

EFFECT OF HIGH-PERFORMANCE CONCRETE AND STEEL MATERIALS ON THE BLAST PERFORMANCE OF REINFORCED CONCRETE ONE-WAY SLABS

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ABSTRACT

The mitigation of blast hazards on critical reinforced concrete structures has become a major concern in regards to the safety of people and the integrity of buildings. Recent terrorist incidents and accidental explosions have demonstrated the need to study the effects of such threats on structures in order to develop effective methods of reducing the overall impact of blast loads. With the arrival of innovative materials such as steel fibre reinforced concrete (SFRC), ultra-high performance fibre reinforced concrete (UHPFRC) and high strength steel reinforcement, research is required in order to successfully adapt these new materials in blast-resistant structures. Hence, the objective of this thesis is to conduct an experimental parametric study with the purpose of investigating the implementation of these innovative materials in reinforced concrete slabs and panels.

As part of the study, a total of fourteen one-way slab specimens with different combinations of concrete, steel fibres and steel reinforcement are tested under simulated blast loads using the University of Ottawa Shock-Tube Facility. The test program includes three slabs constructed with normal-strength concrete, five slabs constructed with SFRC and six slabs constructed with UHPFRC. Among these specimens, four are reinforced with high-performance steel reinforcement. The specimens are subjected to repeated blast loading with gradually increasing reflected pressure and reflected impulse until failure. The performance of the slabs is studied using various criteria such as failure load and mode, maximum and residual deflections, as well as tensile cracking, spalling and secondary fragmentation control.

The behaviour of all specimens is compared in different categories to determine the effects of concrete type, steel reinforcement type, steel fibre content and steel fibre type on blast performance.

As part of the analytical study the response of the slab specimens is predicted using dynamic inelastic single-degree-of-freedom (SDOF) analysis. The dynamic analysis is conducted by generating load-deformation resistance functions for the slabs incorporating dynamic material properties.

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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
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

ACRONYMS

Acronym	Definition
ACI	American Concrete Institute
ASTM	American Society of Testing and Materials
BLS	Blast loading simulator
CEB	Comité euro-international du béton (Euro-international Concrete Committee)
CFRP	Carbon fibre reinforced polymer
CRC	Compact reinforced composite
DIF	Dynamic increase factor
FRP	Fibre reinforced polymer
HL	Hairline (crack width)
HS	High strength (steel reinforcement)
HSC	High strength concrete
HSS	Hollow steel section
LTD	Load transfer device
LVDT	Linear variable displacement transducer
MMFX	Microcomposite multistructural formable steel
NBCC	National Building Code of Canada
NS	Normal strength (steel reinforcement)
NSC	Normal strength concrete
NSR	Normal strength reinforcement
RPC	Reactive powder concrete
SCC	Self-consolidating concrete
SDOF	Single-degree-of-freedom
SFRC	Steel fibre reinforced concrete
SHPB	Split Hopkinson Pressure Bar
SS	Stainless steel (reinforcement)
TNT	Trinitrotoluene
UFC	Unified Facilities Criteria
UHPFRC	Ultra-high performance fibre reinforced concrete
UHSC	Ultra-high strength concrete

CHAPTER 1. INTRODUCTION

1.1. GENERAL CONTEXT

In these past few decades, the mitigation of blast loads on critical reinforced concrete infrastructure has become a major concern in regards to the safety of people and the integrity of structures. Recent historical events including terrorist incidents, such as the World Trade Center bombing (1993) and the Oklahoma City bombing of the Alfred P. Murrah Federal Building (1995), and accidental explosions, such as the Toronto Sunrise propane explosion (2008) and the Lac-Mégantic rail disaster (2013), demonstrate the necessity of studying the effects of these potential threats on structures in order to develop effective methods of reducing the overall impact of blast loads.

The emergence of innovative materials such as steel fibre reinforced concrete (SFRC), ultra-high performance fibre reinforced concrete (UHPFRC) and high strength steel reinforcement bars offers new opportunities to improve the resistance of reinforced concrete structures to extreme dynamic loads. For instance, when compared to conventional normal strength concrete, UHPFRC shows a significant increase in compressive and tensile strength, as well as improvements to ductility, toughness and overall damage tolerance. Similarly, high performance steel reinforcement, with increased strength or ductility, can potentially be used improve the blast performance of concrete structures. However, only a limited number of studies and tests have been conducted on slabs and panels constructed with these materials. In order to study their effectiveness in blast mitigation, this research will attempt to provide additional data and observations in regards to the potential benefits of implementing these materials in reinforced concrete slabs.

