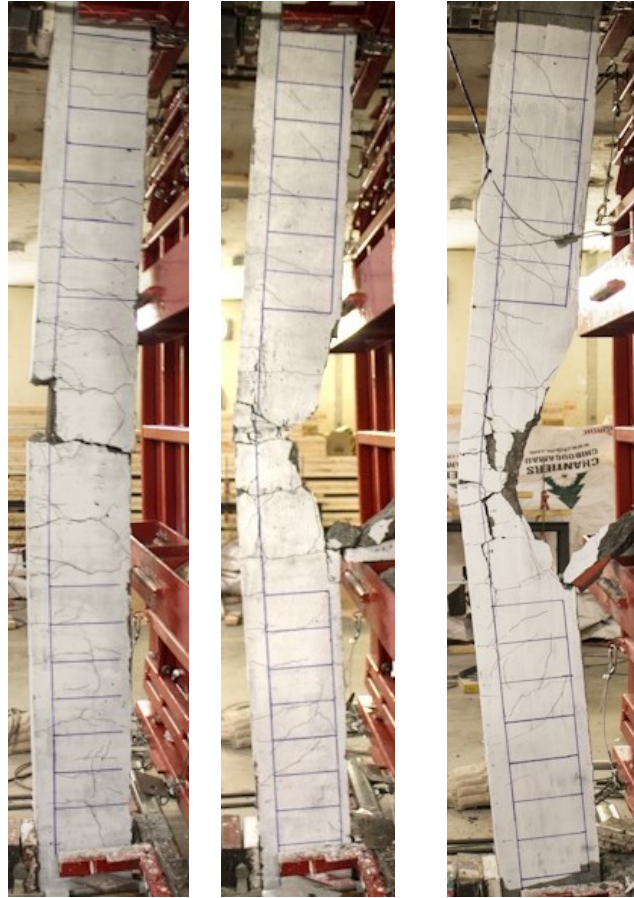
The HSC-F0-20M-S specimen, which had 20M bars, showed improved ability to resist blast loads with reduced mid-span displacements at equivalent blasts when compared to the other companion specimens with 15M and #4 bars. At Blast 1, the maximum and residual displacements for the HSC-F0-20M-S specimen were $\delta_{\max} = 10.44$ mm and $\delta_{\text{res}} = 1.98$ mm, showing reductions of 21% for maximum displacement and 45% for residual displacement when compared to the HSC-F0-#4-S specimen ($\delta_{\max} = 13.15$ mm and $\delta_{\text{res}} = 3.62$ mm). At blast 2, the maximum and residual mid-span displacements for HSC-F0-20M-S specimen ($\delta_{\max} = 15.12$ mm and $\delta_{\text{res}} = 0.15$ mm) were reduced by 50% and 99% respectively when compared with the HSC-F0-#4-S specimen ($\delta_{\max} = 30.44$ mm and $\delta_{\text{res}} = 14.16$ mm).

A similar trend was observed between the HSC-F0-20M-S specimen and the HSC-F0-15M-S specimen. At blast 1, the HSC-F0-20M-S specimen had maximum and residual mid-span displacements of $\delta_{\max} = 10.44$ mm and $\delta_{\text{res}} = 1.98$ mm, showing a decrease of 9% for maximum displacement and 26% for residual displacement when compared to the HSC-F0-15M-S specimen ($\delta_{\max} = 11.50$ mm and $\delta_{\text{res}} = 2.66$ mm). Similarly, at blast 2, the maximum mid-span displacement for the HSC-F0-20M-S specimen dropped by 29% ($\delta_{\max} = 15.12$ mm) and 97% for residual displacement ($\delta_{\text{res}} = 0.15$ mm) when compared to the displacements of the HSC-F0-15M-S specimen ($\delta_{\max} = 21.40$ mm and $\delta_{\text{res}} = 4.71$ mm). When compared with the HSC-F0-15M-S specimen ($\delta_{\max} = 124.0$ mm and $\delta_{\text{res}} = 22.04$ mm) at blast 3, the HSC-F0-20M-S specimen ($\delta_{\max} = 32.91$ mm and $\delta_{\text{res}} = 12.43$ mm) showed a reduction by 73% for maximum displacement and 44% for residual displacement; it is noted that the HSC-F0-15M-S specimen failed at this blast load while the HSC-F0-20M-S specimen survived this shockwave.

As for the HSC-F0-15M-S specimen, it showed similar improvement in maximum and residual displacements when compared to the HSC-F0-#4-S specimen. At blast 1, the maximum and residual mid-span displacements for the HSC-F0-15M-S specimen ($\delta_{\max} = 11.50$ mm and $\delta_{\text{res}} = 2.66$ mm) were lower by 13% and 27% respectively than the HSC-F0-#4-S specimen ($\delta_{\max} = 13.15$ mm and $\delta_{\text{res}} = 3.62$ mm). At blast 2, the reduction reached 30% for the maximum displacement ($\delta_{\max} = 21.40$ mm) and 67% for the residual displacement ($\delta_{\text{res}} = 4.71$ mm) in the HSC-F0-15M-S specimen, compared to the HSC-F0-#4-S specimen ($\delta_{\max} = 30.44$ mm and $\delta_{\text{res}} = 14.16$ mm).

In addition to improved response at equivalent blast pressures, it should be noted that the HSC-F0-20M-S specimen was capable of resisting a greater blast loading prior to failure when compared to the the two other specimens (Failure at Blast 4 vs. Blasts 3 and 2A, respectively). The failure was similar for the HSC-F0-20M-S and HSC-F0-15M-S specimens, with severe concrete crushing on the compression zone, whereas the HSC-F0-#4-S specimen failed due to spalling of the concrete in the tension zone at mid-span, as seen in Figure 7-8.

In summary the results from this comparison demonstrates that the behaviour and blast resistance of HSC beams improves with the increase in reinforcement ratio.



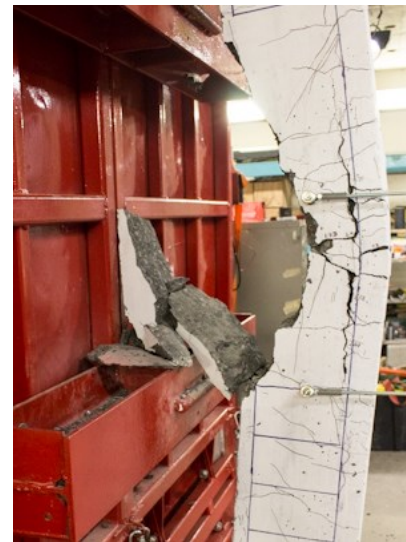
a) HSC-F0-#4-S b) HSC-F0-15M-S c) HSC-F0-20M-S



f) Mid-span spalling at Blast 2A, HSC-F0-#4-S

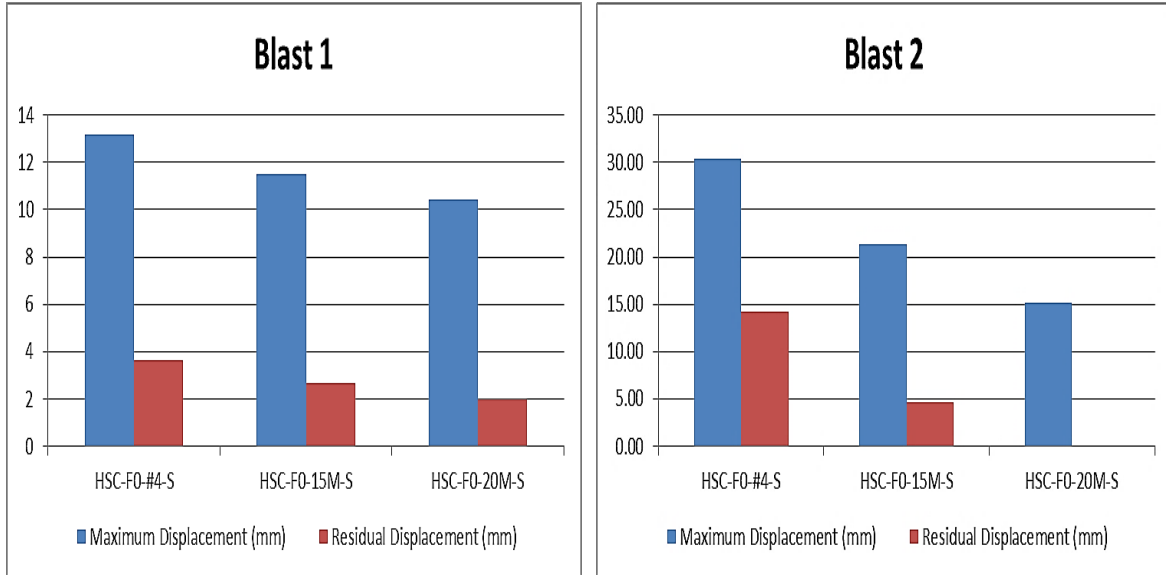


g) MID-span crushing at Blast 3, HSC-F0-15M-S



h) Mid-span crushing at Blast 4, HSC-F0-20M-S

Figure 7-8 Photographs; effects of the reinforcement ratio without steel fibres

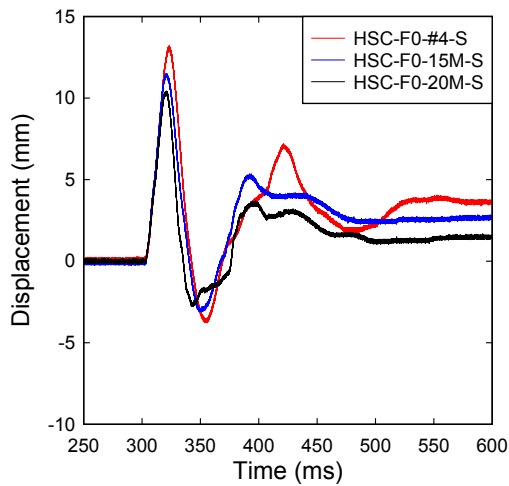


a) Blast 1

b) Blast 2

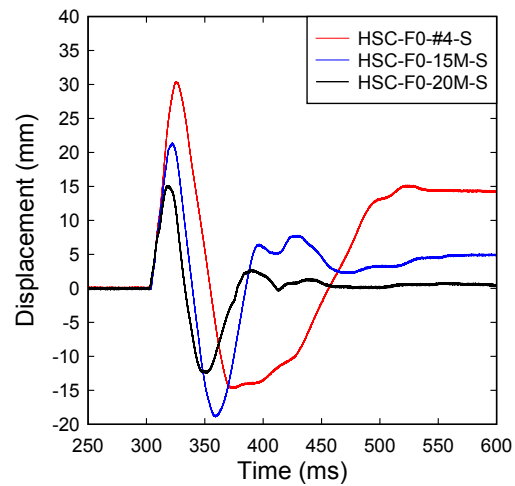
Figure 7-9 Maximum and residual displacements; effects of the reinforcement ratio without steel fibres

Displacement Time History - Blast 1
Effect of reinforcement ratio without steel fibres



a) Blast 1

Displacement Time History - Blast 2
Effect of reinforcement ratio without steel fibres



b) Blast 2

Figure 7-10 Displacement time histories; effects of the reinforcement ratio without steel fibres

7.3.2 Effects of the Reinforcement Ratio: HSC Beams with Steel Fibres

Specimens, HSC-F1(ZP)-#4-0, HSC-F1(ZP)-15M-0, and HSC-F1(ZP)-20M-0, were all designed with a volumetric ratio of 1.0% of ZP steel fibres and various amount of longitudinal reinforcement (2-#4, 2-15M and 2-20M bars). Photographs showing the specimens at failure are included in Figure 7-11. Comparative displacement bar charts and time histories for blasts 2 and 3 are provided in Figure 7-12 and Figure 7-13.

The HSC-F1(ZP)-20M-0 specimen exhibited an improved blast resistance and a reduction in mid-span displacements when compared to the other two specimens with reduced reinforcement ratio. Comparing the HSC-F1(ZP)-20M-0 specimen ($\delta_{\max} = 14.69$ mm and $\delta_{\text{res}} = 3.43$ mm) with the HSC-F1(ZP)-#4-0 specimen ($\delta_{\max} = 18.47$ mm and $\delta_{\text{res}} = 4.10$ mm) at blast 2, the HSC-F1(ZP)-20M-0 specimen showed reductions in maximum and residual displacements by factors of approximately 20% and 16%. At blast 3, the maximum and residual mid-span displacements ($\delta_{\max} = 20.82$ mm and $\delta_{\text{res}} = 4.07$ mm) for the HSC-F1(ZP)-20M-0 specimen dropped by 62% and 87% respectively in comparison to the displacements of the HSC-F1(ZP)-#4-0 specimen ($\delta_{\max} = 55.1$ mm and $\delta_{\text{res}} = 32.26$ mm).

Similarly, the HSC-F1(ZP)-20M-0 specimen resulted in better blast performance when compared to the HSC-F1(ZP)-15M-0 specimen. At blast 2, the maximum and residual displacements for the HSC-F1(ZP)-15M-0 specimen ($\delta_{\max} = 15.02$ mm and $\delta_{\text{res}} = 2.66$ mm) and HSC-F1-20M-0 specimen ($\delta_{\max} = 14.69$ mm and $\delta_{\text{res}} = 3.43$ mm) were similar. However, at blast 3, the maximum and residual mid-span displacements of the HSC-F1(ZP)-20M-0 specimen ($\delta_{\max} = 20.82$ mm and $\delta_{\text{res}} = 4.07$ mm) were reduced by 38% and 60% when compared to the HSC-F1(ZP)-15M-0 specimen ($\delta_{\max} = 33.31$ mm and $\delta_{\text{res}} = 10.08$ mm).

The use of 15M bars also effectively reduced the beam mid-span displacements when compared to the #4 bars. At blast 2, the maximum and residual mid-span displacements for the HSC-F1(ZP)-15M-0 specimen ($\delta_{\max} = 15.02$ mm and $\delta_{\text{res}} = 2.66$ mm) dropped by 19% and 35% respectively when compared to the HSC-F1(ZP)-#4-0 specimen ($\delta_{\max} = 18.47$ mm and $\delta_{\text{res}} = 4.10$ mm). At blast 3, the reduction for the HSC-F1(ZP)-15M-0 specimen ($\delta_{\max} = 33.31$ mm and $\delta_{\text{res}} = 10.08$ mm) reached 40% for the maximum displacement and 69% for the residual displacement, when compared to the HSC-F1(ZP)-#4-0 specimen ($\delta_{\max} = 55.1$ mm and $\delta_{\text{res}} = 32.26$ mm).

In terms of failure, the specimen with 20M reinforcement and 1.0% steel fibres failed at Blast 5, whereas the companions specimens with 15M and #4 reinforcement failed at Blast 4 and 3A, respectively.

Failure type was similar for all three beams. The HSC-F1(ZP)-20M-0 and HSC-F1(ZP)-15M-0 specimens failed with only one major crack at mid-span, whereas the HSC-F1(ZP)-#4-0 specimen had three major cracks along the constant moment region. However, all three specimens witnessed fibre pull out at the major crack locations, as seen in Figure 7-11.

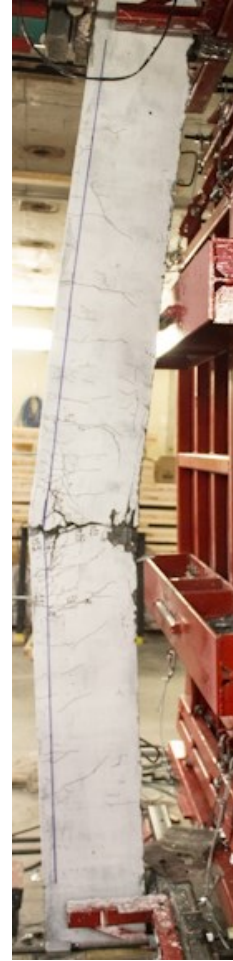
In summary, it can be concluded that the HSFRC beams benefited from increasing the ratio of steel reinforcement, with the use of 20M bars in the HSC-F1(ZP)-20M-0 specimen allowing this beam to resist greater blast loads before failure. This beam also showed a reduction in maximum and residual displacements at equivalent blast loads when compared to the remaining beams in this series.



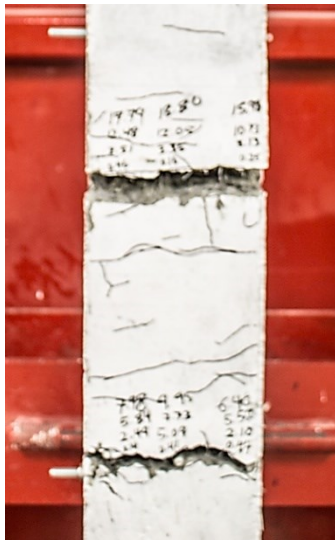
a) HSC-F1(ZP)-#4-0



b) HSC-F1(ZP)-15M-0



c) HSC-F1(ZP)-20M-0



f) Mid-span fibre pulled out at Blast 3A, HSC-F1(ZP)-#4-0



g) Mid-span fibre pulled out at Blast 4, HSC-F1(ZP)-15M-0



h) Mid-span fibre pulled out at Blast 5, HSC-F1(ZP)-20M-0

Figure 7-11 Photographs; effects of the reinforcement ratio with steel fibres

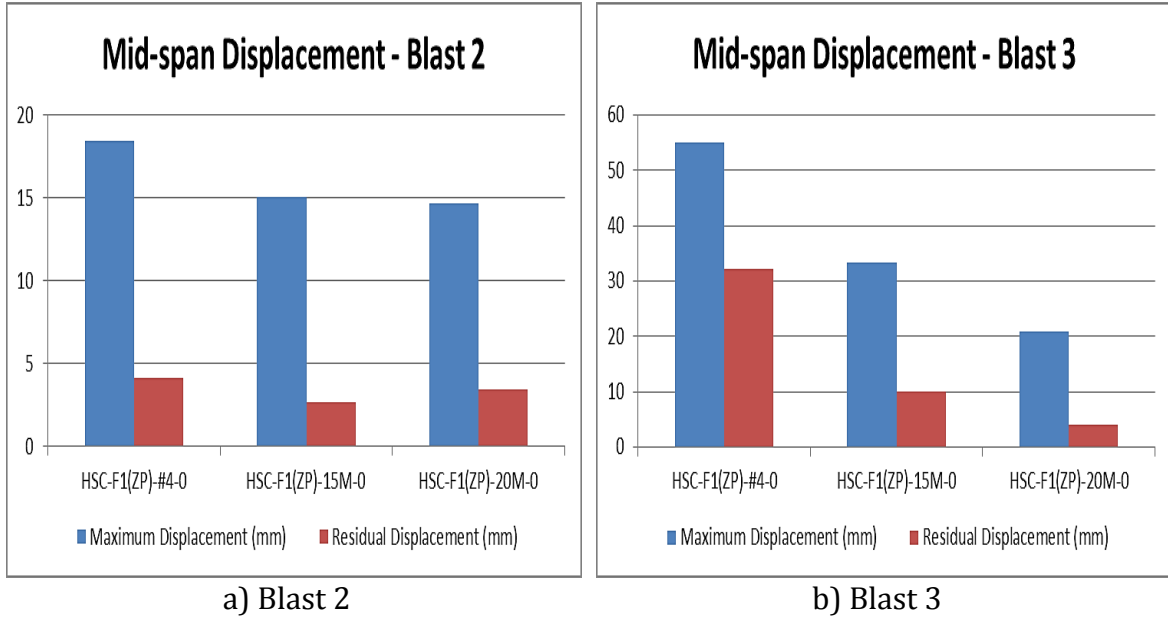
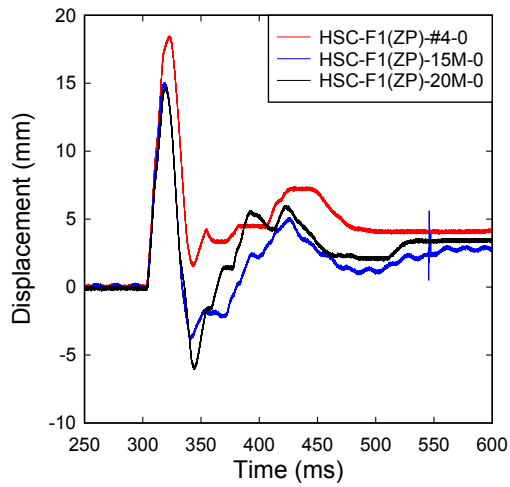


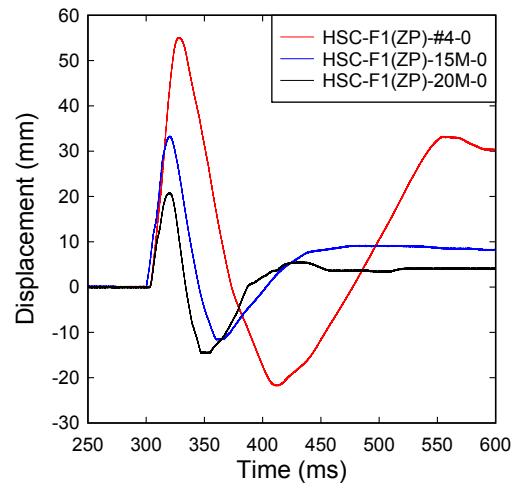
Figure 7-12 Maximum and residual displacements; effects of the reinforcement ratio with steel fibres

Displacement Time History - Blast 2
Effect of reinforcement ratio with steel fibres



a) Blast 2

Displacement Time History - Blast 3
Effect of reinforcement ratio with steel fibres



b) Blast 3

Figure 7-13 Displacement time histories; effects of the reinforcement ratio with steel fibres

7.4 The Effects of the Concrete Strength and Stirrups in HSC Beams

7.4.1 Effects of Concrete Strength

Specimens, NSC-F0-#4-S and HSC-F0-#4-S, were both designed with #4 longitudinal bars and shear reinforcement but were constructed with normal-strength and high-strength concrete, respectively. Photographs showing the failure of the specimens are included in Figure 7-14. Comparative displacement bar charts and time histories for the beams at blasts 2 and 2A are provided in Figure 7-15 and Figure 7-16.

The results indicate that the HSC-F0-#4-S specimen had slightly improved performance in terms of control of mid-span displacements when compared to the companion specimen with normal-strength concrete. The displacements for the HSC-F0-#4-S ($\delta_{\max} = 30.44$ mm and $\delta_{\text{res}} = 14.16$ mm) and NSC-F0-#4-S specimen ($\delta_{\max} = 29.53$ mm and $\delta_{\text{res}} = 13.04$ mm) were very similar after Blast 2. However, at blast 2A, the maximum and residual mid-span displacements dropped by 7% and 74% respectively when comparing the HSC-F0-#4-S specimen ($\delta_{\max} = 44.53$ mm and $\delta_{\text{res}} = 6.59$ mm) to the NSC-F0-#4-S specimen ($\delta_{\max} = 47.96$ mm and $\delta_{\text{res}} = 25.11$ mm). Both specimens failed at this blast.

The failure type was similar in both specimens, with yielding of flexural reinforcement and concrete spalling in the tension zone, as seen in Figure 7-14.

In summary the results show some limited benefits associated with the use of high-strength concrete in the beams, where the use of HSC was capable of reducing mid-span displacements (especially the residual displacement) at the last shot (blast 2A). It should be noted that the "normal-strength" SCC used in this study had a somewhat high compressive strength of 58 MPa; further research on beams constructed with lower strength concrete (i.e. in the range of 25-40 MPa) is recommended.



a) NSC-F0-#4-S



b) HSC-F0-#4-S

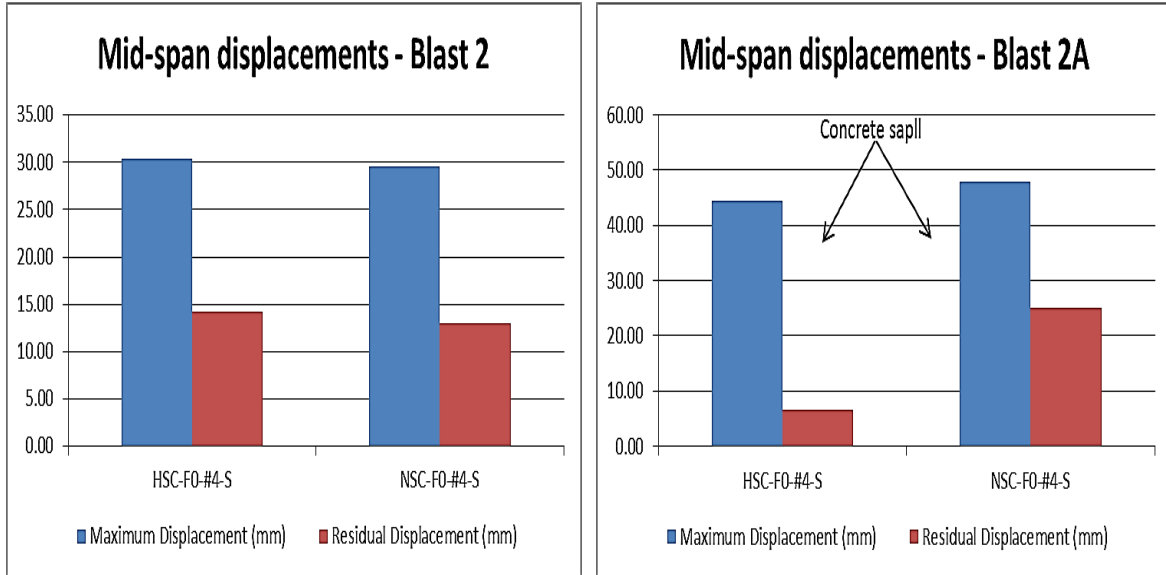


c) Mid-span spall at Blast 2A,
NSC-F0-#4-S



d) Mid-span spall at Blast 2A,
HSC-F0-#4-S

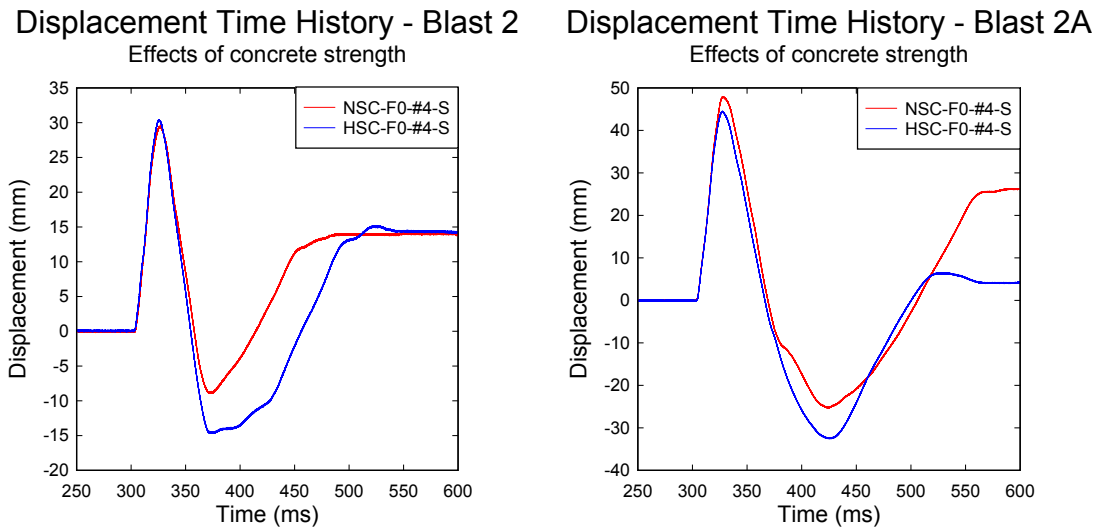
Figure 7-14 Photographs; effects of concrete strength



a) Blast 2

b) Blast 2A

Figure 7-15 Maximum and residual displacements; effects of concrete strength



a) Blast 2

b) Blast 2A

Figure 7-16 Displacement time histories; effects of concrete strength

7.4.2 Effect of Stirrups in HSC Beams without Steel Fibres

The high-strength reinforced concrete specimens, HSC-F0-20M-S and HSC-F0-20M-0, were designed with and without transverse reinforcement, respectively. Photographs showing the failure type are included in Figure 7-17. Comparative displacement bar charts and time histories for blasts 1 and 2 are provided in Figure 7-18 and Figure 7-19.

The presence of stirrups had an important effect on beam performance in terms of increasing blast resistance and preventing shear failure. When comparing the HSC-F0-20M-S specimen ($\delta_{\max} = 10.44$ mm and $\delta_{\text{res}} = 1.98$ mm) with the HSC-F0-20M-0 specimen ($\delta_{\max} = 12.55$ mm and $\delta_{\text{res}} = 3.37$ mm) at blast 1, the HSC-F0-20M-S specimen showed a reduction by 17% for maximum displacement and by 41% for residual displacement. At blast 2, the HSC-F0-20M-S specimen ($\delta_{\max} = 15.12$ mm and $\delta_{\text{res}} = 0.15$ mm) dropped the maximum mid-span displacement by 22% and the residual mid-span displacement by 98 % as compared to the HSC-F0-20M-0 specimen ($\delta_{\max} = 19.31$ mm and $\delta_{\text{res}} = 9.21$ mm) which experienced a shear failure at this shot. The use of transverse reinforcement prevented shear failure in beam HSC-F0-20M-S while simultaneously providing better control of the displacements at mid-span. The failure mode for the HSC-F0-20M-S specimen was a steel-controlled flexure failure, with yielding of the steel reinforcement and concrete crushing at mid-span as seen in Figure 7-17. The use of shear reinforcement in the beam allowed it to withstand additional blast loading compared to the other companion beam with no shear reinforcement, with failure recorded at Blast 4.

In summary, the results demonstrate the importance of providing minimum transverse reinforcement in high-strength concrete beams in order to prevent brittle shear failure under blast loading.



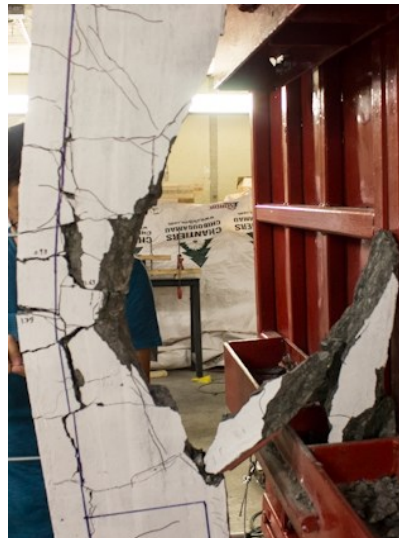
a) HSC-F0-20M-0



b) HSC-F0-20M-S



c) Shear-failure at Blast 2,
HSC-F0-20M-0



d) Mid-span crush at Blast 4,
HSC-F0-20M-S

Figure 7-17 Photographs; effects of stirrups for beams without steel fibres

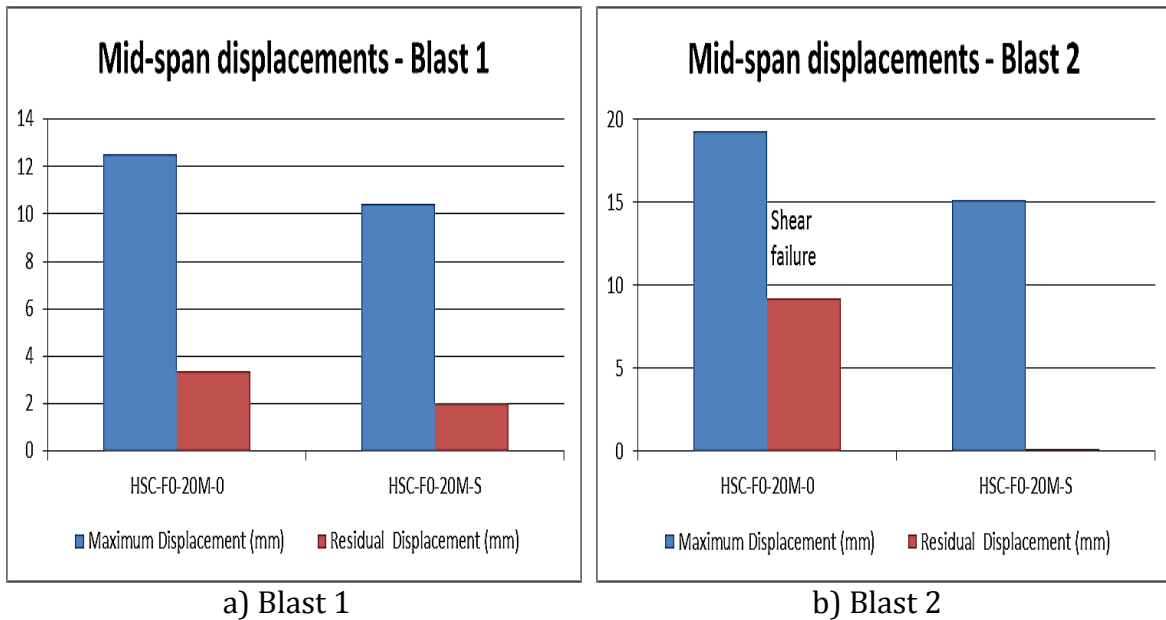


Figure 7-18 Maximum and residual displacements; effects of stirrups for beams without steel fibres

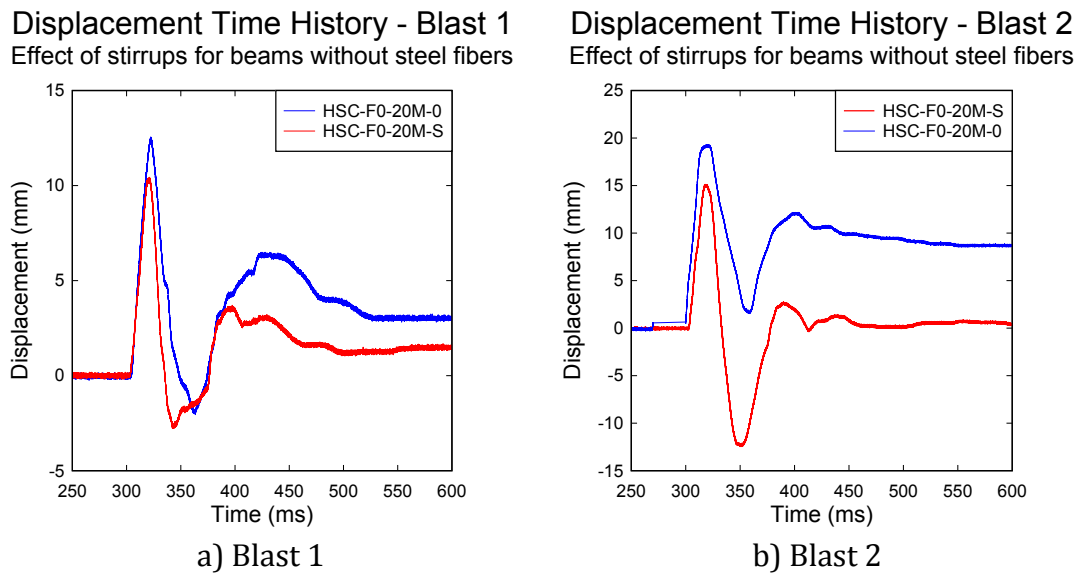


Figure 7-19 Displacement time histories; effects of stirrups for beams without steel fibres

7.5 Effects of the Steel Fibres

The results of this study show that the inclusion of steel fibres enhances the blast behaviour of reinforced concrete beams. The use of fibres resulted in reduced displacements at equivalent blast loads, while also improving damage control and reducing secondary fragmentation. Specimens containing steel fibres also saw a boost in ductility with increased blast resistance. The following sub-sections provide observations with regards to the effects of steel fibres when compared to HSC beams with and without transverse reinforcement.

7.5.1 Effects of the Steel Fibres: HSFRC vs. HSC Beams with Stirrups

Specimens, HSC-F0-#4-S, HSC-F0-15M-S, and HSC-F0-20M-S, were all designed with high-strength concrete and reinforced with 6.3 mm stirrups arranged at spacing of 100 mm. Specimens HSC-F1(ZP)-#4-0, HSC-F1(ZP)-15M-0, and HSC-F1(ZP)-20M-0 had the same amount of longitudinal steel as their companions, but contained a 1.0% volumetric ratio of steel fibres (ZP), and did not have any shear reinforcement. Photographs showing the failure of the beams are included in Figure 7-20. Comparative displacement bar charts and time histories for blasts 2 and 3 are provided in Figure 7-21 to Figure 7-23.

For specimens with #4 rebar, the HSC-F1(ZP)-#4-0 specimen showed improved behaviour when compared to the companion HSC specimen containing stirrups and no fibres. When comparing to the HSC-F1(ZP)-#4-0 specimen at blast 2, the HSC-F1-#4-0 specimen showed a reduction in maximum displacement by 39% ($\delta_{\max} = 18.47$ mm vs. 30.44 mm) and in residual displacement by 71% ($\delta_{\text{res}} = 4.10$ mm vs. 14.16 mm). At blast 3, the HSC-F0-#4-S specimen had already failed after application of blast 2A (40 psi) pressures, while the HSC-F1(ZP)-#4-0 specimen showed maximum and residual displacements of $\delta_{\max} = 61.0$ mm and $\delta_{\text{res}} = 38.89$ mm, and survived up to shot 3A (60 psi).

For specimens with 15M bars, the HSC-F1(ZP)-15M-0 specimen which had 1% fibres also performed better when compared to the companion HSC-F0-15M-S specimen which had no fibres. At blast 2, the HSC-F1(ZP)-15M-0 specimen ($\delta_{\max} = 15.02$ mm and $\delta_{\text{res}} = 2.66$ mm) showed reductions of 30% and 44% when comparing maximum and residual displacements to the HSC-F0-15M-S specimen ($\delta_{\max} = 21.4$ mm and $\delta_{\text{res}} = 4.71$ mm). Similarly, at blast 3, the maximum and residual mid-span displacements of the HSC-F1(ZP)-15M-0 specimen ($\delta_{\max} = 33.31$ mm and $\delta_{\text{res}} = 10.08$ mm) were appreciably lower by 73% and 54% respectively when compared to the displacements of the HSC-F0-15M-S specimen ($\delta_{\max} = 124.0$ mm and $\delta_{\text{res}} = 22.04$ mm), which failed at this level of blast loads.

For specimens with 20M rebar, the HSC-F1(ZP)-20M-0 specimen showed an improvement in reducing maximum and residual displacements over the HSC-F0-20M-S specimen. At blast 2, the maximum and residual mid-span displacements were slightly reduced for the specimen with stirrups. At blast 3, the reduction reached 37% for the maximum displacement and 67% for the residual displacement, when comparing the HSC-F1(ZP)-20M-0 specimen ($\delta_{\max} = 20.82$ mm and $\delta_{\text{res}} = 4.07$ mm) with the HSC-F0-20M-S specimen ($\delta_{\max} = 32.91$ mm and $\delta_{\text{res}} = 12.43$ mm).

The addition of a 1.0% volumetric ratio of steel fibres also allowed the HSFRC beams to resist higher blast loading when compared to companion HSC specimens without fibres. In addition, the provision of steel fibres delayed and controlled concrete crushing in the compression region and prevented concrete spalling. However, failure at extreme blast pressures was associated with concrete splitting at major crack locations at mid-span, with pull-out of steel fibres, as shown in Figure 7-20.

In summary, the results demonstrate clear benefits associated with the use of steel fibres in HSC beams subjected to blast loading. The addition of steel fibres effectively replaced transverse reinforcement (preventing shear failure), and improved flexural performance by reduced displacements at equivalent blast loads. The provision of fibres also allowed the beams to sustain greater blast loads, while also enhancing damage tolerance.

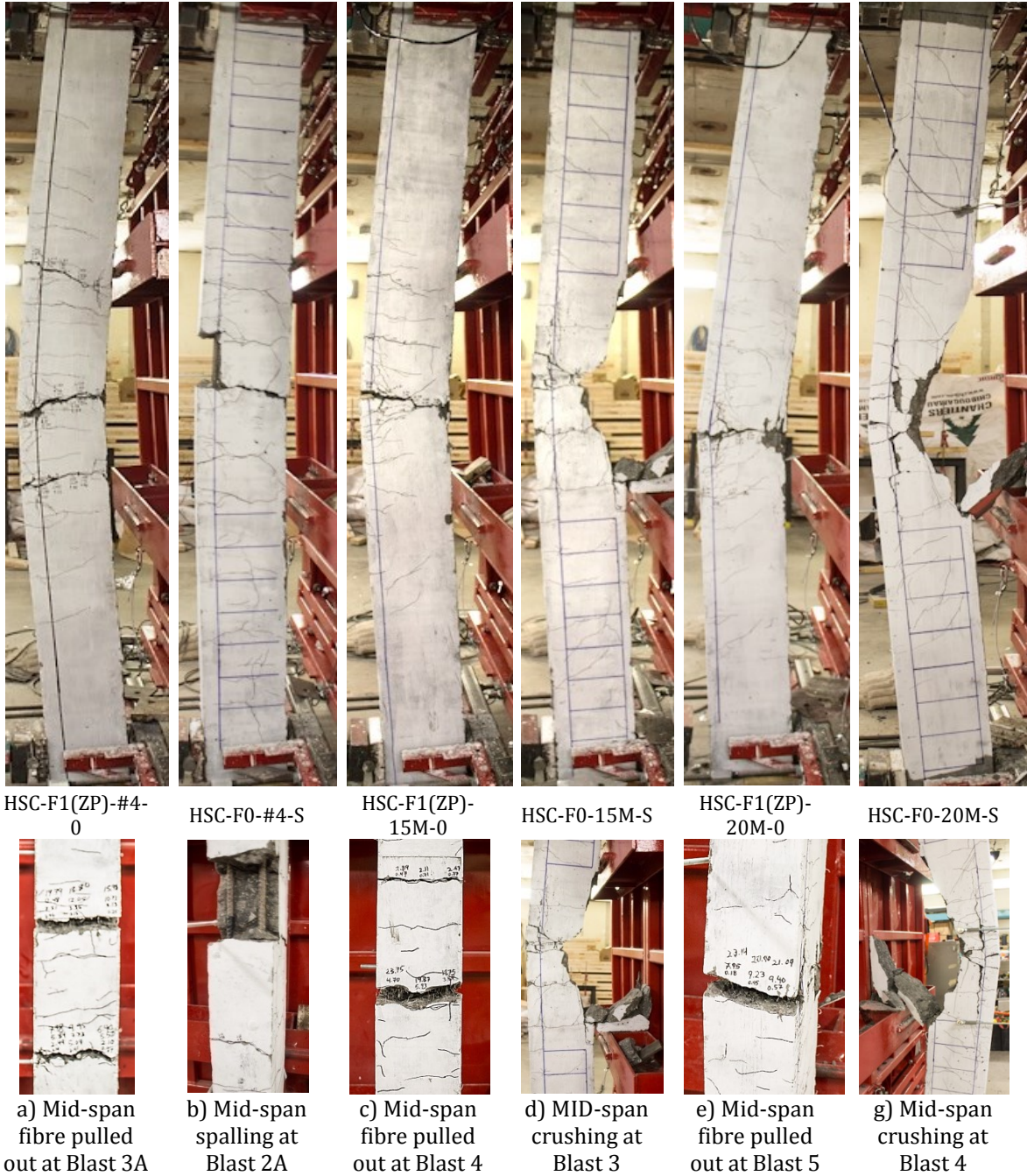
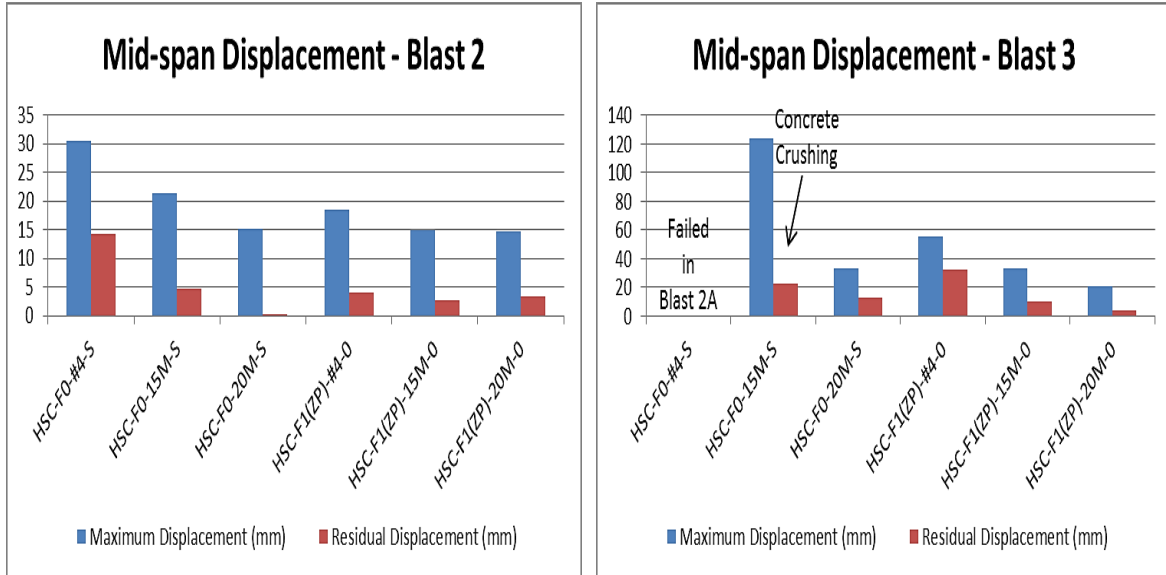


Figure 7-20 Photographs; effects of steel fibres, HSFRC vs. HSC beams with stirrups

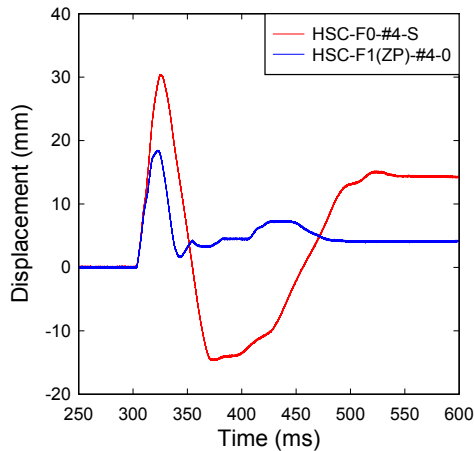


a) Blast 2

b) Blast 3

Figure 7-21 Maximum and residual displacements; effects of steel fibres, HSFRC vs. HSC beams with stirrups

Displacement Time History - Blast 2
Effect of steel fibres for beams with stirrups



Displacement Time History - Blast 3
Effect of steel fibres for beams with stirrups

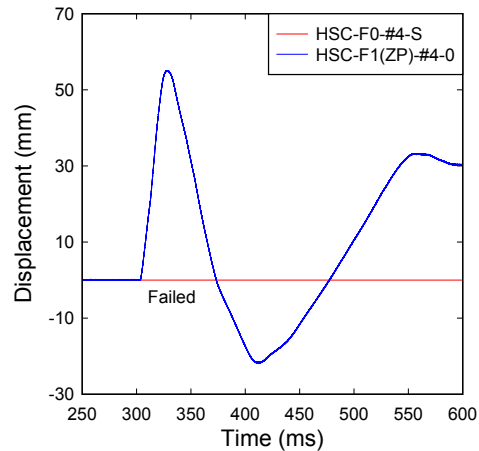
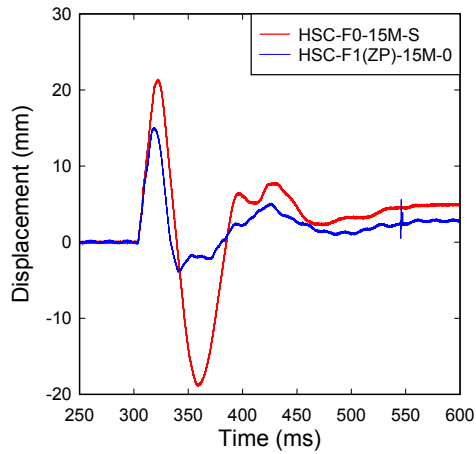


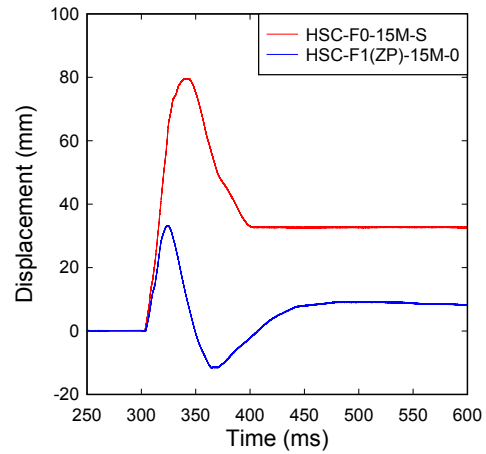
Figure 7-22 Displacement time histories; effects of steel fibres for series #4, HSFRC vs. HSC beams with stirrups

Displacement Time History - Blast 2
Effect of steel fibers for beams with stirrups



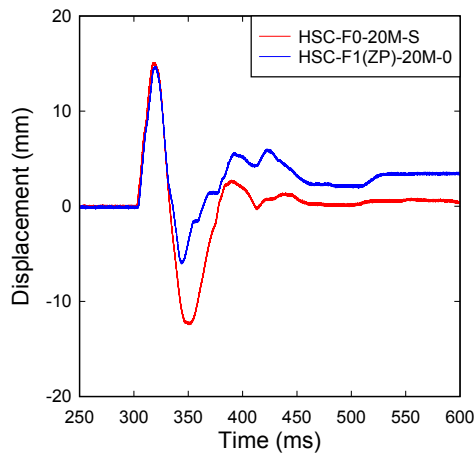
a) Blast 2

Displacement Time History - Blast 3
Effect of steel fibers for beams with stirrups



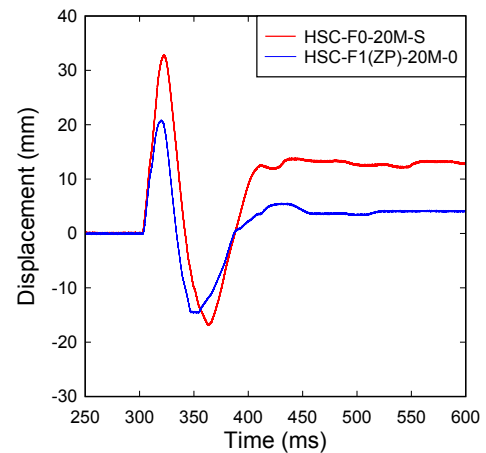
b) Blast 3

Displacement Time History - Blast 2
Effect of steel fibers for beams with stirrups



a) Blast 2

Displacement Time History - Blast 3
Effect of steel fibers for beams with stirrups



b) Blast 3

Figure 7-23 Displacement time histories; effects of steel fibres for series 15M and 20M, HSFRC vs. HSC beams with stirrups

7.5.2 Effects of the Steel Fibres: HSFRC vs. HSC Beams without stirrups

The high-strength concrete specimens, HSC-F1(ZP)-20M-0 and HSC-F0-20M-0, were designed without stirrups and were cast with and without steel fibres. Photographs showing the beams at failure are included in Figure 7-24. Comparative displacement bar charts and time histories for blasts 1 and 2 are provided in Figure 7-25 and Figure 7-26.

When compared to the HSC-F0-20M-0 specimen ($\delta_{\max} = 12.55$ mm and $\delta_{\text{res}} = 3.37$ mm) at blast 1, the HSC-F1(ZP)-20M-0 specimen ($\delta_{\max} = 8.44$ mm and $\delta_{\text{res}} = 1.4$ mm) showed a reduction by 33% for maximum displacement and 58% for residual displacement. At blast 2, the HSC beam failed suddenly in shear due to the lack of transverse reinforcement, while the addition of steel fibres prevented shear failure.