1.2. RESEARCH OBJECTIVES

The objective of this research is to conduct an experimental parametric study in order to investigate the implementation of innovative materials in reinforced concrete slabs and panels. Specimens are to be subjected to simulated blast loads in order to observe the effects of various material parameters, such as concrete type, steel reinforcement type and steel fibre type and volumetric ratio, on the behaviour of these slabs. The benefits of implementing high performance materials against traditional materials will then be assessed through a comparison of the dynamic responses of the slabs.

1.3. SCOPE OF WORK

A total of fourteen one-way reinforced concrete slab specimens were designed to be tested at the University of Ottawa Shock-Tube Facility. The material parameters used in the design of these specimens include:

- The concrete mix; self-consolidating concrete (SCC) and compact reinforced composite (CRC), a proprietary UHPFRC.

- The steel reinforcement type; regular strength non-deformed and deformed bars, high strength, low carbon MMFX₂ bars, and high ductility stainless steel EnduraMet 32 (XM-28) bars.
- The steel fibres; a selection of two different types of steel fibres per concrete mix with varied fibre content, tensile strength and aspect ratios.

Overall, the test program includes three slabs constructed with normal-strength concrete (SCC), five slabs constructed with SFRC (SCC and fibers) and six slabs constructed with UHPFRC (CRC). Among these specimens, four are reinforced with high-performance steel reinforcement (high-strength or stainless steel).

The specimens are to be tested against repeated blast loads with increasing reflected pressure and reflected impulse until failure. Important performance criteria to be used to study the performance of the slabs includes: failure load magnitude and mode, maximum and residual mid-span displacements, tensile cracking control, and secondary fragmentation mitigation. These criteria will be used to compare the behaviour of each specimen against one another and to determine the benefits or disadvantages of high performance and traditional materials in this application. This research also includes an analytical phase which aims at predicting the response of the test slabs using dynamic inelastic single-degree-of-freedom (SDOF) modeling.

1.4. THESIS BREAKDOWN

This document contains seven chapters divided as listed below:

Chapter 1 - Introduction:

- Introduces the significance of this research, describes the various objectives and defines the scope of work.

Chapter 2 - Literature review:

- Summarizes previous research on the blast performance of reinforced concrete slabs and structural components built with high performance steel fibre reinforced concrete materials.

Chapter 3 - Experimental program:

- Defines the various specimens and material parameters.
- Describes the Shock-tube and components related to simulated blast load tests.
- Outlines the test setup and procedure.

Chapter 4 - Experimental results:

- Summarizes the data obtained from the experimental program.
- Describes the behaviour of specimens for each blast test.

Chapter 5 - Discussion:

- Summarizes various observations related to the effects of the concrete type, the steel reinforcement type, or the steel fibre type and content ratio on the blast performance of the slabs.
- Presents comparisons between traditional and innovative materials based on response and damage criteria.

Chapter 6 – Analysis & Models:

- Defines models in order to predict the blast response of each specimen configuration using a single-degree-of-freedom approach.
- Compare the various models and dynamic increase factors combinations to select accurate analytical models.
- Presents the resulting comparisons between experimental records and analytical predictions.

Chapter 7 – Conclusion:

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

CHAPTER 2. LITERATURE REVIEW

2.1. CHAPTER OVERVIEW

This chapter provides a review of existing researches pertaining to this thesis. This includes a summary of previous research on the blast performance of reinforced concrete slabs with regular and high performance materials, research on the blast behavior of ultra-high performance fibre reinforced concrete (UHPFRC) elements, and studies on the effects of high strain rates which are characteristic of impact and blast load responses.

2.2. PREVIOUS RESEARCH ON THE BLAST BEHAVIOUR OF REINFORCED CONCRETE SLABS

Several studies were conducted in the past regarding the blast behaviour of reinforced concrete elements. Methods of improving these components include casting with higher strength concrete or steel reinforcement, incorporating fibres to the concrete mix, retrofitting the specimens with fibre reinforced polymers or aluminum foams, and more. The researches summarized in the next few pages are but a fraction of these studies which directly involve slabs and panels.