Overall, steel fibre reinforcement significantly improved the performance of the beam against impulsive shear stresses, allowing the HSFRC specimen to fail in a ductile flexural mode, with an ability to resist stronger blast loads. In terms of failure, the HSC-F1(ZP)-20M-0 specimen failed in flexure due to pull out of the steel fibres after application of Blast 5 loads, whereas the HSC-F0-20M-0 specimen failed due to shear after the application of Blast 2 pressures, as shown in Figure 7-24.



a) HSC-F1(ZP)-20M-0



b) HSC-F0-20M-0

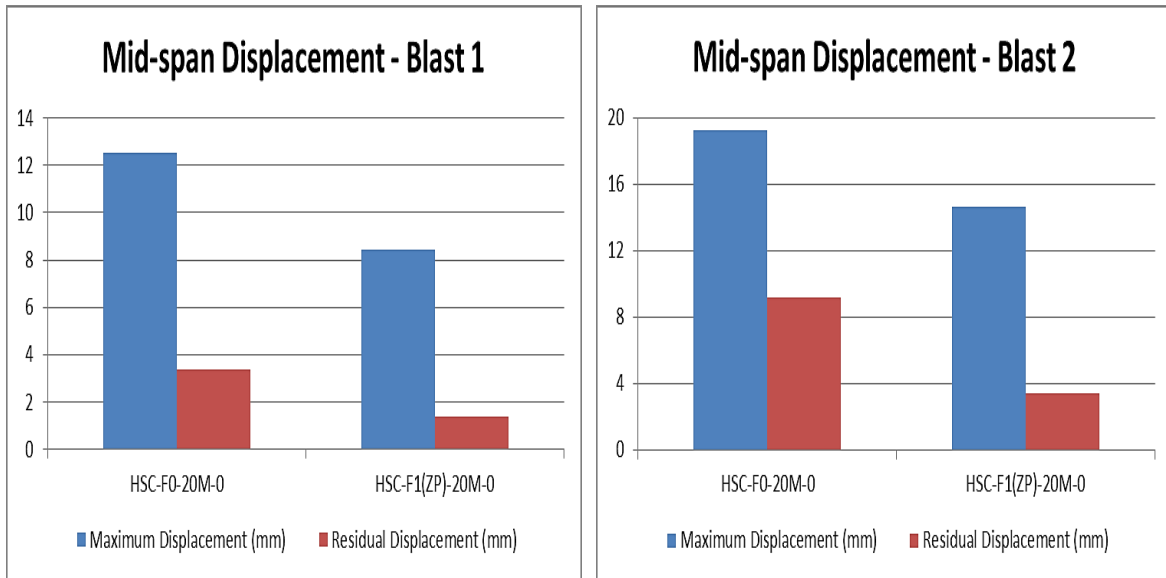


c) Mid-span fibre pulled out at Blast 5, HSC-F1(ZP)-20M-0



d) Shear-failure (top) at Blast 2, HSC-F0-20M-0

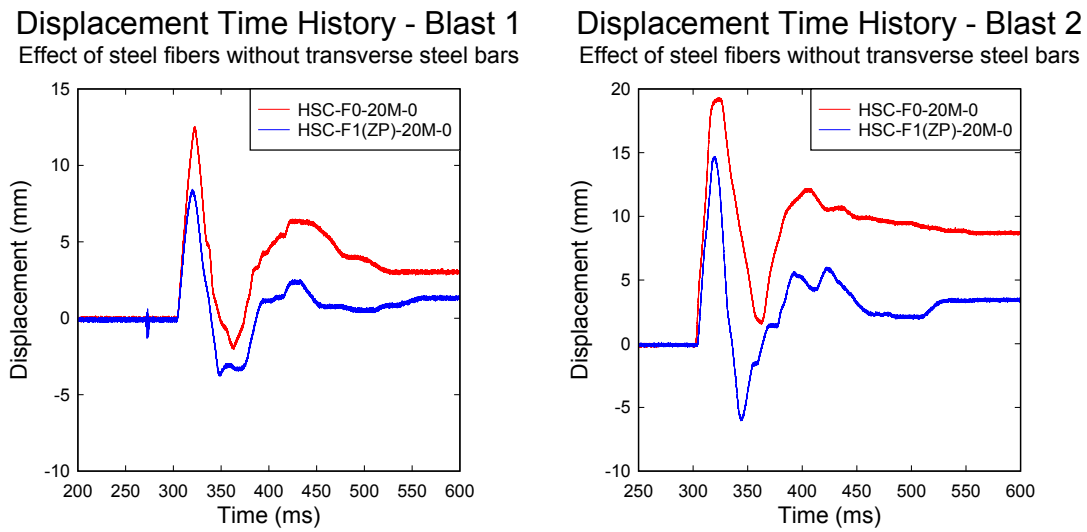
Figure 7-24 Photographs; effects of steel fibres, HSFRC vs. HSC beams without stirrups



a) Blast 1

b) Blast 2

Figure 7-25 Maximum and residual displacements; effects of steel fibres, HSFRC vs. HSC beams without stirrups



a) Blast 1

b) Blast 2

Figure 7-26 Displacement time histories; effects of steel fibres, HSFRC vs. HSC beams without stirrups

7.6 Effects of Fibre Properties

Based on the experimental program, reinforced concrete beams with high-strength concrete benefited from the use of steel fibres. The following sub-sections further investigate the effect of fibre content and fibre type on beam response.

7.6.1 Effects of Fibre Content

The high strength specimens, HSC-F0-20M-S, HSC-F0.5(ZP)-20M-S, and HSC-F1(ZP)-20M-S, were designed with a 0.0%, 0.5%, and 1.0% fraction volume of steel fibres (ZP) and transverse reinforcement at spacing of 100 mm. Also included in this comparison is specimen HSC-F1(ZP)-20M-0 which was constructed with 1.0% fibres but without shear reinforcement. Photographs showing the condition of the specimens at failure are included in Figure 7-27. Comparative displacement bar charts and time histories for blasts 3 and 4 are provided in Figure 7-28 and Figure 7-29.

The addition of 0.5% steel fibres in HSC-F0.5(ZP)-20M-S allowed this specimen to resist greater blast loads and reduce mid-span displacements when compared to the companion beam without fibres. For example, at Blast 3, the HSC-F0.5(ZP)-20M-S showed a reductions of 27% in maximum displacement ($\delta_{\max} = 24.09$ mm vs. 32.91 mm) and 43% in residual displacement ($\delta_{\text{res}} = 7.13$ mm vs. 12.43 mm) when compared to the HSC-F0-20M-S specimen. At blast 4, the maximum and residual mid-span displacements for the HSC-F0.5(ZP)-20M-S specimen ($\delta_{\max} = 48.55$ mm and $\delta_{\text{res}} = 29.59$ mm) reduced the maximum and residual displacements by 58.8% and 58.7% respectively, when compared to the HSC-F0-20M-S specimen ($\delta_{\max} = 118.06$ mm and $\delta_{\text{res}} = 71.69$ mm).

Although the specimen with 1.0% fibres and stirrups (HSC-F1(ZP)-20M-S) showed improved response when compared to the specimen without fibres (HSC-F0-20M-S), the trend is less clear when comparing to specimen HSC-F0.5(ZP)-20M-S which had 0.5% fibres. At blast 3, the use of 1% vs. 0.5% fibres resulted in a 13% decrease in maximum displacement ($\delta_{\max} = 24.09$ mm vs. $\delta_{\max} = 27.64$ mm), however the increase in fibre content increased residual displacement ($\delta_{\text{res}} = 7.13$ mm vs. $\delta_{\text{res}} = 4.04$ mm) by 43.34%. At blast 4, the HSC-F0.5(ZP)-20M-S specimen ($\delta_{\max} = 48.55$ mm and $\delta_{\text{res}} = 29.59$ mm) showed reductions in maximum and residual displacements by 28% and 29% respectively when compared to the specimen with 1% fibres and stirrups ($\delta_{\max} = 67.03$ mm and $\delta_{\text{res}} = 41.84$ mm).

The benefit of increasing fibre content is more clear when comparing specimens HSC-F1(ZP)-20M-0 and HSC-F0.5(ZP)-20M-S, where the use of 1.0% fibres in the beam without stirrups results in a reduction of maximum and residual

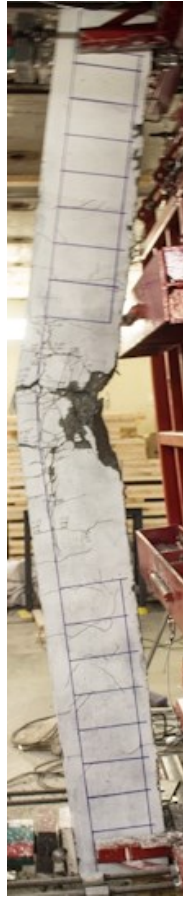
displacements at equivalent blasts. At blast 3, the maximum and residual mid-span displacements for the HSC-F1(ZP)-20M-0 specimen ($\delta_{\max} = 20.82$ mm and $\delta_{\text{res}} = 4.07$ mm) were lower by 14% and 43% respectively when compared to the HSC-F0.5(ZP)-20M-S specimen ($\delta_{\max} = 24.09$ mm and $\delta_{\text{res}} = 7.13$ mm) which had 0.5% fibres. At blast 4, the HSC-F1(ZP)-20M-0 specimen resulted reduced maximum displacement ($\delta_{\max} = 42.06$ mm) by 13% & with a 35% decrease for the residual displacement ($\delta_{\text{res}} = 19.27$ mm) in comparison to the HSC-F0.5(ZP)-20M-S specimen ($\delta_{\max} = 48.55$ mm and $\delta_{\text{res}} = 29.59$ mm). At blast 5, the HSC-F1(ZP)-20M-0 specimen resulted reduced maximum displacement by 48% ($\delta_{\max} = 62.4$ mm vs. $\delta_{\max} = 119.52$ mm) and residual displacement by 52% ($\delta_{\text{res}} = 37.36$ mm vs. $\delta_{\text{res}} = 77.69$ mm) when compared to the HSC-F0.5(ZP)-20M-S specimen.

In terms of blast resistance, the specimens with fibres failed after application of Blast 5, while the control specimen without fibres failed at Blast 4. The HSC-F0-20M-S specimen showed extensive damage at failure compared to the two other specimens with fibres, whereas the failure for the HSC-F0.5(ZP)-20M-S and HSC-F1(ZP)-20M-S specimens was almost the same with more controlled concrete crushing at mid-span, as seen in Figure 7-27. The HSC-F1(ZP)-20M-0 specimen showed improved damage tolerance when compared to all other beams in this set.

In summary, the limited tests which were conducted show that increasing fibre content from 0.5% to 1% in high-strength concrete beams with stirrups did not result in further enhancement in blast performance. The benefit of increasing fibre content to 1% was more obvious in the beam without stirrups (HSC-F1(ZP)-20M-0) pointing to the possibility of reduced fibre efficiency which may have resulted from loss of mix uniformity when 1% fibres were added to the more heavily congested beam with stirrups. Further research examining the effect of fibre content on beam blast response is recommended.



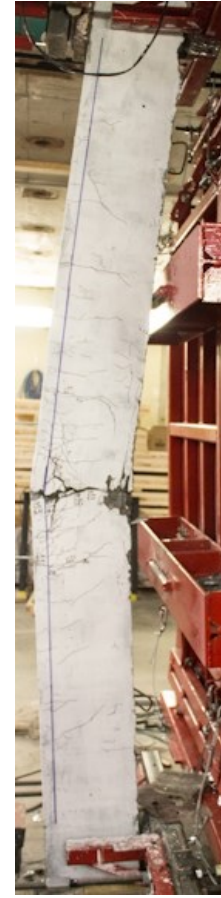
a) HSC-F0-20M-S



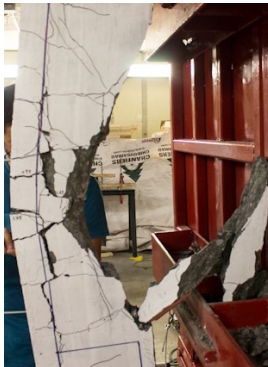
b) HSC-F0.5(ZP)-20M-S



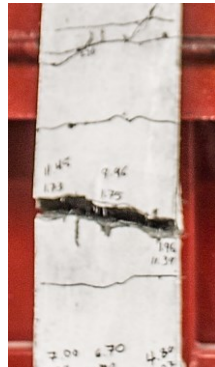
c) HSC-F1(ZP)-20M-S



d) HSC-F1(ZP)-20M-0



d) Mid-span crush Blast 4, HSC-F0- 20M-S



e) Mid-span fibre pulled out at Blast 5, HSC-F0.5(ZP)-20M-S

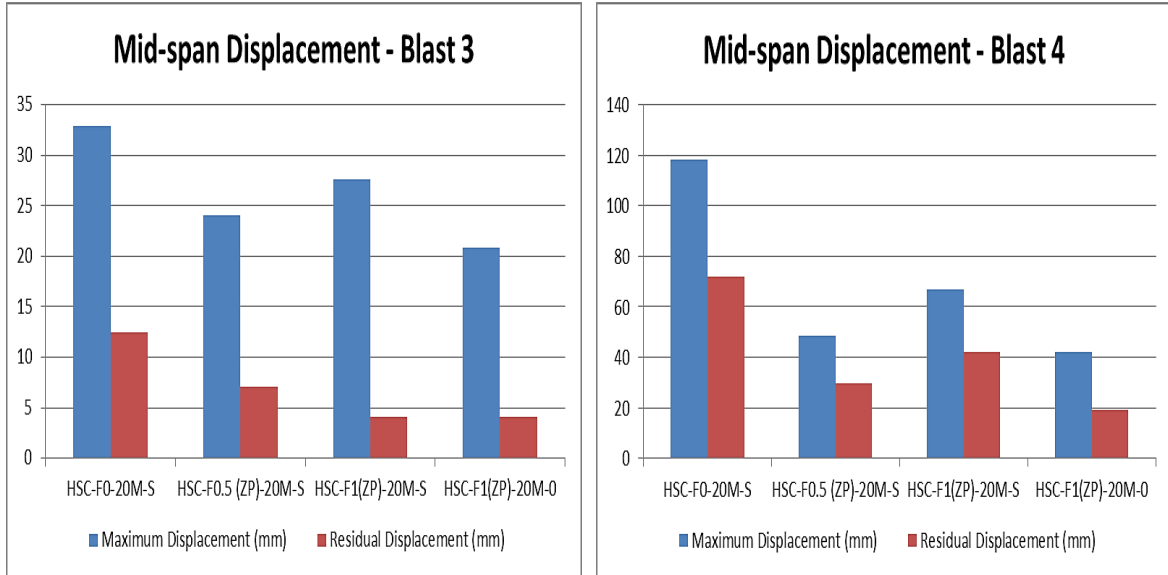


f) Mid-span crush & fibre pulled out at Blast 5, HSC-F1(ZP)-20M-S



g) Mid-span fibre pulled out at Blast 5, HSC-F1(ZP)-20M-0

Figure 7-27 Photographs; effects of fibre content ratio

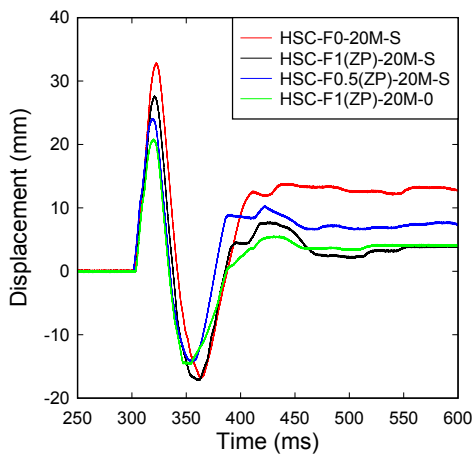


a) Blast 3

b) Blast 4

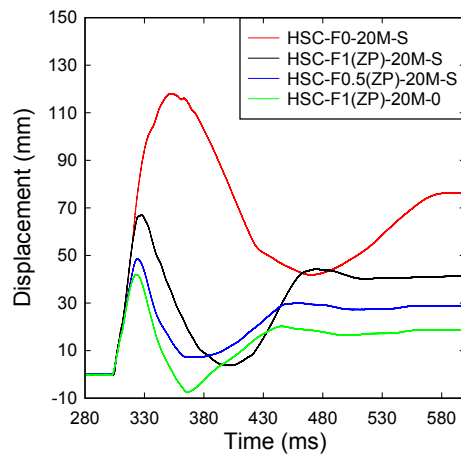
Figure 7-28 Maximum and residual displacements; effects of fibre content ratio

Displacement Time History - Blast 3
Effect of fibre content ratio



a) Blast 3

Displacement Time History - Blast 4
Effect of fibre content ratio



b) Blast 4

Figure 7-29 Displacement time histories; effects of fibre content ratio

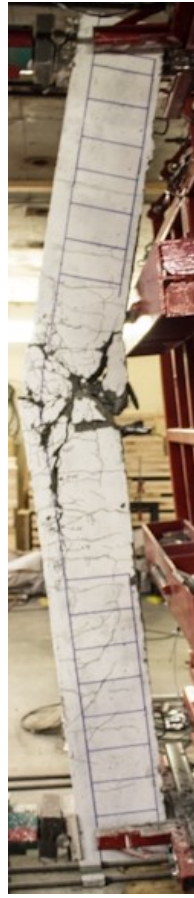
7.6.2 Effects of Fibre Type

The high strength specimens, HSC-F0.5(ZP)-20M-S and HSC-F0.5(5D)-20M-S, were both designed with stirrups and a volumetric ratio of 0.5% of steel fibres. However the specimens were reinforced with two different fibre types (ZP= short fibre or 5D = long fibre). When compared to the ZP fibre, the 5D fibre has increased length, aspect-ratio and tensile resistance with improved anchorage details (double vs. single hook). Photographs showing the failure type are included in Figure 7-30. Comparative displacement bar charts and time histories for blasts 3 and 4 are provided in Figure 7-31 and Figure 7-32.

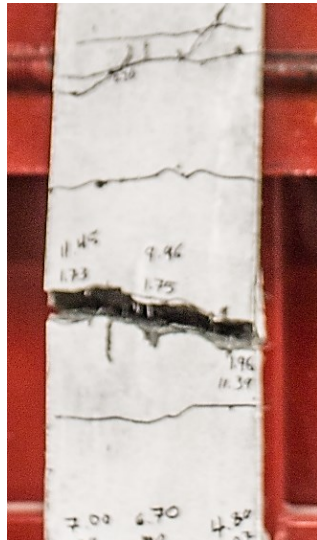
The results indicate that the specimen reinforced with the shorter ZP fibres showed improved blast performance when compared to the specimen with the longer 5D fibres. In terms of displacement control, the HSC-F0.5(ZP)-20M-S specimen ($\delta_{\max} = 24.09$ mm and $\delta_{\text{res}} = 7.13$ mm) showed similar maximum displacement but reduced residual displacement when compared to the HSC-F0.5(5D)-20M-S specimen ($\delta_{\max} = 25.37$ mm and $\delta_{\text{res}} = 4.74$ mm) at blast 3, the HSC-F0.5(ZP)-20M-S specimen showed a reduction by 5% for maximum displacement but an increase by 34% for residual displacement. At blast 4, the maximum and residual mid-span displacements were decreased by 23% and 18% respectively when comparing the HSC-F0.5(ZP)-20M-S specimen ($\delta_{\max} = 48.55$ mm and $\delta_{\text{res}} = 29.59$ mm) to the HSC-F0.5(5D)-20M-S specimen ($\delta_{\max} = 62.66$ mm and $\delta_{\text{res}} = 35.90$ mm).

In terms of failure mode, the HSC-F0.5(5D)-20M-S specimen showed more damage at failure, with multiple diagonal cracks in the mid-span, in comparison to the HSC-F0.5(ZP)-20M-S specimen which had the shorter ZP fibres, as seen in Figure 7-30. However, both specimens failed at Blast 5 with controlled concrete crushing and pull out of steel fibres.

In summary, the results demonstrate improved performance for HSFRC beams containing short steel fibres (ZP) with reduced mid-span displacements in all five shots. While the long steel fibre (5D) resulted in increased blast resistance and lower mid-span displacements when compared to the control beam without fibres, performance was not enhanced when compared to the companion beam with ZP fibres.



a) HSC-F0.5(ZP)-20M-S b) HSC-F0.5(5D)-20M-S



c) Mid-span fibre pulled out at Blast 5, HSC-F0.5(ZP)-20M-S



d) Mid-span crushing & fibre pulled out at Blast 5, HSC-F0.5(5D)-20M-S

Figure 7-30 Photographs; effects of fibre type

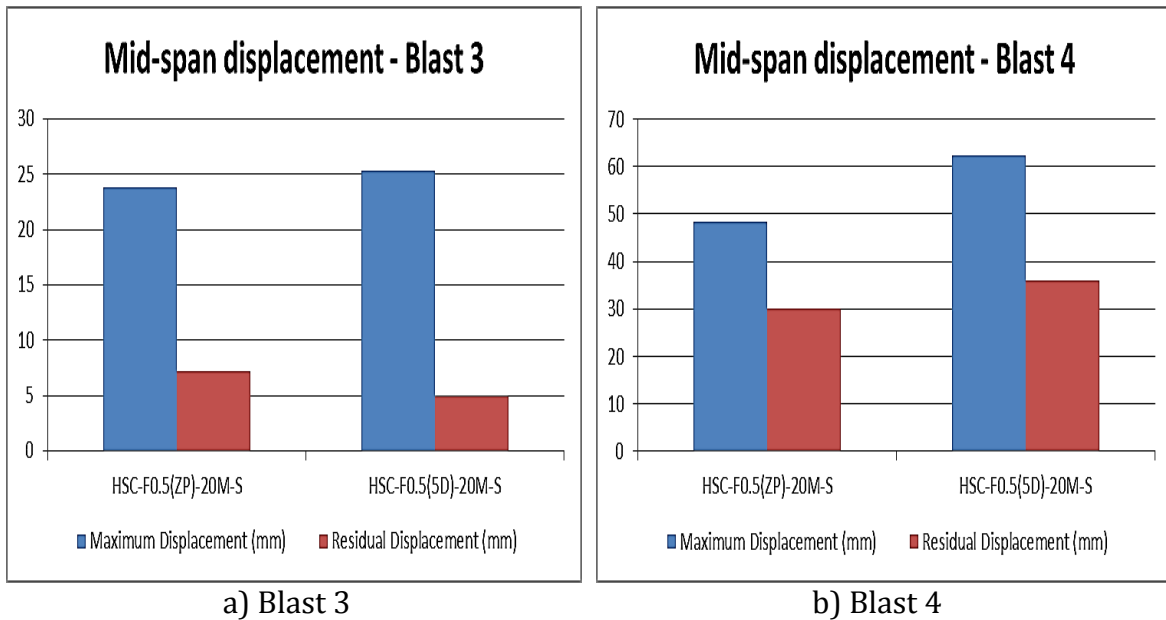


Figure 7-31 Maximum and residual displacements; effects of fibre type

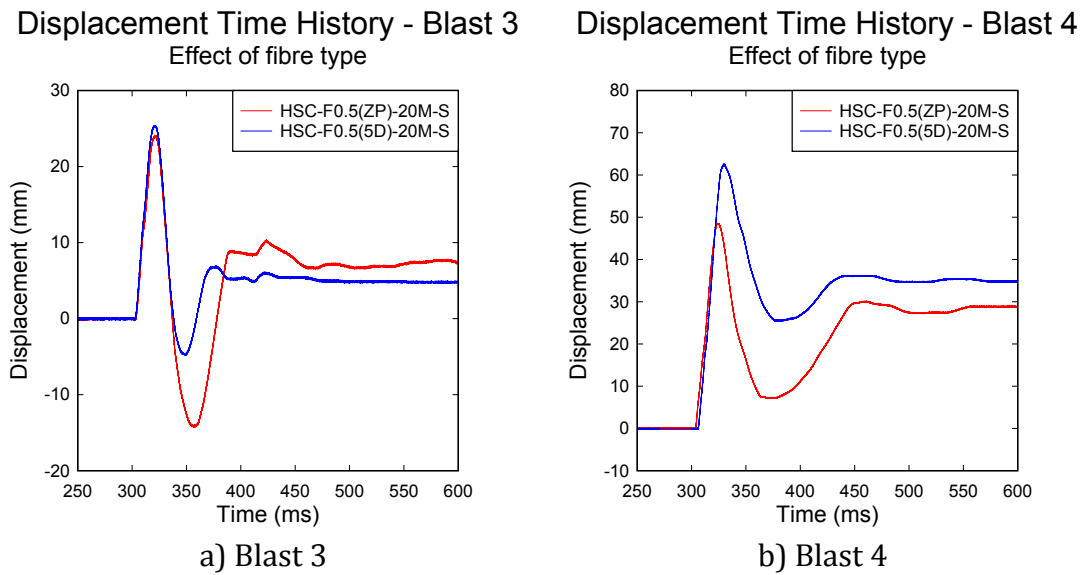


Figure 7-32 Displacement time histories; effects of fibre type

7.7 The Effects of Combined use of Stirrups and Fibres

Specimens, HSC-F1(ZP)-20M-0 and HSC-F1(ZP)-20M-S were designed with high-strength concrete and a volumetric ratio of 1.0% of steel fibres. However, the HSC-F1(ZP)-20M-S specimen also had shear reinforcement with stirrups spaced at 100 mm in the shear spans. Photographs showing the specimens at failure are included in Figure 7-33. Comparative displacement bar charts and time histories for blasts 3 and 4 are provided in Figure 7-34 and Figure 7-35.

The results indicate that the mid-span displacements were better controlled by the HSC-F1(ZP)-20M-0 specimen which had only fibres, in comparison to the HSC-F1(ZP)-20M-S specimen, which had fibres and stirrups. For example, at Blast 3, the HSC-F1(ZP)-20M-0 specimen ($\delta_{\max} = 20.82$ mm and $\delta_{\text{res}} = 4.07$ mm) showed a reduction by 25% for maximum displacement with no effect on residual displacement when compared to the HSC-F1-20M-S specimen ($\delta_{\max} = 27.64$ mm and $\delta_{\text{res}} = 4.04$ mm). At blast 4, the maximum and residual mid-span displacements in the HSC-F1(ZP)-20M-0 specimen were decreased by factors of 37% ($\delta_{\max} = 42.06$ mm vs. $\delta_{\max} = 67.03$ mm) and 54% ($\delta_{\text{res}} = 19.27$ mm vs. $\delta_{\text{res}} = 41.84$ mm), when compared to the HSC-F1(ZP)-20M-S specimen.

In addition, the HSC-F1(ZP)-20M-0 specimen showed better control of cracking and showed fewer damages when compared to the HSC-F1(ZP)-20M-S, as seen in Figure 7-33. It can be seen that the damage in the compression zone was increased for the specimen with fibres and stirrups, with more significant concrete crushing, while the HSC-F1(ZP)-20M-0 specimen more effectively controlled crushing at failure. In both cases failure was associated with concrete splitting and fibre pull-out in the tension zone.

The observations from the limited tests do not show clear benefits associated with the combined use of steel fibres and stirrups in high-strength concrete beams. It is noted that use of 1.0% fibres in beams with stirrups required more vibration of concrete during construction and this may have caused some loss of fibre uniformity and efficiency in the HSC-F1(ZP)-20M-S specimen. It is also noted that 1.0% fibres was sufficient to prevent shear failure in the beams in the current study; it is recommended that further research be conducted on the beams with larger shear demand to examine the effect of combining stirrups and fibres on shear resistance in beams subjected to blast loads.



a) HSC-F1(ZP)-20M-0



b) HSC-F1(ZP)-20M-S

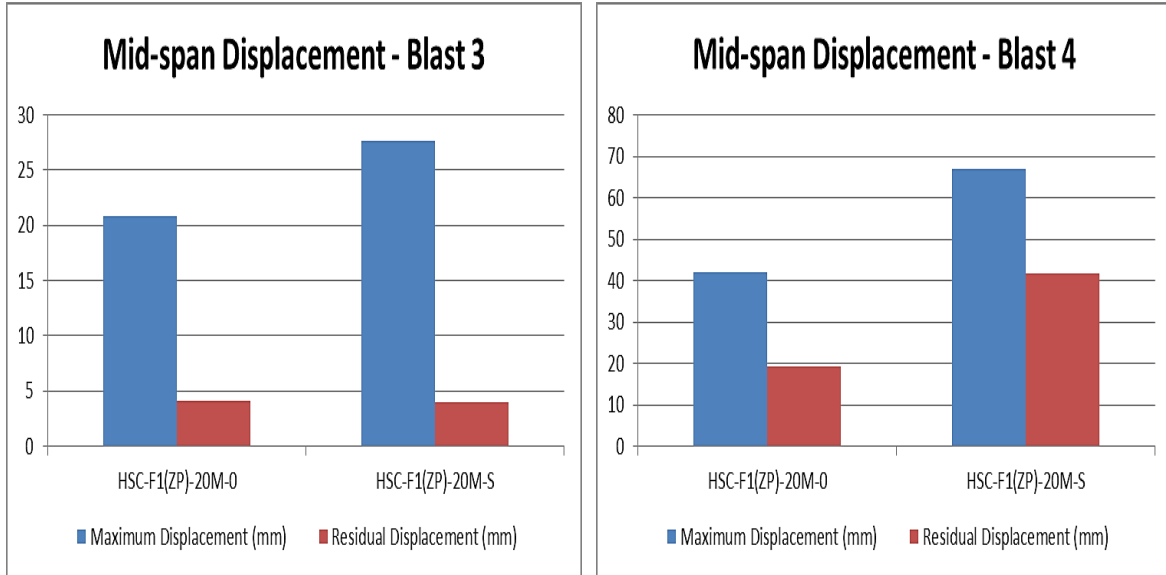


c) Mid-span fibre pulled out at Blast 5, HSC-F1(ZP)-20M-0



d) Mid-span crush & fibre pulled out at Blast 5, HSC-F1(ZP)-20M-S

Figure 7-33 Photographs; effects of combined use of stirrups and fibres

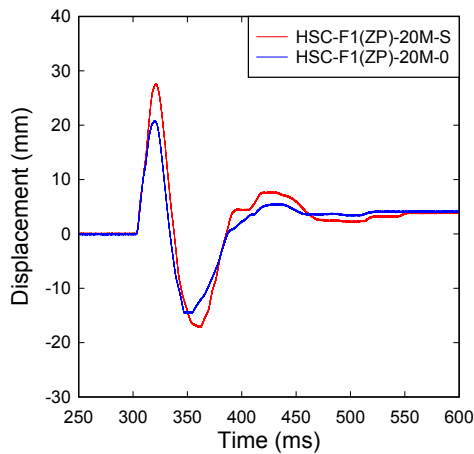


a) Blast 3

b) Blast 4

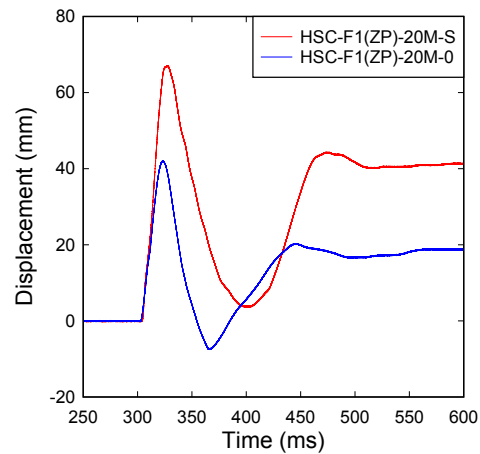
Figure 7-34 Maximum and residual displacements; effects of combined use of stirrups and fibres

Displacement Time History - Blast 3
Effects of stirrups for beams with steel fibers



a) Blast 3

Displacement Time History - Blast 4
Effects of stirrups for beams with steel fibers



b) Blast 4

Figure 7-35 Displacement time histories; effects of combined use of stirrups and fibres

7.8 Damage Tolerance and Fragmentation

In this section, the effect of fibres on cracking, damage tolerance and secondary fragmentation of the high-strength concrete beams is examined. The maximum crack width recorded for each specimen is included as a summary in Table 7-1.

In general the use of fibres resulted in a better control of cracking, prevented cover concrete from spalling, and controlled crushing of concrete. The use of fibres also improved damage tolerance and reduced fragmentation. Photographs in Figure 7-36 show the excessive damage that occurred in the HSC beams at failure. Beam HSC-F0-#4-S showed spalling of tension concrete at failure. Specimens HSC-F0-15M-S and HSC-F0-20M-S showed severe crushing of concrete in the compression zone. The addition of fibres to HSC transformed these behaviours. In the specimen with #4 rebar spalling was eliminated. Similarly the fibres prevented sudden crushing of concrete in the 15M and 20M specimens. It is noted that the beams in the current study sustained large rebound displacements and were tested under repeated blast loads. In general the severe failures observed in the plain HSC specimens indicate the need to provide top steel reinforcement to prevent failure under rebound loading.

The provision of fibres also had a major effect on secondary fragmentation. High-speed video camera footage was used to examine the effect of fibres on flying debris and secondary blast fragments. When compared to the HSC beams, the HSFRC specimens were able to reduce the debris and effectively eliminate secondary fragmentation, even at failure. The HSC specimens had large chunks and fragments of concrete fly away from the beams at failure, as seen in Figure 7-37. The size of the fragments varied from smaller sand-like particles to much larger debris which could fit in one's hand. In contrast the HSFRC beams showed minimal fragmentation at failure (see Figure 7-38).

Increasing the longitudinal reinforcement ratio generally tended to increase the amount of fragmentation in the HSC beams. As can be seen in Figure 7-39(c) and 7-39(d), the specimen with 20M reinforcement experienced excessive fragmentation. Finally, in the #4 series the beam with high-strength concrete showed more brittle spalling (with chunks of cover flying away), while spalling was less severe in the specimen with normal-strength concrete, as shown in Figure 7-39(a) and 7-39 (b).

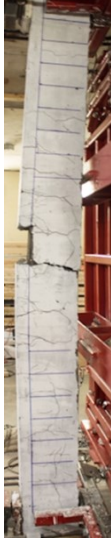
While steel fibres played a crucial role in reducing the debris, the content and type of steel fibres also affected the fragmentations of the specimens. As shown in Figure 7-40, the specimen with long steel fibres (5D) showed more fine fragments

flying away from the beam when compared to companion beam with short steel fibres (ZP). Finally, the specimen with only 1% steel fibres had less debris than the one with 1% steel fibres and stirrups. In summary, the specimen with only steel fibres (HSC-F1(ZP)-20M-0) performed better against blast loads and reduced the debris when compared to specimens with steel fibres and stirrups.

Table 7-1 Summary of principal crack widths

Specimen	Principle Crack Width (mm)						
	Blast 1 17 psi	Blast 2 30 psi	Blast 2A 40 psi	Blast 3 50 psi	Blast 3A 60 psi	Blast 4 70 psi	Blast 5 80 psi
NSC-F0-#4-S	HL	1.70	FAIL	-	-	-	-
HSC-F0-#4-S	HL	1.65	FAIL	-	-	-	-
HSC-F1(ZP)-#4-0	HL	1.14	3.85	12.0	18.80	-	-
HSC-F0-15M-S	HL	1.10	-	5.0	-	-	-
HSC-F1(ZP)-15M-0	HL	HL	-	3.90	-	19.80	-
HSC-F0-20M-0	HL	FAIL	-	-	-	-	-
HSC-F0-20M-S	HL	HL	-	2.0	-	FAIL	-
HSC-F1(ZP)-20M-0	HL	HL	-	0.95	-	9.25	20.90
HSC-F1(ZP)-20M-S	HL	HL	-	1.90	-	9.85	20.20
HSC-F0.5(ZP)-20M-S	HL	HL	-	1.80	-	9.90	N/A
HSC-F0.5(5D)-20M-S	HL	HL	-	0.50	-	5.20	17.95

* HL = Hairline cracks



a) HSC-F0-#4-S after Blast 2A



b) HSC-F0-15M-S after Blast 3



c) HSC-F0-20M-S after Blast 4



c) HSC-F1(ZP)-20M-0 after Blast 5

Figure 7-36 Comparison of damage tolerance in HSC and SFRC beams

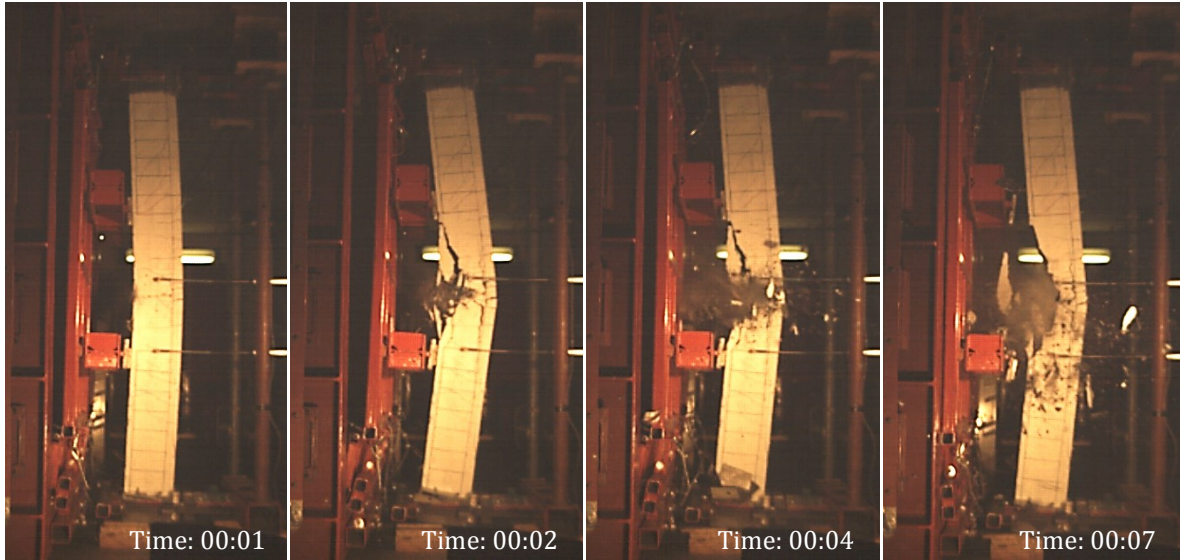


Figure 7-37 HSC-F0-20M-S secondary fragmentation (without fibres)

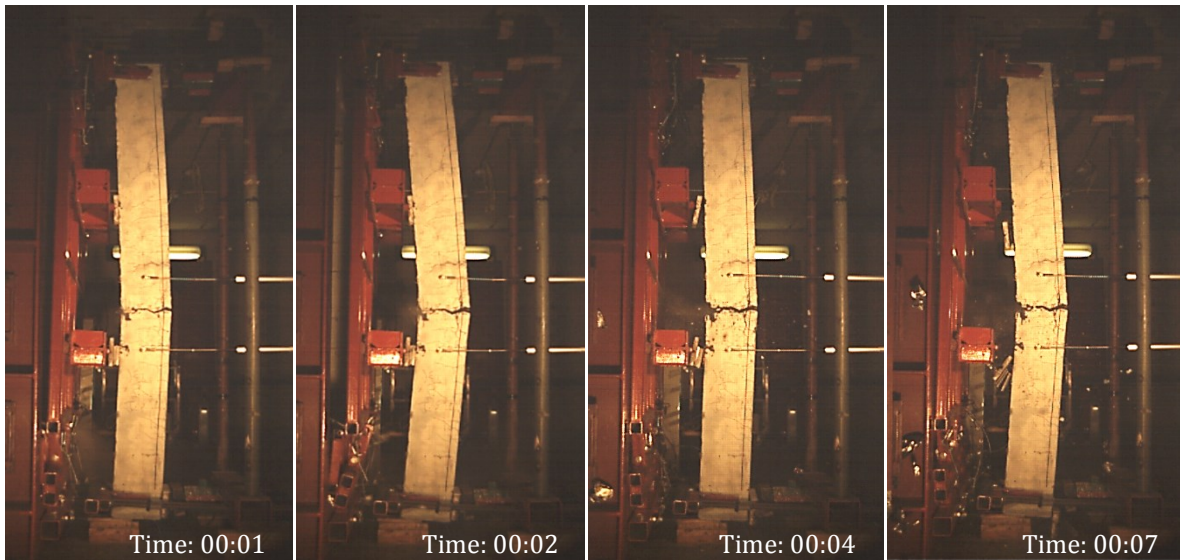


Figure 7-38 HSC-F1(ZP)-20M-0 secondary fragmentation (with fibres)

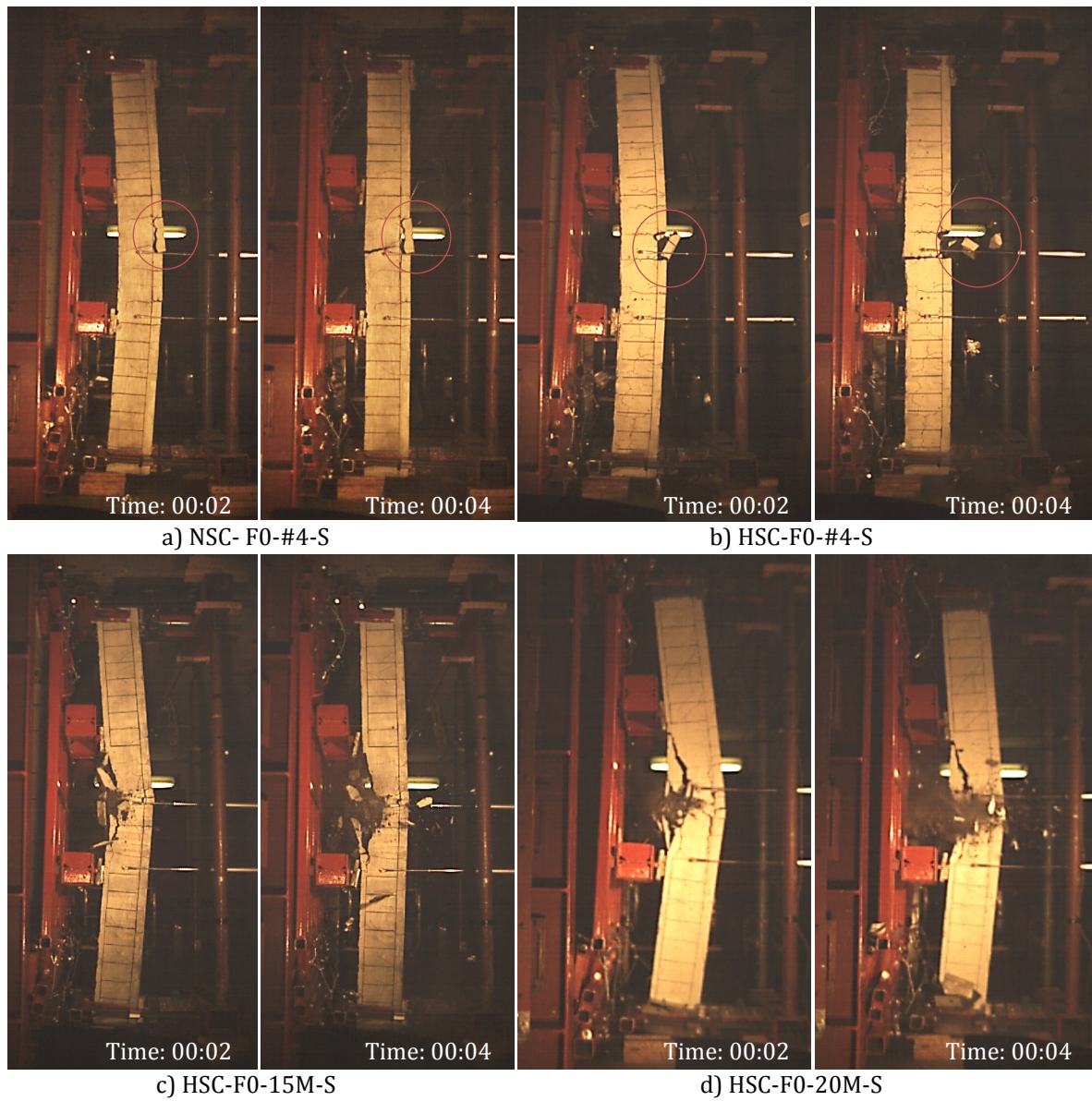


Figure 7-39 Mitigation of secondary fragmentation; effect of reinforcement ratio and concrete strength

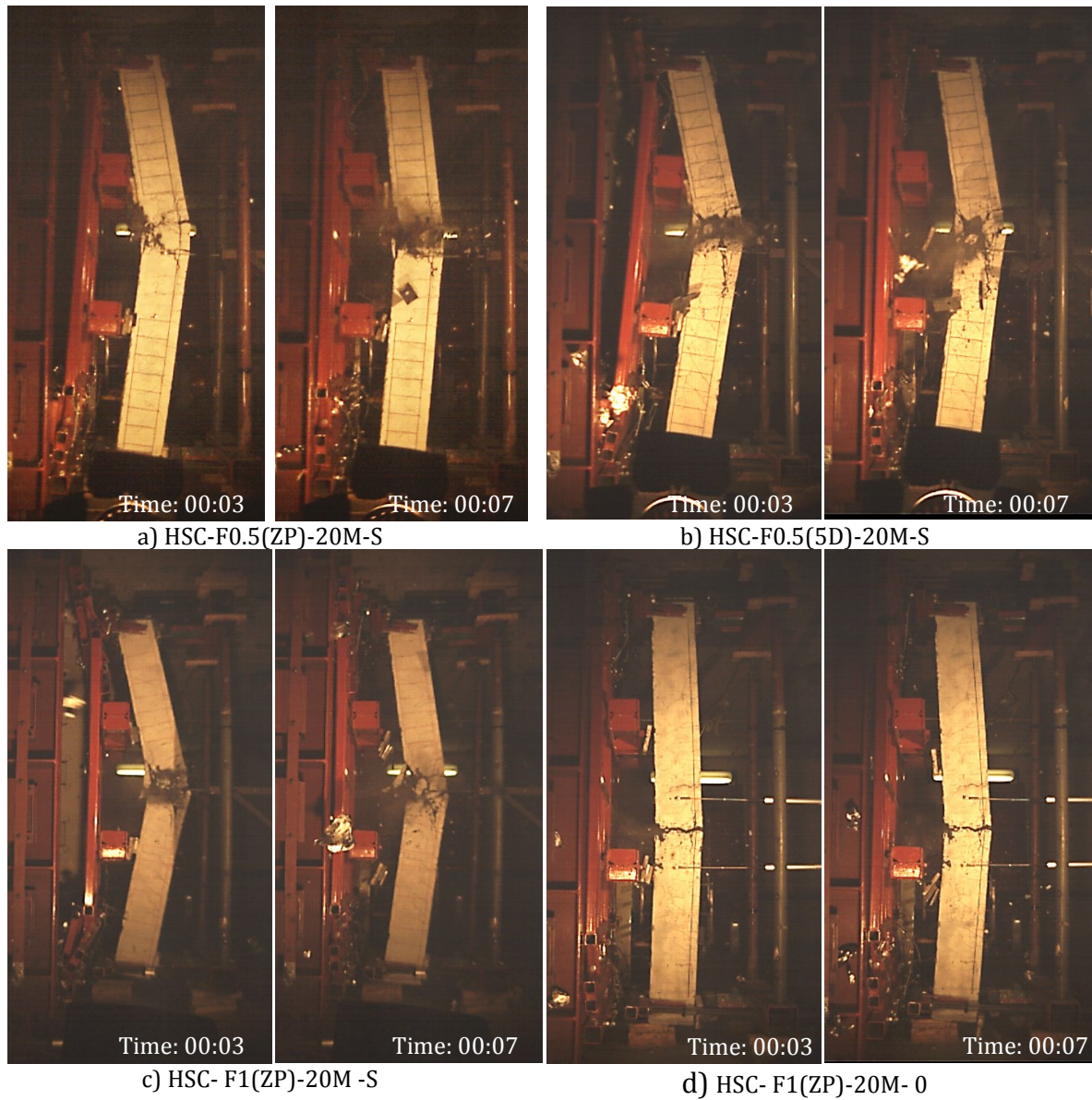


Figure 7-40 Mitigation of secondary fragmentation; effect of fibre properties and combining stirrups and fibres

Chapter 8: Dynamic Analysis

8.1 Chapter Overview

This chapter presents the dynamic analysis of the beams tested in the research program. Analysis was conducted by defining material stress-strain models incorporating high-strain effects for concrete and steel reinforcement, and using a single degree of freedom (SDOF) model and lumped inelasticity approach as described in this chapter.

8.2 Material Models

This section describes the material models used to define the stress-strain response of plain concrete, SFRC and steel reinforcement which are implemented in the dynamic analysis.

8.2.1 Plain Concrete Models

Two models are considered for plain unconfined concrete in this analysis. The Popovics model (1973) is the default model which is considered for normal and high strength concrete. The unconfined version of the Cusson and Paultre model (1995), which was developed using the Popovics model, is also considered in the sensitivity analysis. Figure 8-1 shows stress-strain curves generated using these two models for the SCC and HSC. In terms of the tensile stress-strain curve, a linear relationship with maximum tensile strength taken as $0.33\sqrt{f'_c}$ and slope equal to the modulus of elasticity of concrete was used.

Popovics (1973)

The behaviour of unconfined plain concrete is taken into account using a modified version of the Popovics (1973) model proposed by Porasz (1989), where the factor (k) is calibrated to better capture the post-peak portion of the stress-strain curve (Bentz, 2000). Equation [8-1] presents the relationship used to define the stress-strain response of unconfined concrete. For the ascending branch, the factor (k) is taken as 1.0 (for $\varepsilon/\varepsilon_0 \leq 1.0$). For the descending branch, the factor (k) can be calculated using equation [8-2] (for $\varepsilon/\varepsilon_0 \geq 1.0$). If the value of the strain at peak (ε_0) is unknown, the formulas in equations [8-2] can be used.

$$f_c = \frac{f'_c \beta \left(\frac{\varepsilon}{\varepsilon_0}\right)}{\beta - 1 + \left(\frac{\varepsilon}{\varepsilon_0}\right)^{\beta k}} \quad [8-1]$$

$$\beta = 0.8 + \frac{f'_c}{17}, \quad k = 0.67 + \frac{f'_c}{62}, \quad E_c = 3320\sqrt{f'_c} + 6900, \quad \varepsilon_0 = \frac{f'_c}{E_c} \frac{\beta}{\beta - 1} \quad [8-2]$$

Cusson and Paultre (1995)

In addition to proposing a confinement model, Cusson and Paultre (1995) propose a modified version of their model for unconfined concrete. The resulting model combines the ascending branch of the stress-strain curve proposed by Popovics (1973) with a descending branch which follows the model proposed by Fafitis and Shah (1985). Equation [8-3] represents the stress-strain relationship for this model, including two formulas for the ascending and descending branches. The coefficients (k_u, k_{1u}), in equation [8-3], can be determined by using equations [8-4] and [8-5] while the value of k_{2u} for unconfined concrete is considered to be 1.5 and ε_{50cu} is assumed to be 0.004 mm/mm. When the values of modulus of elasticity and the strain at peak stress are not known, equations [8-6] and [8-7] can be used to estimate their values.

$$f_{cu}(\varepsilon_{cu}) := \begin{cases} f'_{cu} \left[\frac{k_u(\varepsilon_{cu}/\varepsilon'_{cu})}{k_u - 1 + (\varepsilon_{cu}/\varepsilon'_{cu})^{k_u}} \right] & \text{for } \varepsilon_c \leq \varepsilon_{cu} \\ f'_{cu} \cdot \exp[k_{1u}(\varepsilon_{cu} - \varepsilon'_{cu})^{k_{2u}}] & \text{for } \varepsilon_c > \varepsilon_{cu} \end{cases} \quad [8-3]$$

$$k_u = \frac{E_c}{E_c - (f'_{cu}/\varepsilon'_{cu})} \quad [8-4]$$

$$k_{1u} = \frac{\ln(0.5)}{(\varepsilon_{50cu} - \varepsilon'_{cu})^{k_{2u}}} \quad [8-5]$$

$$E_c = 3320\sqrt{f'_{cu}} + 6900 \quad [8-6]$$

$$\varepsilon'_{cu} = 0.001684 + 0.000016f'_{cu} \quad [8-7]$$

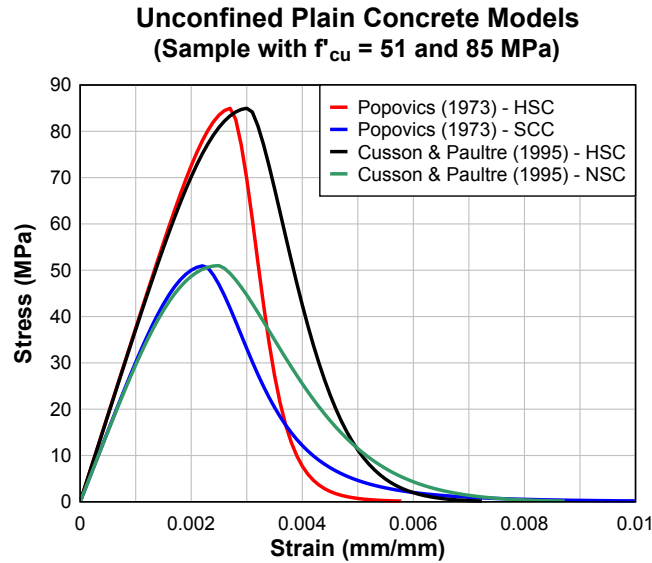


Figure 8-1 Unconfined plain concrete stress-strain models sample

8.2.2 Steel Fibre Reinforced Concrete Models

Despite the fact that several researchers have proposed models to predict the compressive behaviour of steel fibre-reinforced concrete (SFRC), many of these models lack precision as they have been empirically developed based on limited data sets (Bencardino et al. 2008). Three models are considered for SFRC in this research, including two compressive stress-strain models and one tensile stress-strain model. In terms of compression, the model proposed by Mansur et al. (1999) for HSFRC was used for specimens containing 1.0% steel fibres. The model proposed by Ou et al. (2012) is also considered for specimens with fibre contents of 0.5% and 1.0%. The tension behaviour of SFRC is modeled using the model proposed by Lok and Pei (1998). A sample showing the three stress-strain models is included in Figure 8-2.

Mansur et al (1999)

This model was developed using a modified version of the Carreira and Chu (1985) stress-strain relationship as shown in equation [8-8]. For the ascending branch, the two factors k_1 and k_2 are taken as 1.0 until the stress-strain curve reaches the peak stress. For the descending branch, factors k_1 and k_2 can be calculated from equations [8-9], reflecting the influence of steel fibres on post-peak response. If there is no experimental data for the strain at peak and modulus of elasticity, equations [8-10] can be used to estimate the values of ε_0 and E_c .

$$f_c = \frac{f'_c k_1 \beta \left(\frac{\varepsilon}{\varepsilon_0}\right)}{k_1 \beta - 1 + \left(\frac{\varepsilon}{\varepsilon_0}\right)^{\beta k_2}} \quad [8-8]$$

$$\beta = \frac{1}{1 - \frac{f_0}{\varepsilon_0 E_c}}, \quad k_1 = \left(\frac{50}{f_0}\right)^{3.0} \left[1 + 2.5 \left(\frac{V_f l}{D_f}\right)^{2.5}\right]$$

$$k_2 = \left(\frac{50}{f_0}\right)^{1.3} \left[1 - 0.11 \left(\frac{V_f l}{D_f}\right)^{-1.1}\right] \quad [8-9]$$

$$E_c = (10,300 - 400V_f)f_0^{1/3}, \quad \varepsilon_0 = \left[0.00050 + 0.00000072 \left(\frac{V_f l}{D_f}\right)\right]$$

$V_f = \text{volume fraction of steel fibers}, \quad \frac{l}{D_f} = \text{the aspect ratio} \quad [8-10]$

Ou et al. (2012)

This model was also developed from the stress-strain relationship proposed by Carreira and Chu (1985) as defined in equation [8-11]. The compressive strength is defined as a function of peak strength of plain unconfined concrete and the fibre reinforcing index using equation [8-12]. The reinforcing index is the product of fibre content and aspect ratio (fibre length divided by the fibre diameter) as shown in equation [8-15]. For strain at peak and the factor β , equations [8-13] and [8-14] are used.

$$f_{cuf}(\varepsilon_{cuf}) = f'_{cuf} \left[\frac{\beta \left(\frac{\varepsilon_{cuf}}{\varepsilon'_{cuf}} \right)}{\beta - 1 + \left(\frac{\varepsilon_{cuf}}{\varepsilon'_{cuf}} \right)^\beta} \right] \quad [8-11]$$

$$f'_{cuf} = f'_{cu} + 2.35(RI_v) \quad [8-12]$$

$$\varepsilon'_{cuf} = \varepsilon'_{cu} + 0.0007(RI_v) \quad [8-13]$$

$$\beta = 0.71(RI_v)^2 - 2.00(RI_v) + 3.05 \quad [8-14]$$

$$RI_v = v_f(l_f/d_f) \quad [8-15]$$

Lok & Pei (1998)

The model is used to model the stress-strain behaviour of SFRC in tension. The model is composed of pre-cracking and post-cracking stages. Before cracking, the curve is linear with a slope equal to the modulus of elasticity of concrete. After reaching the cracking stress f_{ctf} (defined using equation [8-16] in the current study), the post-cracking stage begins. This stage is modeled using a bilinear relationship, with two processes related to the pull-out of the steel fibres from the concrete matrix. The first process (branch 1) connects the point of cracking to a stress f_2^* and strain ε_2^* as defined in equations [8-17] and [8-18]; it is noted that this branch can either be linearly ascending or descending. In process two, steel fibres enter the final pullout stage and this is reflected using a linear descending equation from strain ε_2^* to a strain of 0.02 mm/mm where the tensile stress reaches 0 MPa. In the current study the matrix bond strength (τ_{bond}) is taken to be between 6 and 7 MPa for hooked-end fibres (Lim et al. (1987)), while the modulus of elasticity for the steel fibres is taken as 200 GPa.

$$f_{ctf} = 0.33\sqrt{f'_c} \quad [8-16]$$

$$f_2^* = \frac{1}{2} v_f \tau_{bond} \frac{l_f}{d_f} \quad [8-17]$$

$$\varepsilon_2^* = \tau_{bond} \frac{l_f}{d_f} \frac{1}{E_{fp}} \quad [8-18]$$

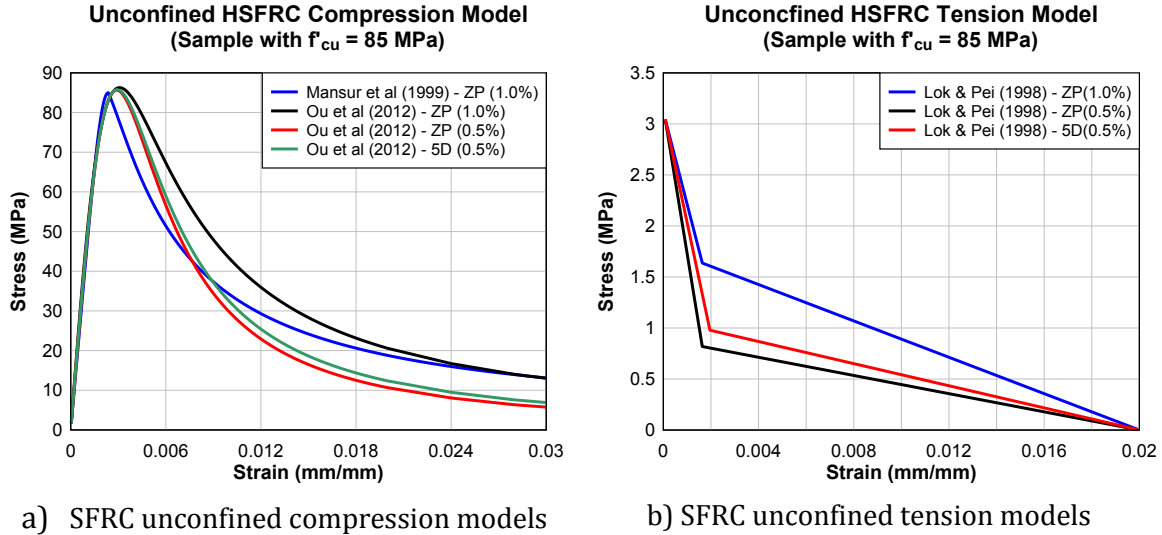


Figure 8-2 Unconfined SFRC stress-strain models sample

8.2.3 Model for Steel Reinforcement in Tension

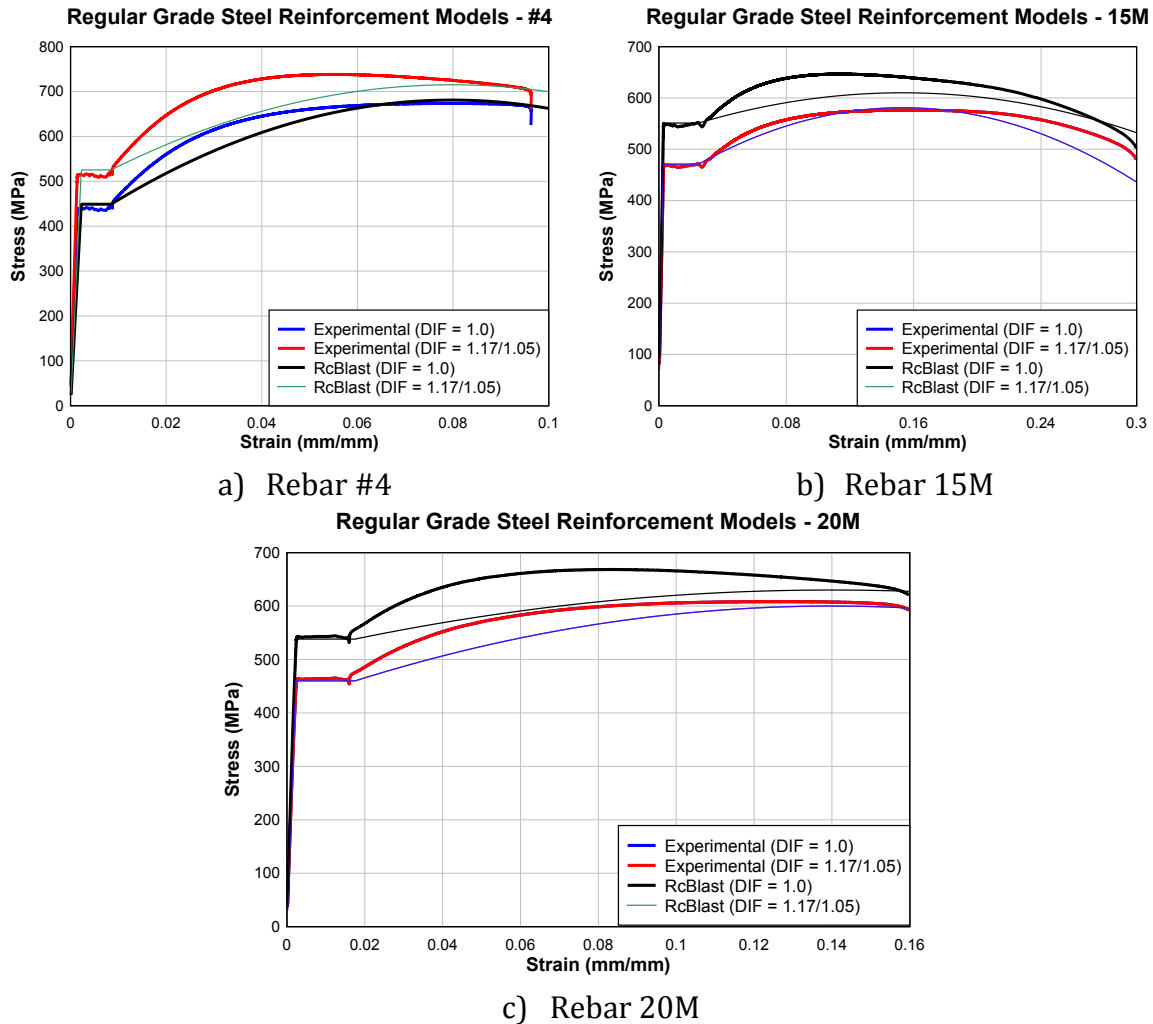
The tensile stress-strain relationship of steel is taken into account using a model proposed by Jacques et al (2012) that is also used in RCBlasT (2014), the software which is used in this research for blast analysis. Equations [8-19] through [8-22] show the parameters used in the stress-strain curve for elastic, post-yield and strain-hardening stages. A comparative sample showing the model predictions along with the experimentally obtained stress-strain relationships for of each reinforcing bar are included in Figure 8-3.

$$f_s = E_s \varepsilon_s \quad \text{for } \varepsilon_s \leq \varepsilon_y \quad [8-19]$$

$$\varepsilon_y = f_y / E_s \quad [8-20]$$

$$f_s = f_y + (\varepsilon_s - \varepsilon_y) \left(\frac{f_{sh} - f_y}{\varepsilon_{sh} - \varepsilon_y} \right) \quad \text{for } \varepsilon_y < \varepsilon_s \leq \varepsilon_{sh} \quad [8-21]$$

$$f_s = f_y + (f_u - f_y) \left[2 \left(\frac{\varepsilon_s - \varepsilon_{sh}}{\varepsilon_u - \varepsilon_{sh}} \right) - \left(\frac{\varepsilon_s - \varepsilon_{sh}}{\varepsilon_u - \varepsilon_{sh}} \right)^2 \right] \quad \text{for } \varepsilon_{sh} < \varepsilon_s \leq \varepsilon_u \quad [8-22]$$



8.3 Dynamic Increase Factors

Concrete and steel are materials which are sensitive to high strain-rate effects. The section describes various models for predicting the dynamic increase factors (DIF) for concrete, SFRC and steel reinforcement.

8.3.1 Dynamic Increase Factors for Designing Model

The UFC-3-340-02 (2008) documentation provides constant DIF values for the design of blast-resistant reinforced concrete elements in the far and close-in design ranges. The dynamic increase factors for concrete and steel reinforcement are independent of strain-rate but are function of the type of stresses acting on the member: bending, diagonal tension, direct shear, bond, and compression (as shown in Table 8-1). For this study, the factors for bending in the far-range are considered with factors of 1.19, 1.17 and 1.05 taken for concrete, steel at yield and steel at ultimate, respectively.

Table 8-1 Dynamic increase factors for design of reinforced concrete elements (UFC-3-340-02, 2008)

Type of stress	Far design range			Close-in design range		
	Reinforcing bars		Concrete	Reinforcing bars		Concrete
	f_{dy}/f_y	f_{du}/f_u	f'_{dc}/f'_c	f_{dy}/f_y	f_{du}/f_u	f'_{dc}/f'_c
Bending	1.17	1.05	1.19	1.23	1.05	1.25
Diagonal tension	1.00	-	1.00	1.10	1.00	1.00
Direct shear	1.10	1.00	1.10	1.10	1.00	1.10
Bond	1.17	1.05	1.00	1.23	1.05	1.00
Compression	1.10	-	1.12	1.13	-	1.16

8.3.2 Dynamic Increase Factors for Concrete

Several models have been developed by researchers to estimate the DIF of concrete in compression or tension. Three DIF models were considered to predict the dynamic strength of plain concrete and HSFRC in this research study. For plain concrete in compression the Saatcioglu et al. (2011) model is considered, while the Zhang and Mindness (2011) model is considered for high-strength SFRC in compression. Finally, the Malvar and Ross (1998) is considered for the dynamic tensile strength of both plain concrete and HSFRC.

Saatcioglu et al. (2011)

This model provides simplified equations for predicting the dynamic compressive strength of plain concrete. Equation [8-23] describes the DIF relationships used in this model. In this research, it is assumed that the strain rate is equal to 1.0 s^{-1} which results in a DIF value of 1.30 for the plain HSC in compression.

$$DIF = \begin{cases} 0.03\ln(\dot{\epsilon}) + 1.30 & \geq 1.0 \text{ for } \dot{\epsilon} < 30 \text{ s}^{-1} \\ 0.55\ln(\dot{\epsilon}) - 0.47 & \text{for } \dot{\epsilon} \geq 30 \text{ s}^{-1} \end{cases} \quad [8-23]$$

Zhang & Mindness (2011)

This model was proposed to predict the dynamic strength of high-strength steel fibre reinforced concrete. The relationships in the model follow those in the CEB model and RCM model (see Ch. 2). As shown in Equation [8-24] the model includes two phases which depend on the strain-rate ($\dot{\epsilon}_{BLT}$). The various variables in the models can be calculated using the equations in [8-25].

$$DIF_c = \begin{cases} (\phi\dot{\epsilon})^\alpha & \text{for } \dot{\epsilon} < \dot{\epsilon}_{BLT} \\ \beta(\dot{\epsilon})^{1/3} & \text{for } \dot{\epsilon} \geq \dot{\epsilon}_{BLT} \end{cases} \quad [8-24]$$

$$\alpha = \frac{\ln(\beta \cdot \dot{\epsilon}_{BLC}^{1/3})}{\ln(\phi \cdot \dot{\epsilon}_{BLT})}, \quad \beta = \frac{5}{9} \cdot \exp\left(\frac{-f'_{cuf}}{230}\right), \quad \phi = 10^5, \quad \dot{\epsilon}_{BLC} = 25 \cdot \exp\left(\frac{f'_{cuf}}{130}\right) \quad [8-25]$$

$\dot{\epsilon}_{BLT}$ = based on the compressive strength of the concrete as shown in Table 2-2

Malvar & Ross (1998)

There is lack of models related to the effects of high strain rates on the tension response of HSFRC. As such, the dynamic strength of HSFRC is considered using the Malvar & Ross (1998) model for concrete in tension. Equation [8-26] expresses the DIF values in this model which follow a similar format to those of the CEB model. The first phase is used when $\dot{\epsilon} \leq 1s^{-1}$ whereas the second phase is used when $\dot{\epsilon} > 1s^{-1}$. The values of $\dot{\epsilon}_s$, δ , and β can be determined from the equations in [8-27].

$$DIF_t = \begin{cases} \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^\delta & \text{for } \dot{\epsilon} \leq 1s^{-1} \\ \beta \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^{1/3} & \text{for } \dot{\epsilon} > 1s^{-1} \end{cases} \quad [8-26]$$

$$\delta = \frac{1}{\left(1 + \frac{8f'_c}{f'_{co}}\right)}, \quad \log \beta = 6\delta - 2, \quad f'_{co} = 10\text{MPa}, \quad \dot{\epsilon}_s = 10^{-6}s^{-1} \quad [8-27]$$

8.3.3 Dynamic Increase Factors for Steel Reinforcement

The Saatcioglu et al. (2011) model was used in this research in order to account for high strain rate effects on the steel reinforcement bars. Equation [8-28] shows the relationships for DIF at yield and ultimate stress proposed in this model. In this study, it is assumed that the strain rate ($\dot{\epsilon}$) is equal to $1 s^{-1}$ which results in values of $DIF_y = 1.30$ and $DIF_u = 1.10$.

$$\begin{aligned} \text{at yield} &\rightarrow DIF_y = 0.034\ln(\dot{\epsilon}) + 1.30 \geq 1.0 \\ \text{at ultimate} &\rightarrow DIF_u = 0.0101\ln(\dot{\epsilon}) + 1.10 \geq 1.0 \end{aligned} \quad [8-28]$$

8.4 Dynamic Analysis Using a Lumped Inelasticity Approach

Single degree of freedom (SDOF) analysis was used to analytically predict the dynamic response of each specimen using equation [8-29].

$$K_{LM}(u(t))m\ddot{u}(t) + R(u(t)) = AP_r(t) \quad [8-29]$$

The variables $u(t)$ and $\ddot{u}(t)$ on the left side of equation [8-29] represent the displacement and the acceleration of the beam at mid-span, while m is the total mass of the system, taken as the mass of the beam plus the load-transfer device (450 kg). The parameter $K_{LM}(u(t))$ is the load mass transformation factor as a function of the displacement and $R(u(t))$ is the resistance of the beam as a function of displacement. On the right side of equation [8-29], A represents the loaded area

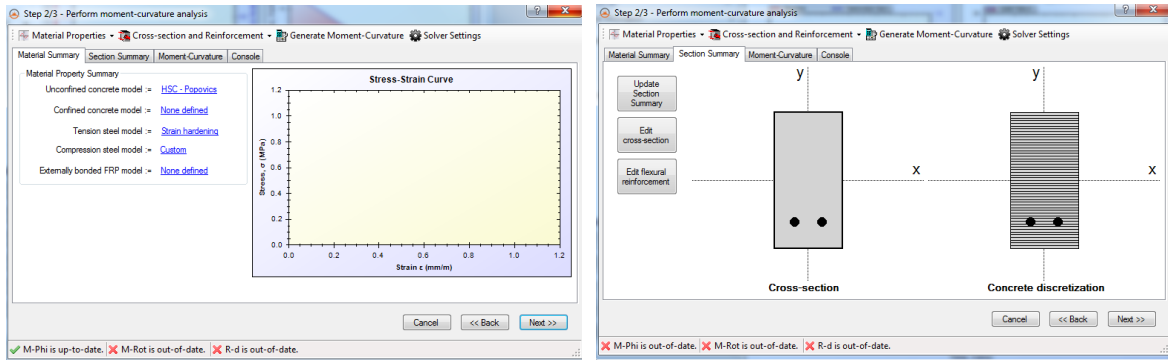
impacted by the blast pressure (taken as 3.4 m^2 based on the LTD area at the shock-tube opening), while $P_r(t)$ is the reflected pressure as a function of time. The resistance functions used in the dynamic analysis were generated following a lumped inelasticity approach (Jacques et al. 2012), with dynamic analysis and solution of the equation of motion conducted software RCBlast (Jacques, 2014).

Analysis in RCBlast begins by defining several parameters such as the specimen cross-sectional dimensions, arrangement of reinforcing bars, and material stress-strain curves which incorporate high strain rate effects, as shown in Figure 8-4. After identifying the beam and material properties, RCBlast conducts sectional analysis to generate a moment-curvature relationship for the beam section. Next, the beam is idealized as a half-span symmetric linear elastic flexural member with a non-linear rotational spring at mid-span as shown in Figure 8-5 to generate the beam's resistance function. The current version of RC Blast does not include an option for generating resistance curves for beams subjected to four-point bending; as such the functions were generated using an excel-based spreadsheet (Figure 8-6).

The procedure used to transform the moment-curvature relationship into a load-deflection curve is illustrated in Figure 8-7. Before yielding, elastic deflections were obtained by integrating curvature using the moment area method as shown in Figure 8-7a. After yield, equation [8-30] was used to determine the displacement at mid-span of the beam, where φ_u and φ_y are the curvature values at ultimate and yield respectively and where l'_p is the plastic hinge length, which is estimated as being equal to the effective depth of the beam cross-section (see Figure 8-7b).

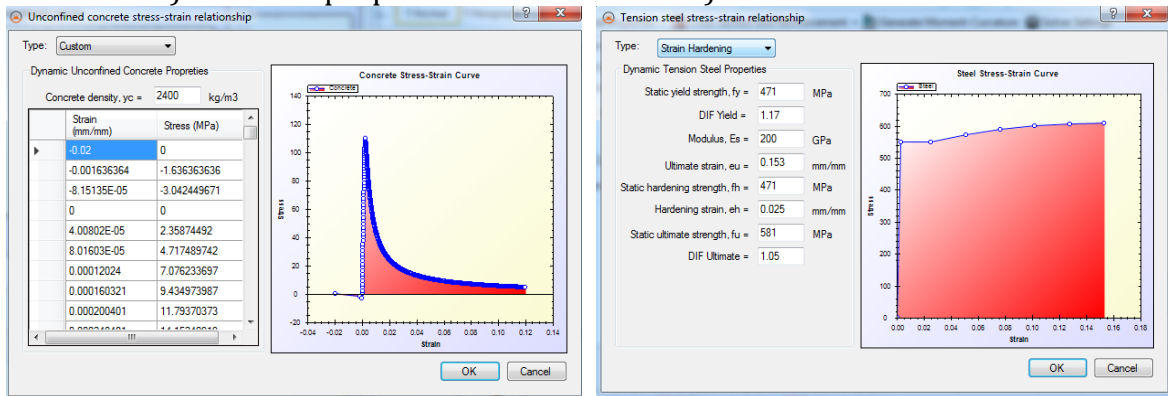
$$\Delta_u = \Delta_y + (\varphi_u - \varphi_y)l'_p \left(l - \frac{l'_p}{2} \right), \quad l = \text{half-span of the beam} \quad [8-30]$$

Resistance functions and equivalent SDOF transformation factors were constructed by solving the force-deflection equation at incrementally increasing load stages. The load transformation factors were taken as being equal to $K_{LM} = 0.6$ and 0.56 before and after yield, respectively. The resulting resistance function and transformation factors were then inputted into RCBlast. An equivalent triangular blast load, having the same peak reflected pressure and impulse found in the experimentally recorded pressure-time history, was used for analysis. The equation of motion was then solved in the program to determine analytical maximum displacements for each beam specimen, as shown in Figure 8-8.



a) Material properties

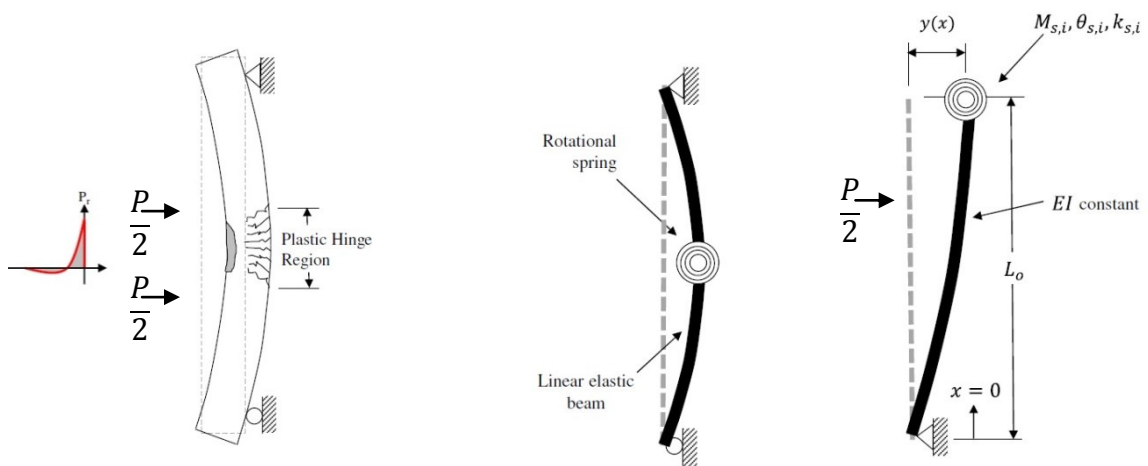
b) Cross-section and steel bars



c) Concrete model

d) Steel reinforcement model

Figure 8-4 RCBlas - Identifying material properties and details of beam cross-section

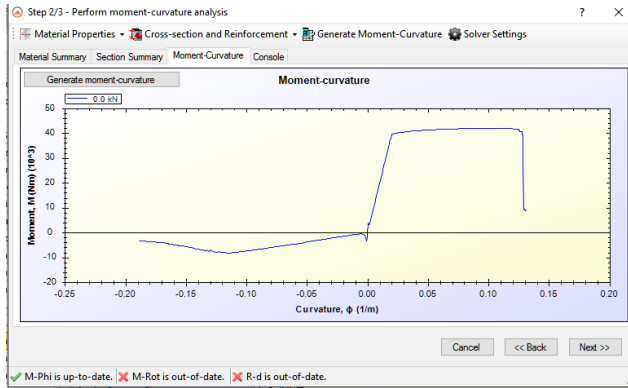


a) Real beam

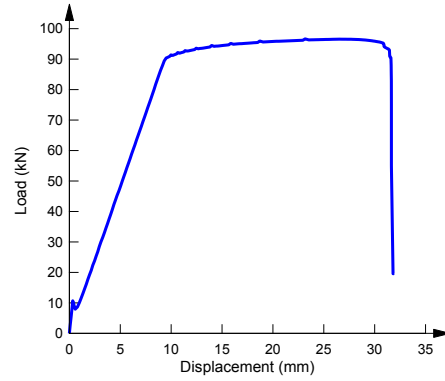
b) Idealized structure

c) Half-span idealized beam

Figure 8-5 Reinforced concrete beam; lumped inelasticity approach (Jacques et al. 2012)

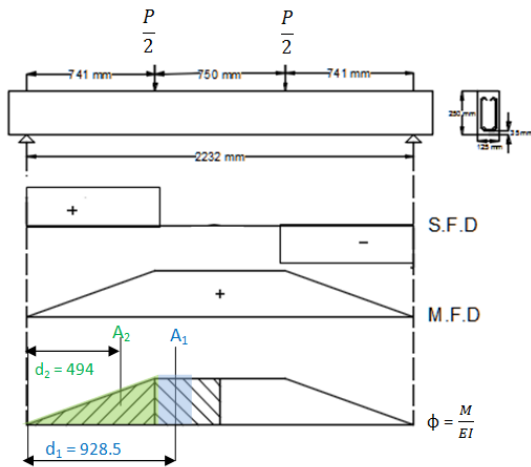


a) Moment-Curvature curve

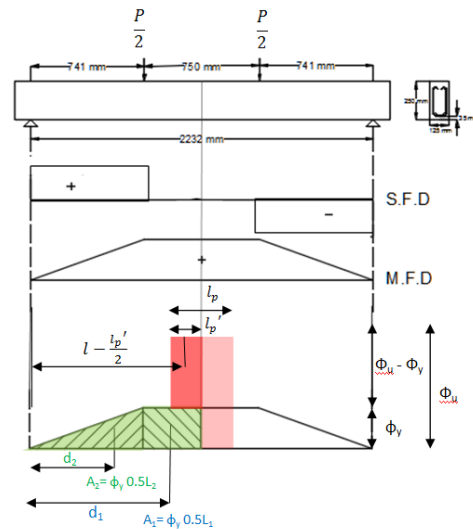


b) Load-Deflection curve

Figure 8-6 RCBlas – Transferring the Moment-Curvature curve into a Load-Deflection curve



a) Curvatures before yield



b) Curvatures after yield

Figure 8-7 Transferring curvatures into deflections for simply supported beam

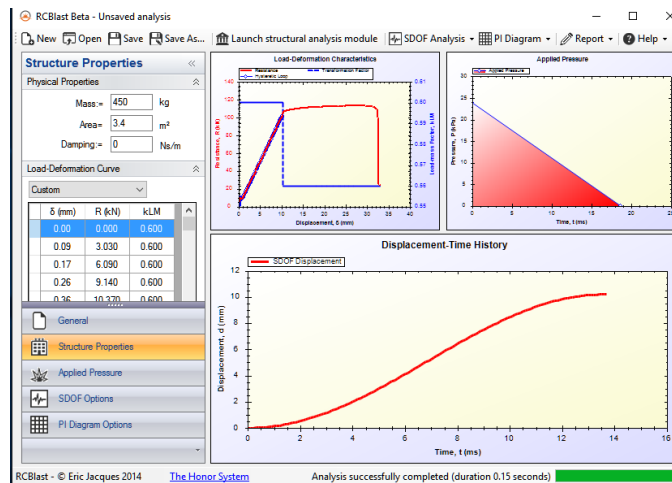


Figure 8-8 RCBlas – The analytical dynamic displacement results

8.5 Dynamic Analysis Results

This section summarizes the results of the dynamic analysis using the analysis procedure discussed in the previous section. The "default" combination of material models and DIFs used in the analysis are summarized in Table 8-2. The response of unconfined plain concrete was modeled using the Popovics (1973) model for compression and linear-slope model for tension. For SFRC, the Mansur et al. (1999) and Lok and Pei (1998) models are considered, while the response of the tension steel is taken into account using the Jacques et al. (2012) model. Dynamic effects are taken into account using the DIF values proposed in the UFC documentation. The sections that follow present the analysis results in terms of prediction of maximum displacements for each series: section 8.5.1 for series #4 specimens, section 8.5.2 for series 15M specimens, and section 8.5.3 for series 20M specimens.

Table 8-2 Selected model combinations; results from the sensitivity analysis

Specimen	Concrete models		Steel models Tension [DIF]
	Compression [DIF]	Tension [DIF]	
NSC-F0-#4-S	Popovics (1973) [1.19]	Linear slope [1.0]	Jacques et al. (2012) [1.17/1.05]
HSC-F0-#4-S	Popovics (1973) [1.19]	Linear slope [1.0]	Jacques et al. (2012) [1.17/1.05]
HSC-F0-15M-S	Popovics (1973) [1.19]	Linear slope [1.0]	Jacques et al. (2012) [1.17/1.05]
HSC-F0-20M-S	Popovics (1973) [1.19]	Linear slope [1.0]	Jacques et al. (2012) [1.17/1.05]
HSC-F0-20M-0	Popovics (1973) [1.19]	Linear slope [1.0]	Jacques et al. (2012) [1.17/1.05]
HSC-F1(ZP)-#4-0	Mansur et al (1999) [1.19]	Lok & Pei (1998) [1.0]	Jacques et al. (2012) [1.17/1.05]
HSC-F1(ZP)-15M-0	Mansur et al (1999) [1.19]	Lok & Pei (1998) [1.0]	Jacques et al. (2012) [1.17/1.05]
HSC-F1(ZP)-20M-0	Mansur et al (1999) [1.19]	Lok & Pei (1998) [1.0]	Jacques et al. (2012) [1.17/1.05]
HSC-F1(ZP)-20M-S	Mansur et al (1999) [1.19]	Lok & Pei (1998) [1.0]	Jacques et al. (2012) [1.17/1.05]
HSC-F0.5(ZP)-20M-S	Ou et al. (2012) [1.19]	Lok & Pei (1998) [1.0]	Jacques et al. (2012) [1.17/1.05]
HSC-F0.5(5D)-20M-S	Ou et al. (2012) [1.19]	Lok & Pei (1998) [1.0]	Jacques et al. (2012) [1.17/1.05]

8.5.1 Dynamic Analysis Results – Series #4

The analytical and experimental dynamic results for this series are summarized in Table 8-3 with presentation of experimental and analytical maximum mid-span displacements ($\delta_{\max}$ and δ_{anls} respectively) and maximum displacement ratios ($\delta_{\text{anls}}/\delta_{\max}$). Statistical data related to the accuracy of the analysis are also reported in Table 8-4 with the mean, standard deviation, coefficient of variance, and average percentage error of the prediction ratios for all blast shots, as well as shot by shot. Bar charts showing the analytical and experimental displacements are also been included in Figure 8-9 and Figure 8-10. Figure 8-11 shows another representation of the accuracy of the SDOF analysis results. Figure 8-10 also compares the load resistance curves used in the analysis (with DIF applied), and also compares the experimental resistance curves as obtained from the static tests with the analytical curves without application of DIF.

Overall, the difference between the analytical and experimental dynamic displacements is fairly small in this series. From the statistical analysis, the results determined considering all shots show an average displacement ratio ($\delta_{\text{anls}}/\delta_{\text{exp}}$) of 1.01 with an average percentage error of 10.37%, standard deviation of 0.12, and a coefficient of variance of 12.13%. This shows that the analytical procedure predicted the dynamic results well for each specimen in this series.

At Blast 1, the mean displacement ratio was 1.05 with a standard deviation of 0.14, concluding the predicted displacements are fairly close to the maximum experimental displacements. Moreover, the average percentage error of the three specimens was 11.67% and the coefficient of variance was 13.15%. Blast 2 also exhibited acceptable results when the analytical and experimental dynamic results were compared. The mean was 1.09 with a standard deviation of 0.14. The average percentage error and the coefficient of variance were 12.89% and 12.46% respectively. The results at Blast 2A were also acceptable, with mean displacement ratio of 0.92 with a standard deviation of 0.05. The average percentage error was 7.90% and the coefficient of variance was 5.02%. Blast 3 and 3A were only applied to the specimen with steel fibres. The results show that the analysis over-predicted the experimental displacement at Blast 3, while the analytical displacement at Blast 3A was similar to the experimental result.

The comparison of the experimental (static) resistance curve with the analytical resistance curve without DIF shows that while maximum load and the general trend of the load-displacement were captured by the analysis, initial beam stiffness was generally over-estimated. Furthermore the maximum displacement

was generally under-estimated in the case of the HSC beams without fibres. These factors may have had an influence on the accuracy of the analytical results.

Table 8-3 Summary of analysis for series #4 specimens

Specimen	Idealized Shockwave				Maximum Mid-Span Displacement			
	Blast (psi)	I _r kPa-ms	P _r kPa	T _d ms	δ_{exp} mm	δ_{anls} mm	$\delta_{anls}/\delta_{exp}$	Error (%)
NSC-F0-#4-S	1	170.62	16.16	25.73	12.70	14.96	1.18	17.80%
	2	209.91	28.50	23.26	29.50	33.12	1.12	12.27%
	2A	347.98	34.20	22.46	47.90	> 37.2*	-	-
HSC-F0-#4-S	1	214.88	18.80	24.14	13.15	14.15	1.08	7.60%
	2	328.23	31.42	22.13	30.40	28.51	0.94	6.22%
	2A	370.13	40.30	21.60	44.50	39.53	0.89	11.17%
HSC-F1.0-#4-0	1	216.60	24.29	24.53	10.63	9.61	0.90	9.60%
	2	344.00	41.50	22.93	18.47	22.20	1.20	20.19%
	2A	428.95	48.21	19.04	35.18	33.55	0.95	4.63%
	3	512.00	55.50	20.36	55.10	47.89	0.87	13.09%
	3A	562.62	68.75	19.48	61.0 [#]	61.69	1.01	1.13%

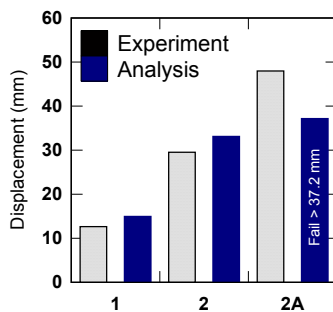
Comments:

* Specimen failed with a displacement more than that showed in the table

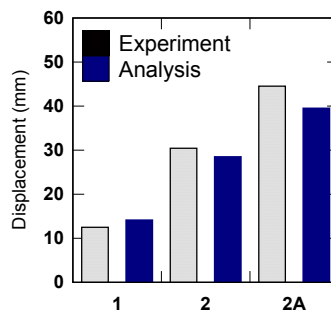
[#] No experimental data taken from the data acquisition

Table 8-4 Analysis statistics for series #4 specimens per blast

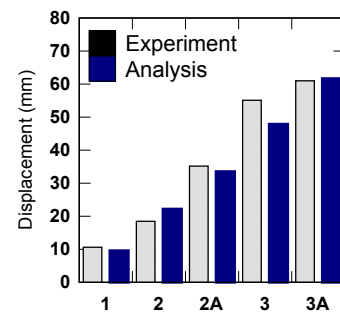
Blast #	Mean	Standard deviation	Coefficient of variance	Error (%)
1-3A	1.01	0.12	12.13%	10.37%
1	1.05	0.14	13.15%	11.67%
2	1.09	0.14	12.46%	12.89%
2A	0.92	0.05	5.02%	7.90%
3	0.87	-	-	13.09%
3A	1.01	-	-	1.13%



a) NSC-F0-#4-S

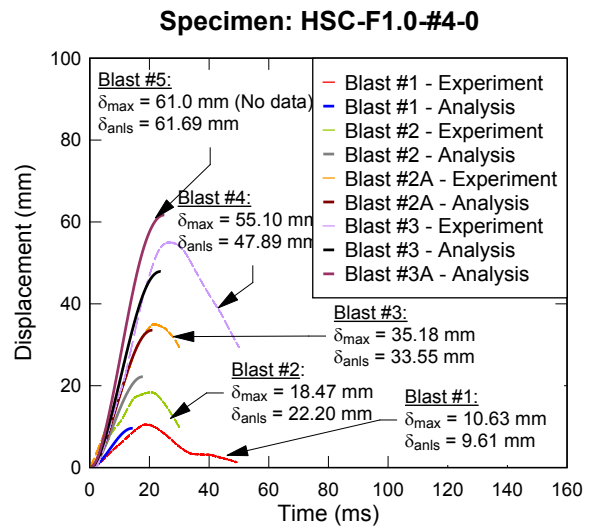
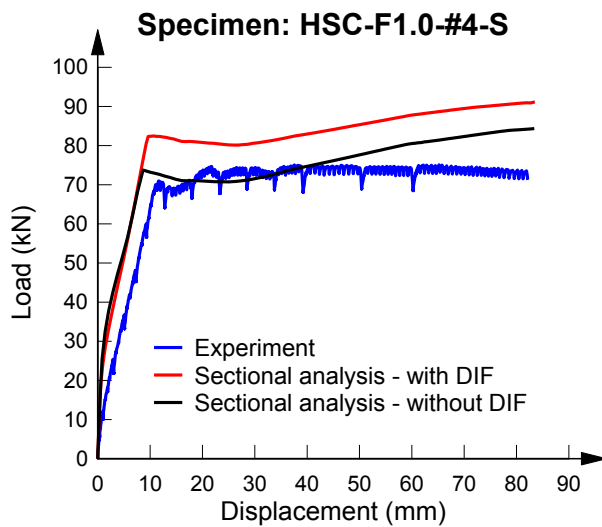
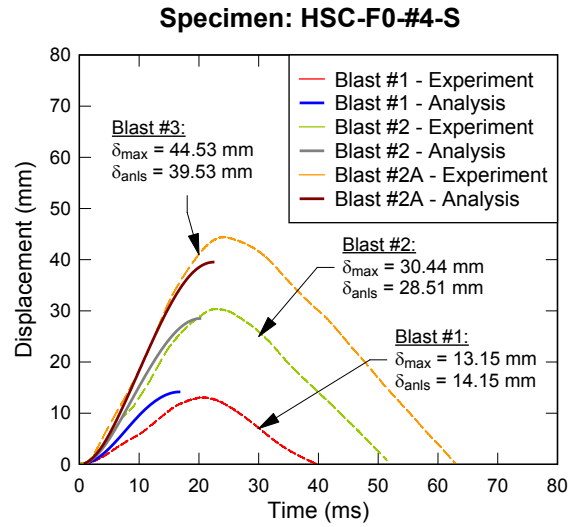
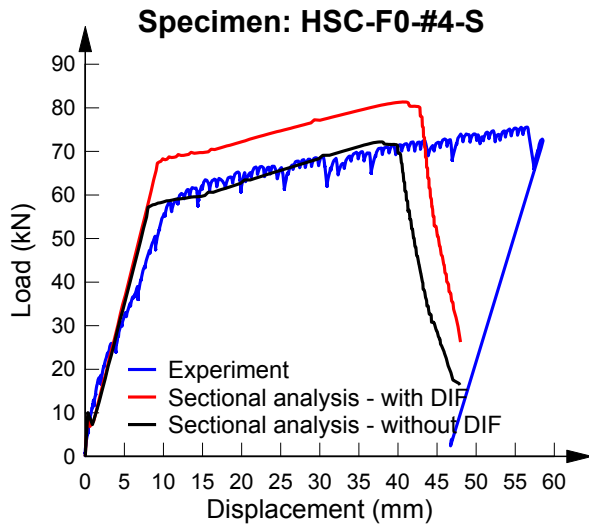
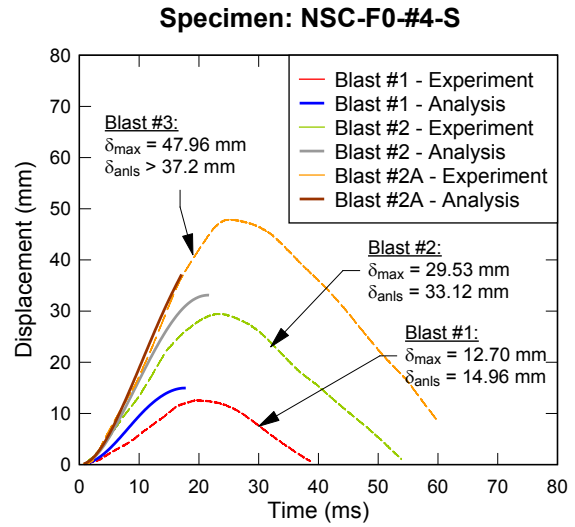
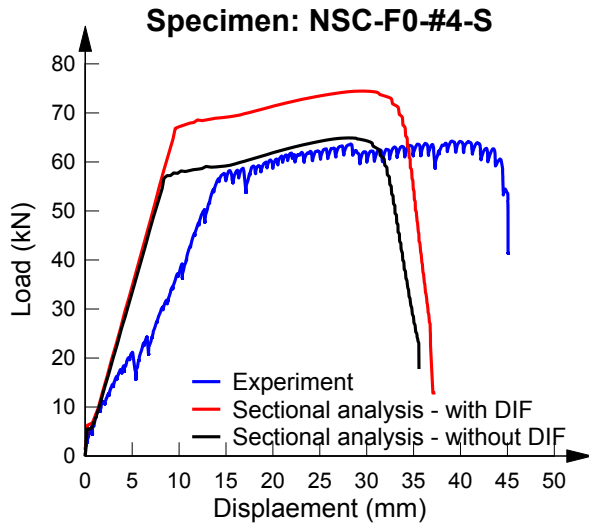


b) HSC-F0-#4-S



c) HSC-F1.0-#4-0

Figure 8-9 Bar charts summarized the analytical and experimental results for series #4 specimens



a) Load-Displacement curve

b) Displacement-Time history

Figure 8-10 Load-deflection relationship and displacement- time history for series #4 specimens

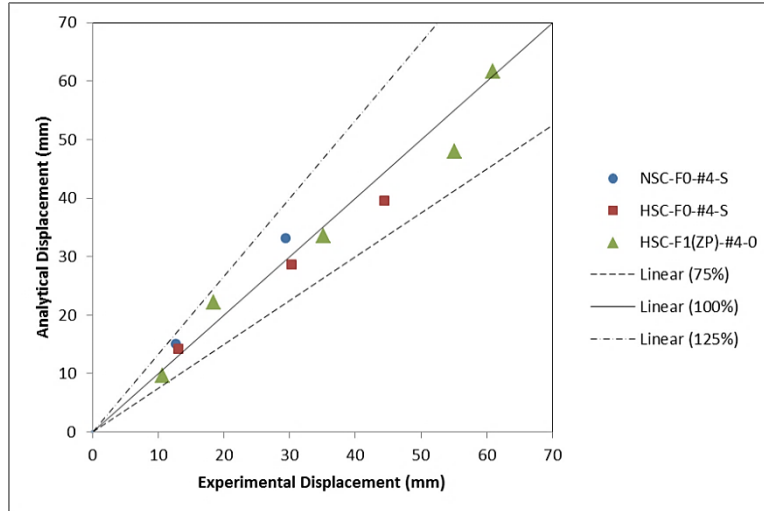


Figure 8-11 Experimental vs. Analytical Displacements – series #4 specimens

8.5.2 Dynamic analysis results – Series 15M

The analytical and experimental dynamic results for this series are summarized in Table 8-5 (δ_{exp} , δ_{anls} and $\delta_{anls}/\delta_{exp}$). Statistical data related to the accuracy of the analysis are also reported in Table 8-6 (mean, standard deviation, coefficient of variance, and average percentage error). Bar charts showing the analytical and experimental displacements are also been included in Figure 8-12 with comparison of displacement response to maximum in Figure 8-13. Figure 8-14 shows another representation of the accuracy of the SDOF analysis results. Figure 8-13 also compares the analytical load resistance curves (with and without DIF) with the experimentally obtained resistance curves from the static tests.

From the statistical analysis, the results considering all shots show an average displacement ratio ($\delta_{anls}/\delta_{exp}$) of 0.84 with an average percentage error of 16.19%, a standard deviation of 0.12, and a coefficient of variance of 14.20%, showing that displacements were generally under-estimated.

The results for Blast 1 show an under-estimation of displacements when comparing the analytical and experimental dynamic results, with a mean of 0.83, average error of 17.38% and a coefficient of variance was 10.1%, with the results being less precise for the specimen with fibres. The predictions at Blast 2 were improved when compared with blast 1 with a mean displacement ratio of 0.93 and average percentage error and coefficient of variance of 7.03% and 7.40% respectively. At Blast 3, the analysis predicted blow-out failure for specimen HSC-F0-15M-S which matches the observation in the experiments (although $\delta_{exp} = 124$ mm this specimen showed severe damage with complete disintegration of concrete in the compression zone at failure, which can be considered a blowout response). At

Blast 3 the comparison to the specimen with steel fibres (HSC-F1-15M-0) shows an analytical-to-experimental displacement ratio of 0.88 with an average percentage error of 11.95%. This ratio was 0.64 with an average percentage error of 36.37% at Blast 4 which indicates the analysis over-predicted the resistance of the fibre-reinforced concrete specimen at this extreme blast loading.

The comparison of the experimental (static) resistance curve with the analytical resistance curves shows that analysis over-predicted the beam stiffness, particularly in the case of the HSC specimen without fibres and this explains the under-estimation of the analytically obtained displacements at Blast 1 and 2. For the specimen with fibres the trend of the resistance curves is similar, although the maximum load was over-estimated which may explain the difference between the analytical and experimental displacements. Furthermore it is noted that the HSFRC specimen was tested under 4 repeated blasts which may have caused damage and reduced fibre contribution at later blasts, whereas the analysis did not account for the effect of accumulated damage on the resistance of the HSFRC beams (e.g. fibre pullout initiated at Blast 3 and therefore resistance at Blast 4 may have been much lower than predicted by the analysis).

Table 8-5 Summary of analysis for series 15M specimens

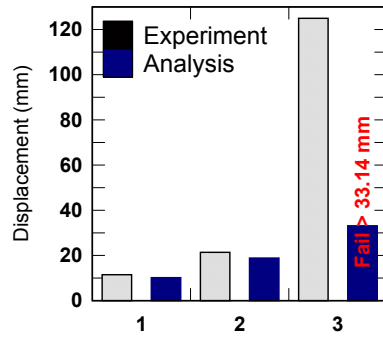
Specimen	Blast (psi)	Idealized Shockwave			Maximum Mid-Span Displacement			
		I _r kPa- ms	P _r kPa	T _d ms	δ_{exp} mm	δ_{anls} mm	$\delta_{anls}/\delta_{exp}$	Error (%)
HSC-F0-15M-S	1	185.03	18.40	18.75	11.50	10.18	0.89	11.48%
	2	269.32	31.50	20.91	21.40	18.86	0.88	11.87%
	3	395.99	42.40	20.65	124.0	> 33.1*	-	-
HSC-F1.0-15M-0	1	213.70	24.49	22.36	10.05	7.71	0.77	23.28%
	2	347.60	40.28	25.76	15.02	14.69	0.98	2.20%
	3	512.00	56.51	25.61	33.31	29.33	0.88	11.95%
	4	657.00	69.48	20.72	78.91	50.21	0.64	36.37%

Comments:

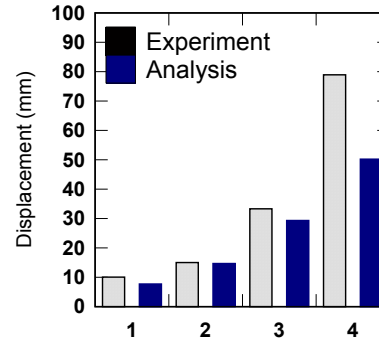
* Specimen failed with a displacement more than that showed in the table

Table 8-6 Analysis statistics for series 15M specimens per blast

Blast #	Mean	Standard deviation	Coefficient of variance	Error (%)
1-4	0.84	0.12	14.2%	16.19%
1	0.83	0.08	10.1%	17.38%
2	0.93	0.07	7.4%	7.03%
3	0.88	-	-	11.95%
4	0.64	-	-	36.37%

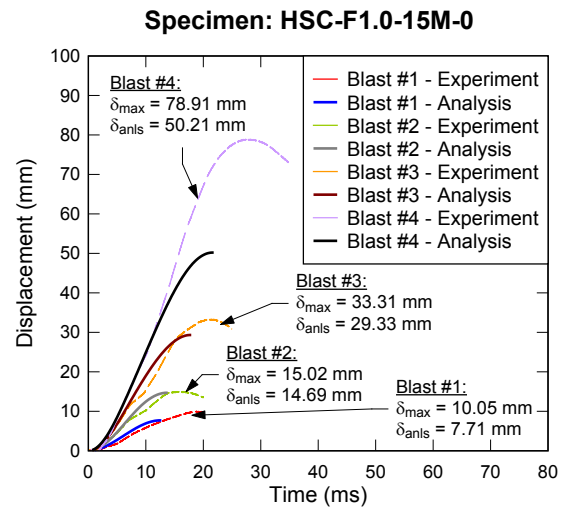
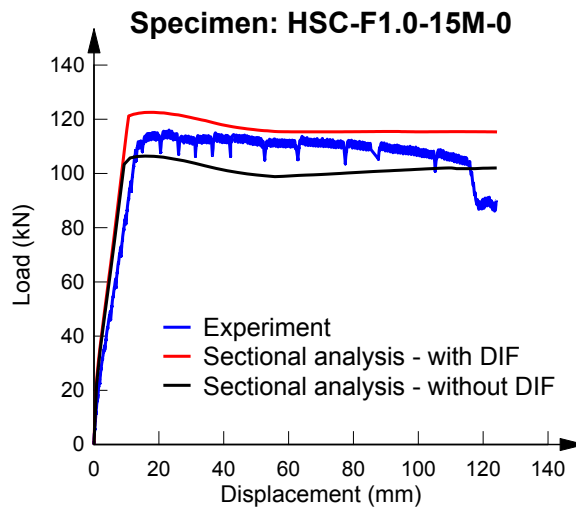
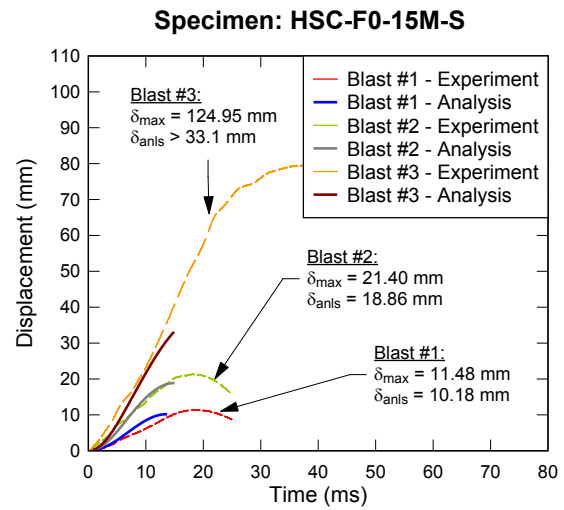
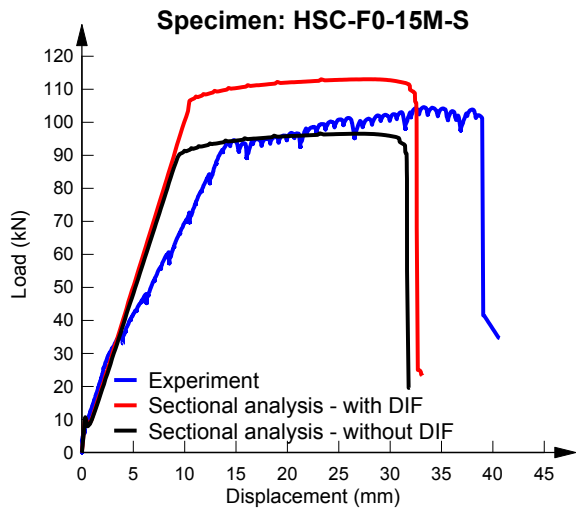


a) HSC-F0-15M-S



b) HSC-F1.0-15M-0

Figure 8-12 Bar charts summarized the analysis results for series 15M specimens



a) Load-Displacement curve

b) Displacement-Time history

Figure 8-13 Load-deflection relationship and displacement-time history for series 15M specimens

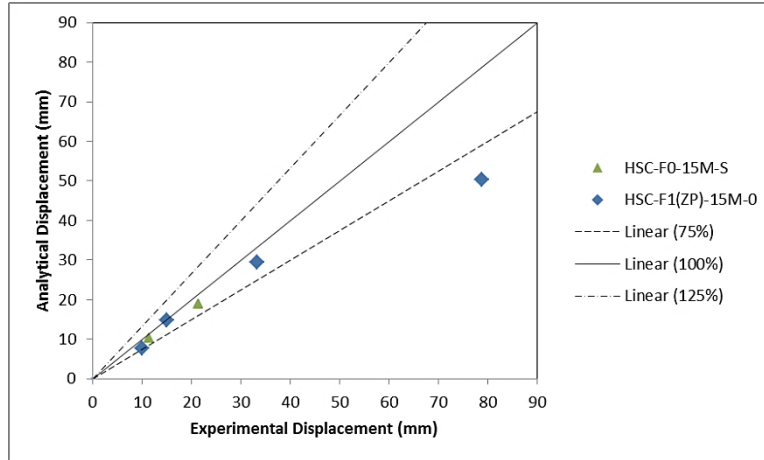


Figure 8-14 Experimental vs. Analytical Displacements – series 15M specimens

8.5.3 Dynamic analysis results – Series 20M

The analytical and experimental dynamic results for this series are summarized in Table 8-7 (δ_{exp} , δ_{anls} and $\delta_{anls}/\delta_{exp}$). Statistical data related to the accuracy of the analysis are also reported in Table 8-8 (mean, standard deviation, coefficient of variance, and average percentage error). Bar charts showing the analytical and experimental displacements are also been included in Figure 8-15 with comparison of displacement response to maximum in Figure 8-16. Figure 8-17 shows another representation of the accuracy of the SDOF analysis results. Figure 8-16 also compares the analytical load resistance curves (with and without DIF) with the experimentally obtained resistance curves from the static tests.

Overall, the difference between the analytical and experimental dynamic displacements is greater in this series when compared to the #4 and 15M series. From the statistical analysis, the results determined from all shots show an average displacement ratio ($\delta_{anls}/\delta_{exp}$) of 0.75 with an average percentage error of 25.05%, standard deviation of 0.20, and coefficient of variance of 26.05%. This shows that the analysis over-predicted the resistance of the beams in this series.

Blast 1 showed noticeable gap between the analytical and experimental dynamic results. The mean of the displacement ratio ($\delta_{anls}/\delta_{exp}$) was 0.80 with a standard deviation of 0.05, with predicted displacements lower than the experimental displacements. Moreover, the average percentage error of the three specimens was 19.87% and the coefficient of variance was 6.82%.

Blast 2 showed acceptable results when comparing the analytical dynamic results to the experimental dynamic results. The mean was 0.88 with a standard

deviation of 0.06, with average percentage error and coefficient of variance of 12.45% and 6.35% respectively. It is noted that analysis is less accurate for the specimens with stirrups and fibres.

The observation from Blast 3 is fairly similar to that at Blast 2, where the mean displacement ratio was 0.87 with a standard deviation of 0.13, average percentage error of 13.14% and coefficient of variance of 14.54%.

Blast 4 showed large differences between the analytical and experimental results. The mean displacement ratio was 0.71 with a standard deviation of 0.16, showing the analysis over-predicted the resistance of most beams in this series, particularly in the case of the specimen with a combination of 1% fibres and stirrups (HSC-F1-20M-S) and the specimen with 0.5% 5D fibres and stirrups (HSC-F0.5(5D)-20M-S). The analysis did however capture the blow-out failure of specimen HSC-F0-20M-S.

Blast 5 showed even larger differences where the analysis was unable to capture the failure of the fibre-reinforced concrete specimens. The displacement ratio was 0.42 with a standard deviation of 0.14, which means the predicted results are much lower when compared to the experimental dynamic results. The average percentage error was 58.06% and the coefficient of variance was 34.37%.

Comparison of the resistance curves in Figure 8-16 shows stiffness was generally over-estimated by the analysis, although the general trend of the load-deflection response was captured for most specimens.

In summary, the analysis under-predicted the experimental dynamic results in this series. The over-estimation of resistance is greater in the last two shots (Blasts 4 and 5) in the HSFRC specimens; the analysis did not consider accumulated damage in these specimens and this may explain the reduced accuracy of the analysis at later blasts. The results are also generally found to be less accurate for specimens HSC-F1-20M-S and HSC-F0.5(5D)-20M-S, and this result points to the over-estimation of the resistance of specimens with combination of 1% ZP /0.5% 5D fibres and stirrups.

Table 8-7 Summary of analysis for series 20M specimens

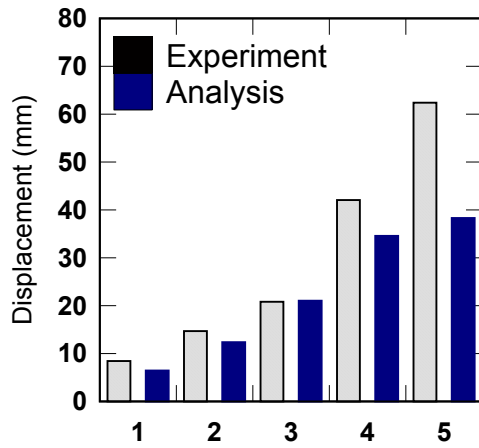
Specimen	Idealized Shockwave				Maximum Mid-Span Displacement			
	Blast (psi)	I _r kPa-ms	P _r kPa	T _d ms	δ_{exp} mm	δ_{anls} mm	$\delta_{anls}/\delta_{exp}$	Error (%)
HSC-F0-20M-S	1	245.48	33.7	21.13	10.4	8.38	0.81	19.42%
	2	316.71	41.15	21.32	15.1	13.44	0.89	10.99%
	3	515.78	77.3	24.14	32.9	24.03	0.73	26.96%
	4	662.17	99.91	21.61	118	> 25.2*	-	-
HSC-F1.0(ZP)-20M-0	1	244.0	23.4	25.3	8.44	6.46	0.77	23.46%
	2	387.0	41.8	24.03	14.69	12.38	0.84	15.72%
	3	571.2	57.1	22.22	20.82	21.04	1.01	1.06%
	4	734.5	69.6	32.84	42.06	34.59	0.82	17.76%
	5	738.0	75.3	24.65	62.4	38.32	0.61	38.59%
HSC-F1.0(ZP)-20M-S	1	260.4	23.8	21.08	9.21	6.73	0.73	26.93%
	2	358.0	41.4	24.06	14.91	11.87	0.80	20.39%
	3	552.0	57.4	21.78	27.64	20.59	0.74	25.51%
	4	732.7	71.1	22.7	67.03	35.47	0.53	47.08%
	5	774.0	79.1	24.79	153.01	43.19	0.28	71.77%
HSC-F0.5(ZP)-20M-S	1	248.0	25.8	22.39	8.54	7.15	0.84	16.28%
	2	370.0	44.5	23.94	13.96	12.91	0.92	7.52%
	3	564.0	59.2	23.4	24.09	22.52	0.93	6.52%
	4	750.0	78.2	26.73	48.55	42.01	0.87	13.47%
	5	792.3	90.8	22.58	119.52	52	0.44	56.49%
HSC-F0.5(5D)-20M-S	1	278.0	25.7	21.17	8.45	7.33	0.87	13.25%
	2	416.4	44.2	22.97	14.57	13.46	0.92	7.62%
	3	565.0	63.5	19.64	25.37	23.93	0.94	5.68%
	4	740.9	74.5	28.27	62.66	38.68	0.62	38.27%
	5	792.3	90.8	24.44	148.58	51.42	0.35	65.39%

Comments:

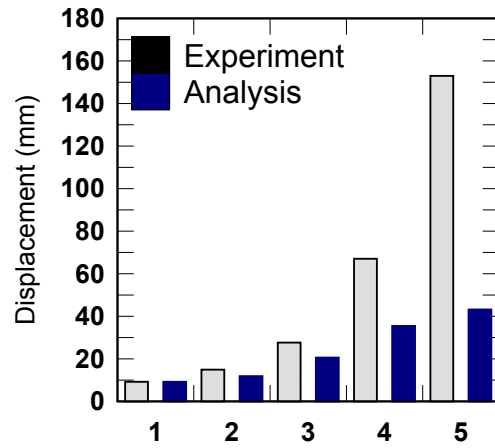
* Specimen failed with a displacement more than that showed in the table

Table 8-8 Analysis statistics for series 20 specimens per blast

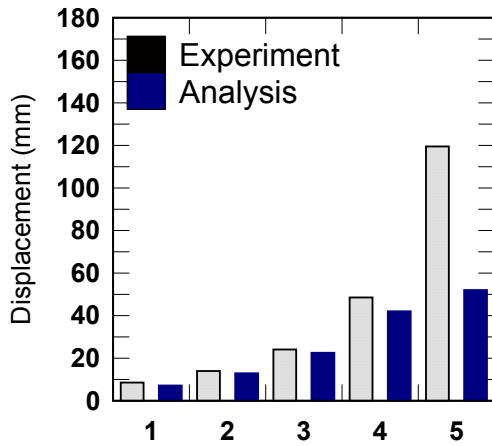
Blast #	Mean	Standard deviation	Coefficient of variance	Error (%)
1-5	0.75	0.20	26.05%	25.05%
1	0.80	0.05	6.82%	19.87%
2	0.88	0.06	6.35%	12.45%
3	0.87	0.13	14.54%	13.14%
4	0.71	0.16	22.76%	29.15%
5	0.42	0.14	34.37%	58.06%



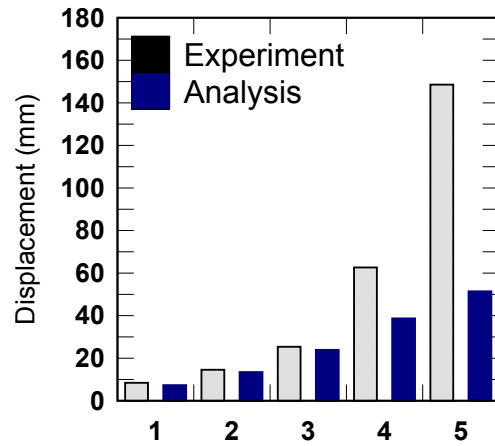
a) HSC-F1.0-20M-0



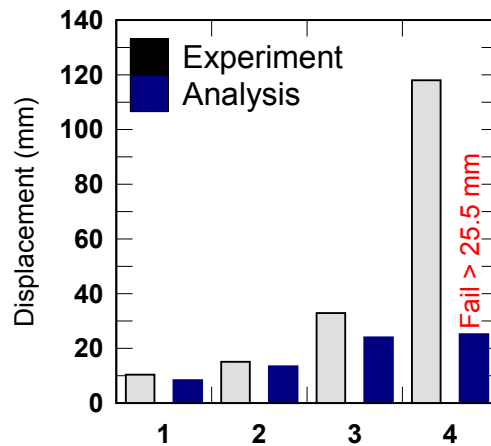
b) HSC-F1.0-20M-S



c) HSC-F0.5-20M-S

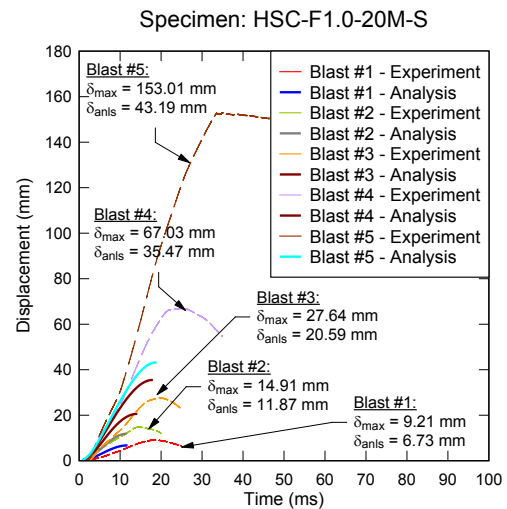
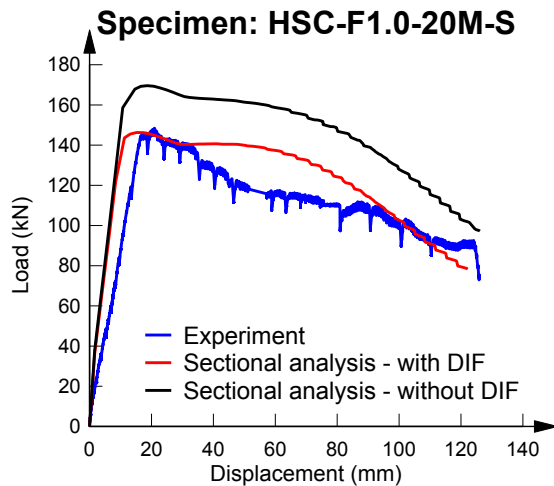
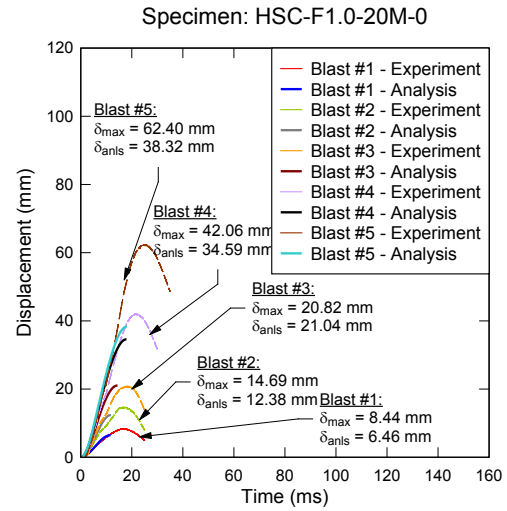
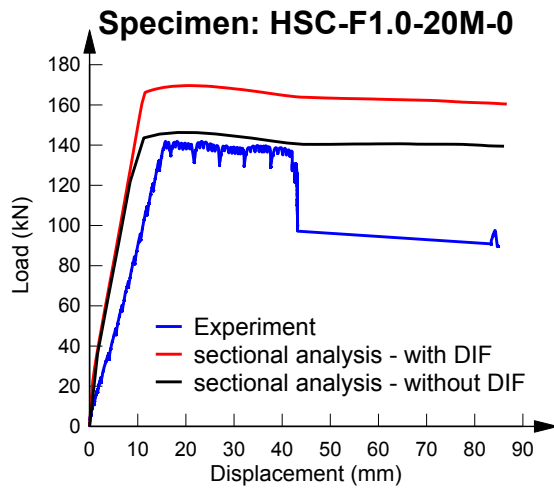
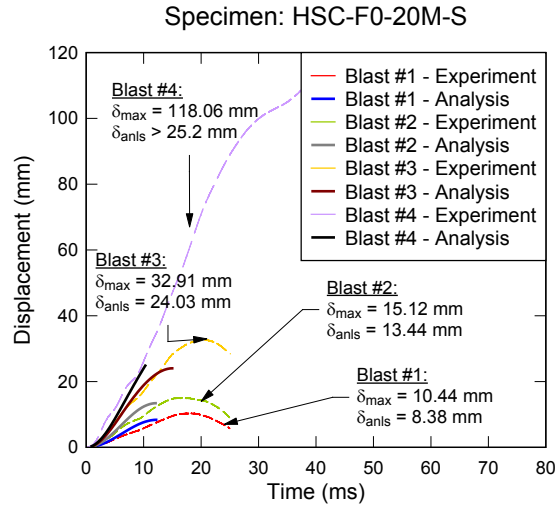
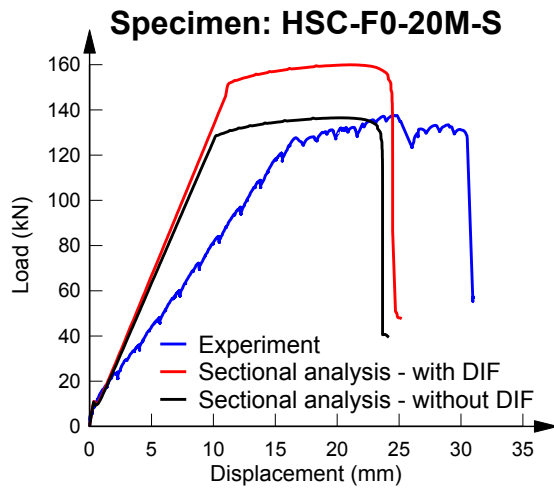


d) HSC-F0.5(5D)-20M-S



e) HSC-F0-20M-S

Figure 8-15 Bar charts summarizing the analysis results for series 20M specimens



a) Load-Displacement curve

b) Displacement-Time history

Figure 8-16 Load-deflection relationship and displacement-time history for series 20M

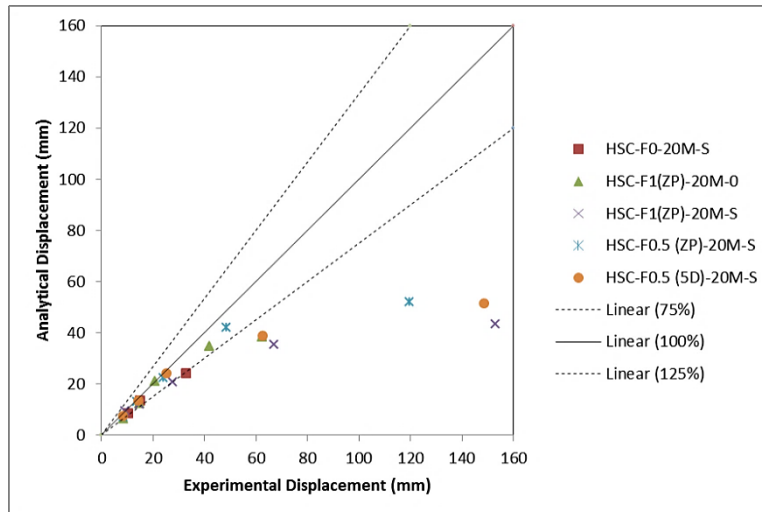


Figure 8-17 Experimental vs. Analytical Displacements – series 20M specimens

8.6 Sensitivity Analysis

This section examines the effect of various modeling parameters on the dynamic analysis. The effect of the choice of concrete model in compression and DIF model are examined.

8.6.1 Sensitivity Analysis – Concrete in Compression Model Selection

This section examines the effect of concrete model selection on the analysis results. Four models were selected for the sensitivity analysis, including Popovics (1973) vs. Cusson & Paultre (1995) for plain concrete, and Mansur et al (1990) vs. Ou et al (2012), for HSFRC in compression. In all cases steel response is considered using the Jacques et al. (2012) model and the Lok & Pei (1998) model was selected for tension in HSFRC. DIF values correspond to those in the UFC-3-340-02 (2008) documentation. The summary of the models and analysis cases is provided in Table 8-9 and the results for the series #4, 15M, and 20M beams are presented in the form of tables (Table 8-10 to Table 8-12) and bar chart comparisons (Figure 8-18 to Figure 8-22).

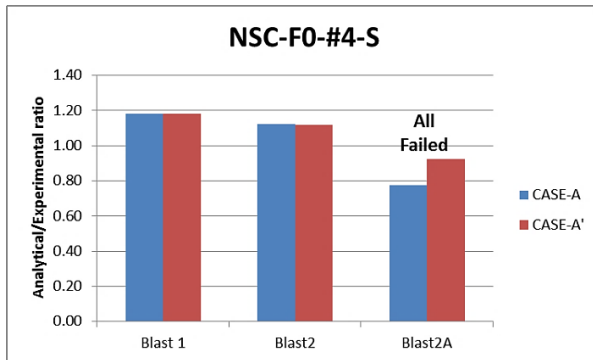
Table 8-9 Sensitivity analysis – models combination for series #4, series 15M, and series 20M

Concrete Models		Steel Models
		Jacques et al. (2012)
NSC/HSC	Popovics (1973)	CASE-A
	Cusson & Paultre (1995)	CASE-A'
HSFRC	Mansur et al (1990)	CASE-B
	Ou et al (2012)	CASE-B'

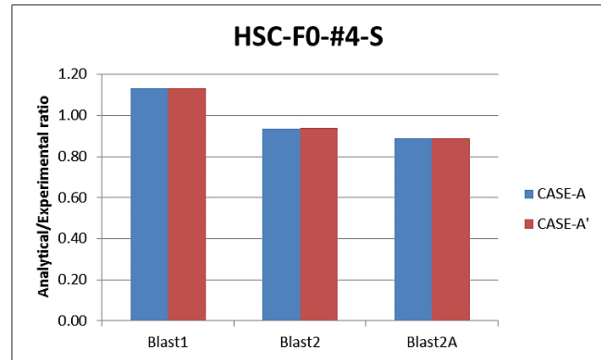
For the NSC and HSC specimens the analysis shows that the effect of concrete compression model is not significant. For example for the #4 series the average $\delta_{anal}/\delta_{exp}$ ratio remains the same regardless of compression model (1.15 in both cases). Similar observation is made in the 15M and 20M series with differences of 0.01 in the ratio for both models. However it should be noted that the Cusson and Paultre model failed to capture failure at Blast 2A for the NSC-F0-#4-S and Blast 3 for the HSC-F0-15M-S specimens due to slightly higher failure strain for concrete in compression in this model. Regarding the HSFRC specimens, the results using the Mansur et al (1990) and Ou et al (2012) were similar; however the results were slightly more accurate for the Mansur et al (1990). For example, the mean analytical-to-experimental ratio for the Mansur and Ou models was 0.99 vs. 0.98 for the #4 series, 0.82 vs. 0.80 for the 15M series and 0.74 vs. 0.73 for the 20M series. In summary the sensitivity analysis shows minor effects associated with the choice of unconfined concrete model for HSC and HSFRC.

Table 8-10 Sensitivity analysis – Analysis statistics for series #4

Specimen ID NSC/HSC model	Mean of ($\delta_{\text{anls}}/\delta_{\text{exp}}$)	
	Popovics (1973)	Cusson & Paultre (1995)
NSC-F0-#4-S	1.15	1.15
HSC-F0-#4-S	0.99	0.99
Average	1.04	1.04
HSFRC model	Mansur et al (1990) Ou et al (2012)	
	HSC-F1(ZP)-#4-0	0.99 0.98



a) Normal strength concrete



b) High strength concrete

Figure 8-18 Sensitivity analysis –Concrete in compression model, specimens plain NSC and HSC

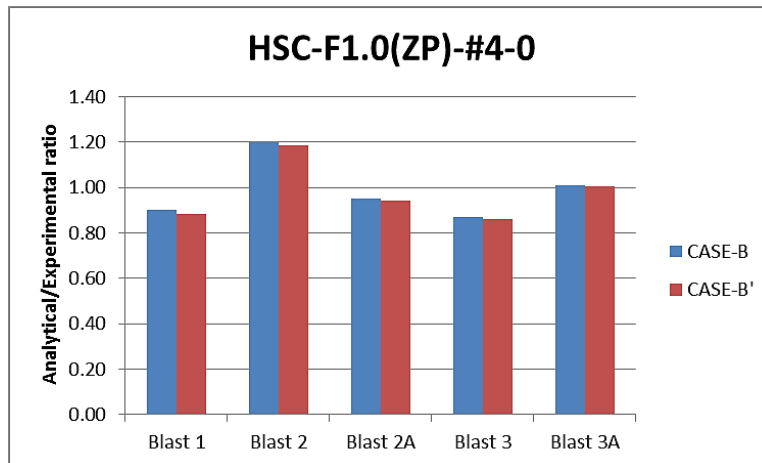
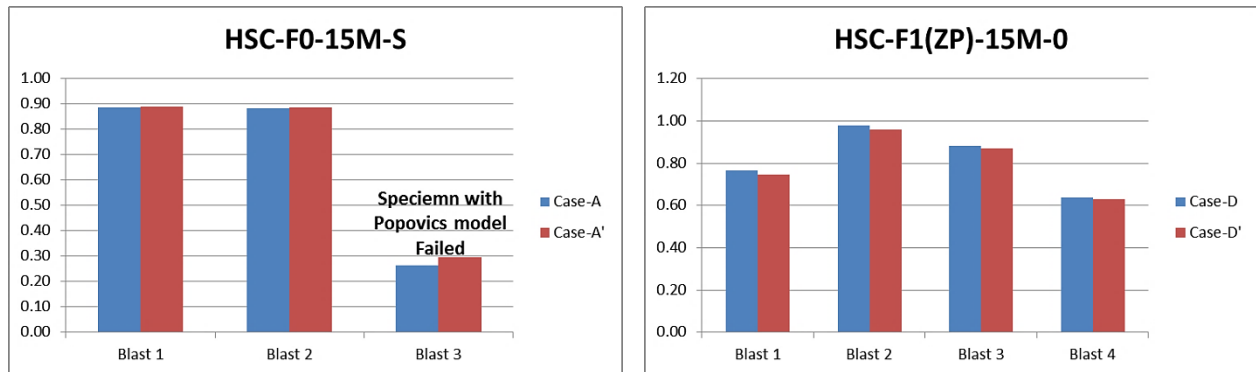


Figure 8-19 Sensitivity analysis – Concrete in compression model, specimen HSFRC

Table 8-11 Sensitivity analysis – Analysis statistics for series 15M

Specimen ID HSC model	Mean of ($\delta_{anls}/\delta_{exp}$)	
	Popovics (1973)	Cusson & Paultre (1995)
HSC-F0-15M-S	0.88	0.89
HSFRC model	Mansur et al (1990)	Ou et al (2012)
HSC-F1(ZP)-15M-0	0.82	0.80



a) Plain concrete specimen

b) HSFRC specimen

Figure 8-20 Sensitivity analysis – Concrete in compression model, plain HSC and HSFRC specimens

Table 8-12 Sensitivity analysis – Analysis statistics for series 20M

Specimen ID HSC model	Mean of ($\delta_{anls}/\delta_{exp}$)	
	Popovics (1973)	Cusson & Paultre (1995)
HSC-F0-20M-S	0.81	0.80
HSCFRC model	Mansur et al (1990)	Ou et al (2012)
HSC-F1(ZP)-20M-0	0.81	0.79
HSC-F1(ZP)-20M-S	0.62	0.60
Average	0.74	0.73

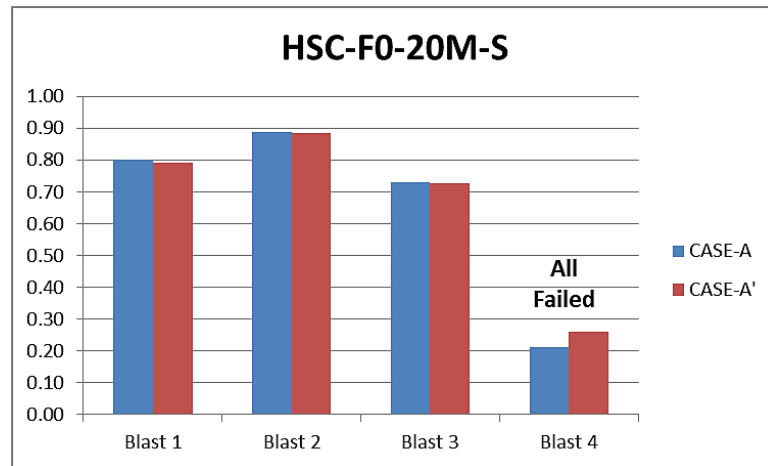


Figure 8-21 Sensitivity analysis – Concrete in compression model, Plain HSC specimen

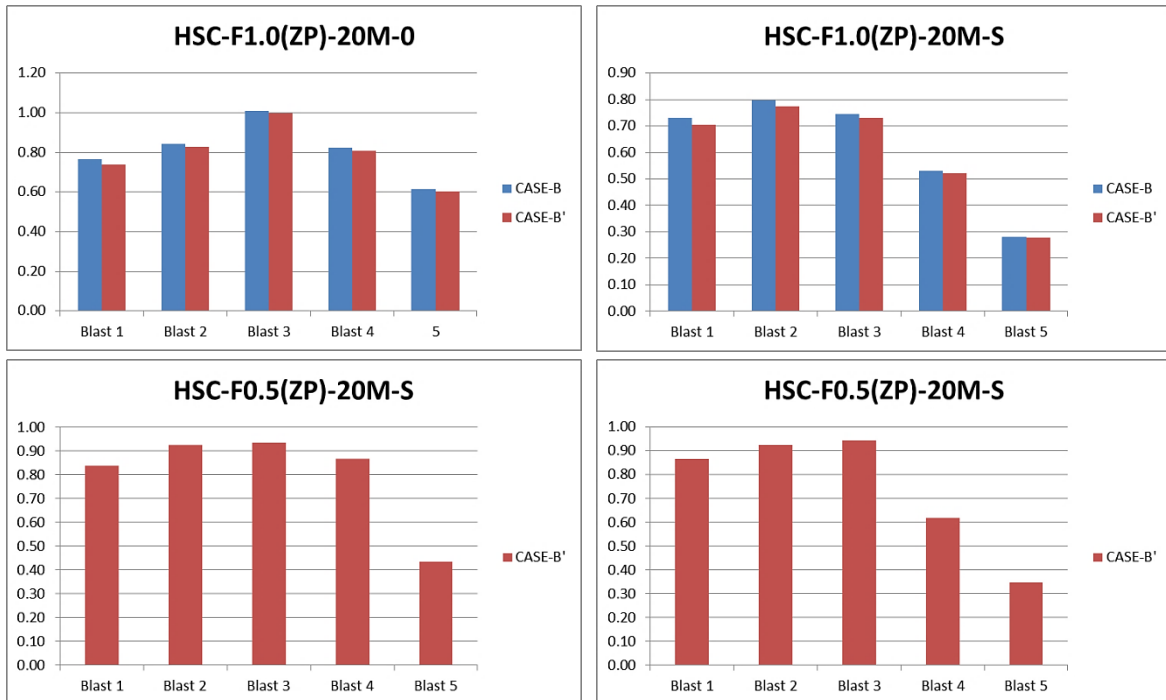


Figure 8-22 Sensitivity analysis – Concrete in compression model, HSFRC specimens

8.6.2 Sensitivity Analysis – Dynamic Increase Factors Selection

This section examines the effect of dynamic increase factor model on the analytical predictions for maximum displacement. For plain concrete specimens, Case A uses the DIF values proposed in the UFC-3-340-02 (2008) documentation with DIF of 1.19 for concrete in compression, 1.17 for steel at yield, and 1.05 for steel at ultimate. Case B uses DIF values estimated using the Saatcioglu et al. (2011) models for concrete in compression and steel in tension, with DIF of 1.3 for concrete and 1.3/1.1 for steel at yield/ultimate, as shown in Table 8-13. For HSFRC specimens, three combinations of dynamic increase factors were examined. Case C uses the UFC-3-340-02 (2008) suggested DIF values. Second, Case D considered the Zhang & Mindness (2011) model for HSFRC in compression with a DIF of 1.28 as well as the Saatcioglu et al. (2011) model for steel reinforcement with DIF of 1.3/1.1 at yield/ultimate, while DIF for concrete in tension is taken as being equal to 1.0. The third (Case E) uses the same dynamic increase factors as in Case D with the exception of using the Malvar & Ross (1998) model for concrete in tension with a DIF of 1.22, as shown in Table 8-14.

The summary of the models and cases is provided in Table 8-13 and Table 8-14. The analysis results for the #4, 15M, and 20M series beams are presented in the form of tables (Table 8-15 to Table 8-20) and bar chart comparisons (Figure 8-23 to Figure 8-27).

The results from all series shows that the analysis is sensitive to the choice of dynamic increase factor, with the best results generally obtained using the UFC-3-340-02 DIF values. In the case of the HSC specimens in the #4 series, the average $\delta_{anal}/\delta_{exp}$ ratio for Case A (UFC) vs. Case B (Saatcioglu et al.) is 1.18 vs. 1.08, showing improved results for Case B. However, the result is opposite for the 15M and 20M series with average ratios of 0.93 vs. 0.89 and 0.86 vs. 0.8 for cases A and B, respectively. In general the ratios decrease with the Saatcioglu et al. model which follows the expected trend since this model predicts larger DIF factors, resulting in higher resistance curves. In the case of the HSFRC beams, the best results are also obtained using Case C which corresponds to the more conservative DIF values of the UFC-3-340-02. The $\delta_{anal}/\delta_{exp}$ ratios reduce for Case C vs. Case D and E which also follows the expected trend since Cases E and D use larger DIF values when compared to Case C. The results indicate the need for further research related to the prediction of DIF for HSFRC.

Table 8-13 Sensitivity analysis- Dynamic increase factors for plain concrete specimens

	Dynamic increase factors		
	Concrete in compression	Concrete in tension	Steel reinforcement
CASE-A	UFC-3-340-02 (2008) $DIF_c = 1.19$	$DIF_{ct} = 1.00$	UFC-3-340-02 (2008) $DIF_y = 1.17, DIF_u = 1.05$
CASE-B	Saatcioglu et al. (2011) $DIF_c = 1.30$	$DIF_{ct} = 1.00$	Saatcioglu et al. (2011) $DIF_y = 1.30, DIF_u = 1.10$

Table 8-14 Sensitivity analysis- dynamic increase factors for HSFRC specimens

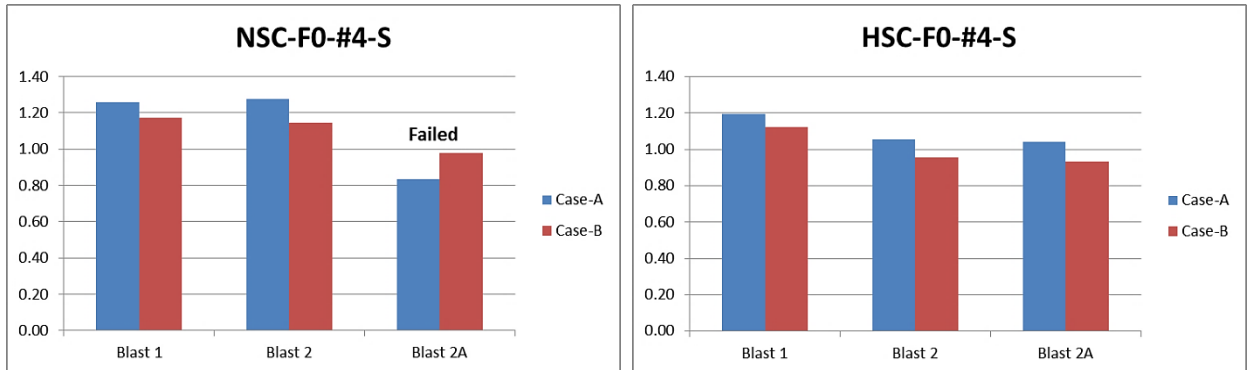
	Dynamic increase factors		
	Concrete in compression	Concrete in tension	Steel reinforcement
CASE-C	UFC-3-340-02 (2008) $DIF_c = 1.19$	$DIF_{ct} = 1.00$	UFC-3-340-02 (2008) $DIF_y = 1.17, DIF_u = 1.05$
CASE-D	Zhang & Mindness (2011) $DIF_c = 1.28$	$DIF_{ct} = 1.00$	Saatcioglu et al. (2011) $DIF_y = 1.30, DIF_u = 1.10$
CASE-E	Zhang & Mindness (2011) $DIF_c = 1.28$	Malvar & Ross (1998) $DIF_{ct} = 1.22$	Saatcioglu et al. (2011) $DIF_y = 1.30, DIF_u = 1.10$

Table 8-15 Sensitivity analysis – Analysis statistics for plain concrete specimens

Specimen ID	Mean of ($\delta_{anls}/\delta_{exp}$)	
	CASE-A	CASE-B
NSC-F0-#4-S	1.27	1.16
HSC-F0-#4-S	1.10	1.00
Average	1.18	1.08

Table 8-16 Sensitivity analysis – Analysis statistics for HSFRC specimens

Specimen ID	Mean of ($\delta_{anls}/\delta_{exp}$)		
	CASE-C	CASE-D	CASE-E
HSC-F1(ZP)-#4-0	0.90	0.89	0.83



a) Normal strength concrete

b) High strength concrete

Figure 8-23 Sensitivity analysis – DIF for specimens NSC-F0-#4-S and HSC-F0-#4-S

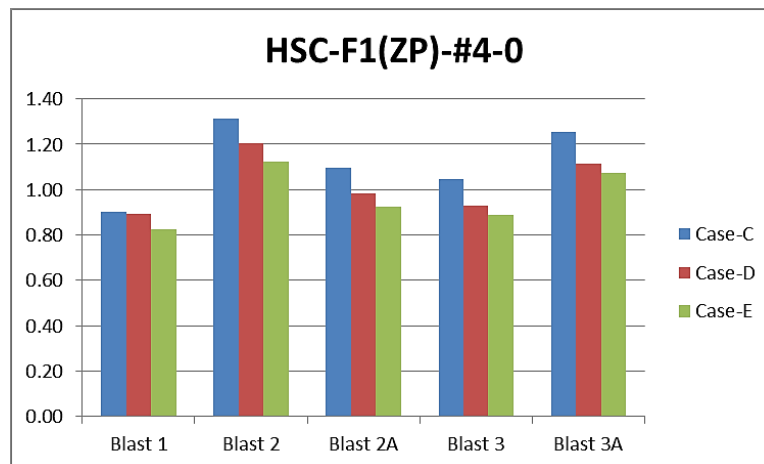


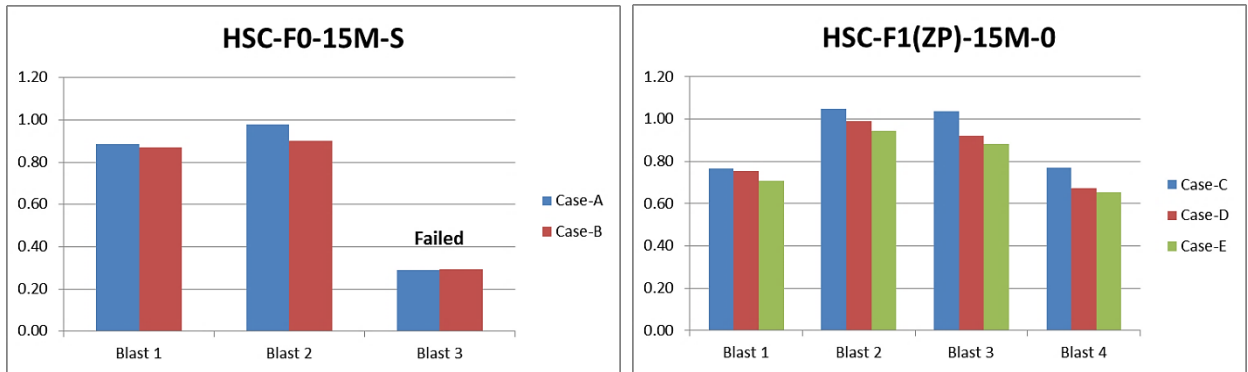
Figure 8-24 Sensitivity analysis – DIF for specimen HSC-F1.0(ZP)-#4-0

Table 8-17 Sensitivity analysis – Analysis statistics for plain concrete specimen

Specimen ID	Mean of ($\delta_{anls}/\delta_{exp}$)	
	CASE-A	CASE-B
HSC-F0-15M-S	0.93	0.89

Table 8-18 Sensitivity analysis – Analysis statistics for HSFRC specimen

Specimen ID	Mean of ($\delta_{anls}/\delta_{exp}$)		
	CASE-C	CASE-D	CASE-E
HSC-F1.0(ZP)-15M-0	0.90	0.84	0.80



a) Plain concrete specimen b) HSFRC specimen
Figure 8-25 Sensitivity analysis – DIF for plain concrete and HSFRC specimens

Table 8-19 Sensitivity analysis – Analysis statistics for series 20M

Specimen ID	Mean of ($\delta_{anls}/\delta_{exp}$)	
	CASE-A	CASE-B
HSC-F0-20M-S	0.86	0.80

Table 8-20 Sensitivity analysis – Analysis statistics for series 20M

Specimen ID	Mean of ($\delta_{anls}/\delta_{exp}$)		
	CASE-C	CASE-D	CASE-E
HSC-F1(ZP)-20M-0	0.90	0.83	0.79
HSC-F1(ZP)-20M-S	0.67	0.62	0.60
HSC-F0.5 (ZP)-20M-S	0.88	0.81	0.78
HSC-F0.5 (5D)-20M-S	0.81	0.75	0.72
Average	0.81	0.75	0.72

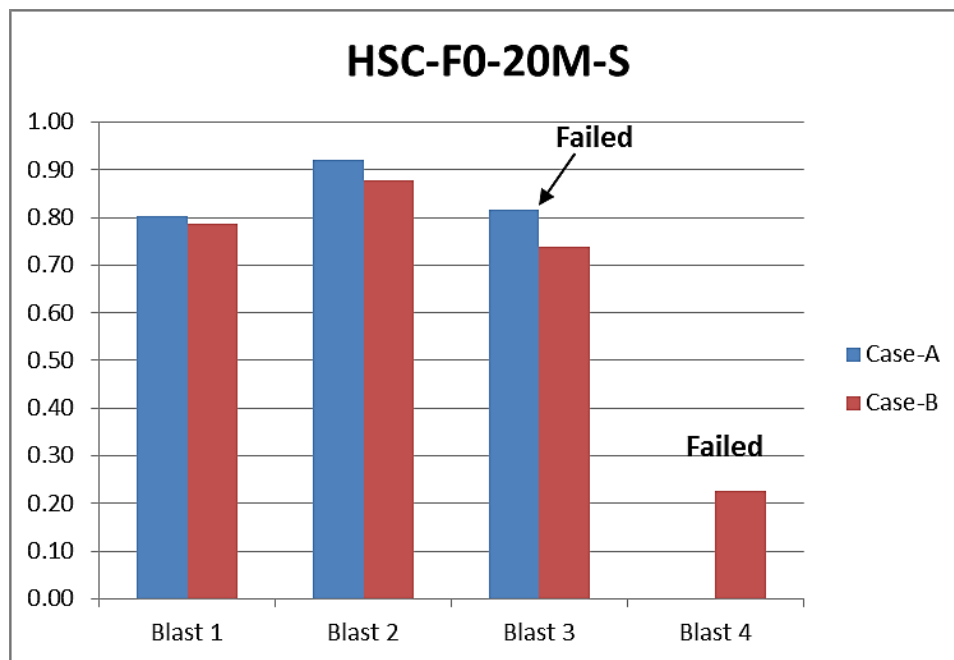


Figure 8-26 Sensitivity analysis – DIF for specimen HSC-F0-20M-S

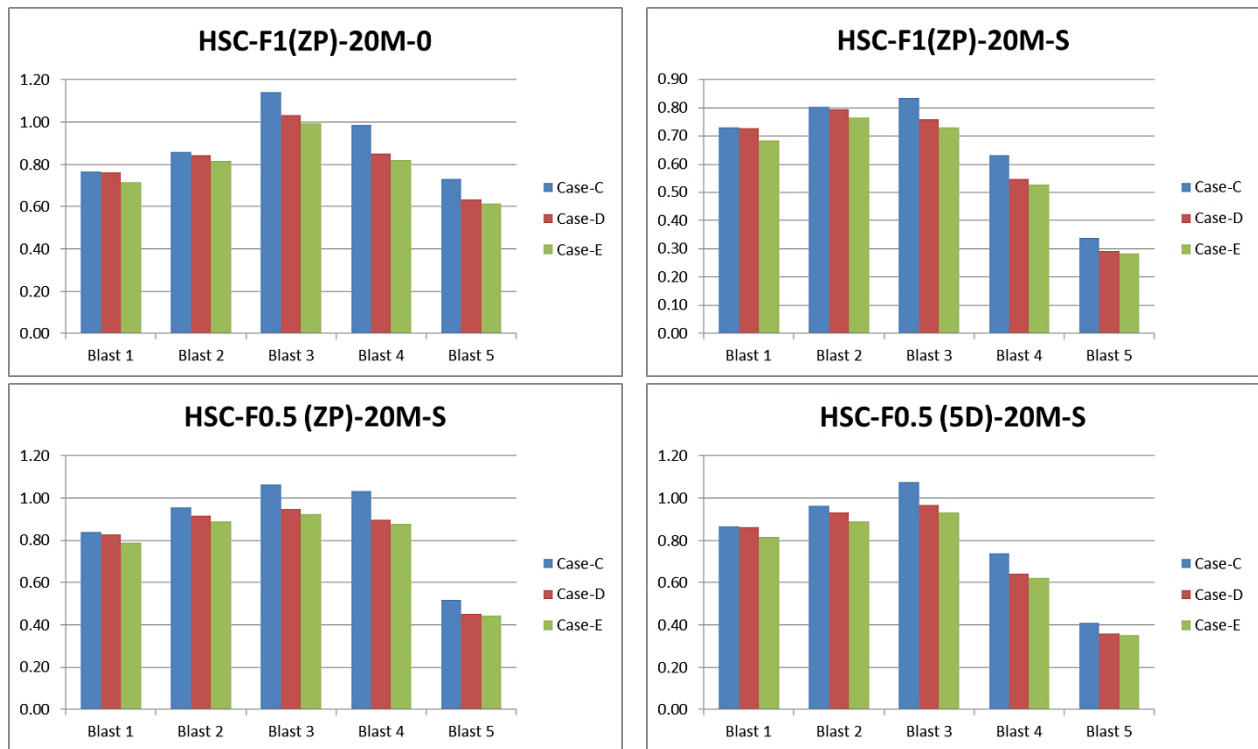


Figure 8-27 Sensitivity analysis – DIF for HSFRC specimens

Chapter 9: Conclusion & Recommendations

9.1 Conclusion

This thesis presented an experimental and analytical study examining the blast performance of high-strength concrete (HSC) and high-strength steel fibre-reinforced concrete (HSFRC) beams. Parameters investigated included the effects of concrete strength, steel reinforcement ratio, steel fibres, fibre content/type, as well as the presence of transverse reinforcement.

A total of twenty beams were tested, including nine beams under quasi-static (low-strain) four-point bending and eleven beams under dynamic (high-strain) blast loads using a shock-tube. For the dynamic tests, the influence of the parameters was investigated by studying various performance criteria including overall blast load capacity, control of maximum and residual mid-span displacements, failure mode, damage tolerance and fragmentation resistance. For the static tests, the performance of the beams was compared in terms of the test parameters on load resistance, ductility, toughness and failure mode. The conclusions from the research study can be summarized as follows:

- Increasing the steel reinforcement ratio greatly improved the blast response of the high-strength concrete beams under static and dynamic loading. Under blast loads maximum and residual mid-span displacements were reduced and the overall blast capacity of the HSC and HSFRC beams increased as the reinforcement ratio was increased. In terms of static loading, the use of larger reinforcement ratio increased maximum load capacity and the stiffness of the HSC beams, although ductility tended to reduce. In general the provision of fibres allowed for excellent ductility regardless of reinforcement ratio.
- The use of transverse reinforcement prevented shear failure from developing in the HSC beams under both static and dynamic blast loads, confirming the necessity of adding stirrups to reinforced concrete flexural members susceptible to shear failure.
- The use of high-strength fibre-reinforced concrete (HSFRC) resulted in improved control of maximum and residual mid-span displacements when compared to companion HSC beams without fibres. Moreover, the same benefits were observed in specimens subjected to static loading, including improved toughness and ductility.
- HSFRC with 1% fibres was capable of effectively replacing transverse reinforcement in beams subjected to both static and blast loads, preventing brittle shear failure and promoting ductile flexural response. The use of

fibres was also capable of improving damage tolerance at failure, with a better control of concrete crushing in the compression zone. Failure in these specimens was observed to be concrete splitting in the moment region with fibre pullout. This phenomenon was observed in both dynamic and static loading tests.

- Although improved blast performance was observed when combining 0.5% fibres with shear reinforcement, increasing the fibre content to 1% in beams with stirrups did not result in further enhancement in blast performance. However the use of 1% fibres in beams without stirrups did provide better control of displacements and improved damage tolerance when compared to the beam with 0.5% fibres.
- The results show that the use of shorter (ZP) fibres allowed for enhanced performance when compared to the longer (5D) fibres.
- The use of fibres significantly improved the damage tolerance and greatly reduced secondary blast fragments in the beams when compared to plain high-strength concrete.
- The behaviour of beams constructed with normal and high-strength concrete was similar under static and blast loads. However, the HSC beam showed increased strength and stiffness under static loads when compared to the companion beam constructed with NSC. Under blast loading, the HSC beam showed slightly reduced mid-span displacements when compared to the companion NSC beam.

Dynamic analysis was conducted to compute the displacement response of the tested beams analytically. This was done by defining material stress-strain models for plain concrete, fibre reinforced concrete incorporating high-strain effects and using a single degree of freedom (SDOF) model and lumped inelasticity approach. The conclusions from the analytical study can be summarized as follows:

- The SDOF method showed the ability of predicting the response of the tested specimens with acceptable accuracy. Some potential sources of error include the over-prediction of stiffness in the analytical resistance curves and the effect of accumulated damage (from repeated blasts) which was not considered in the analysis.
- In general the results were more accurate for the #4 and 15M series when compared to the 20M series.
- In terms of the HSFRC specimens the results were generally acceptable except for the specimens with combination of 1% ZP fibres and stirrups and 0.5% 5D fibres and stirrups, which can be related to the over-estimation of

the resistance of these HSFRC beams. Another source of error includes the effect of accumulated damage, particularly at later blasts (e.g. at Blast 4 / 5).

- The sensitivity analysis showed that the predictions were not greatly affected by the choice of concrete compression model. The choice of DIF model had more noticeable effect with the best results obtained using the design DIF values suggested in the UFC-3-340-02 (2008) documentation. The results also demonstrated the need for further research examining the effect of high-strain rates on the material response of HSFRC.

9.2 Recommendations for Future Research

The following recommendations for future studies are suggested in order to improve understanding of the behaviour of HSC and HSFRC under blast loads:

- Further experimental research on HSC and HSFRC structural components subjected to simulated blast loads (columns and slabs)
- Further testing on HSC and HSFRC beams subjected to uniformly distributed blast loads and with varying boundary conditions;
- The effect of steel fibre content and types on the blast response of HSFRC structural members requires further study to examine the most optimal fibre content and type for design purposes.
- The investigation of the effects of combining steel fibres and transverse reinforcement in HSFRC beams with greater shear demands under blast loading is recommended.
- The investigation of the effects of high strain rates on HSFRC response in tension would be beneficial for determining appropriate dynamic increase factors for use in the design of HSFRC structural members.
- Further analytical research on the blast modeling of HSC and HSFRC members (e.g. finite element modeling).

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