T.S. Lok & J.R. Xiao (1999)

The purpose of this study was to investigate the behaviour of steel fibre reinforced concrete panels when subjected to blast loads. A total of seven live blast tests on 42 specimens were conducted using explosive charges of 8, 20, 30 and 40 kg distanced at 5.0 m from the support setups. Three types of specimen shapes were analyzed such as simply supported rectangular one-way panels, two-way square slabs and open-box reinforced concrete structures. Three different types of fibres and volume ratios of 0.5%, 1.0% and 1.5% were combined to a conventional reinforced concrete mix with aggregates. Certain panels also had wire mesh reinforcement.

It was observed that the failure pattern was identical for each type of specimen shape. One-way panels would show a single linear crack near mid-span, square slabs would exhibit diagonal cracks and box specimens had yield lines close to its boundaries. It was noted that the cracking patterns and the magnitude of damage varied along with the fibre content; however, higher fibre content would not mean a lesser amount of cracking. Figure 2-1 demonstrates some of the benefits of incorporating steel fibres in the design of reinforced concrete components. For a similar air blast, the specimen without steel fibres (WOB) was breached, significantly damaged and resulted in flying debris unlike the fibre reinforced specimen (FAWOB) which countered these issues.

The reflected impulses and displacements were compared with an SDOF analytical model for which the negative pressure was ignored, the structural resistance was based on an elastic-plastic relationship and a dynamic increase factor of 1.4 was used on static models. It was noted several instances that high fibre ratios might weaken the properties of the concrete panels as analytical

results under-estimate the response of the slab. The predicted results for simply-supported SFRC specimens with 0.5% and 1.0% were generally safe while the response for open-box specimens was highly over-estimated due to conservative calculations.

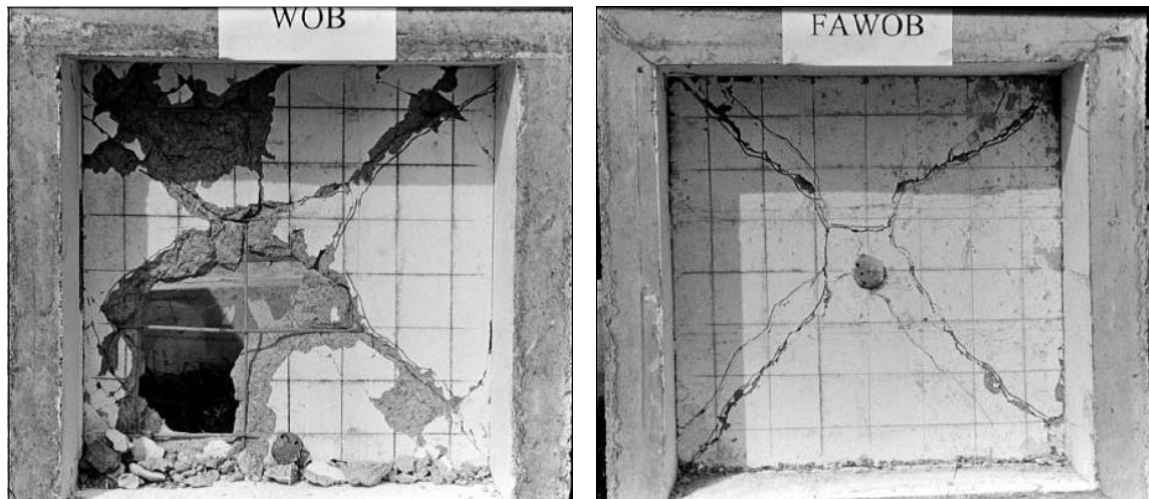


Figure 2-1 Test specimens without and with steel fibres (Lok & Xiao 1999)

A. Schenker et al. (2008)

The purpose of this study, which was conducted by the Protective Technologies Research & Development Center of the Faculty of Engineering Sciences of the Ben-Gurion University of the Negev in association with the Israel Defense Forces, was to observe the capability of externally applied aluminum foams to absorb energy and mitigate blast loads on reinforced concrete slabs. In addition, the live test data would be used to verify and validate their numerical model.

Two types of slabs were built for this experiment: B-30 which consisted of regular 30 MPa concrete, and B-100 which incorporated fibres as internal reinforcement and consisted of higher strength 100 MPa concrete. Four slabs with 3 m spans were constructed and half were protected by multiple layers of aluminum foam. It was described that the density and thickness of the foam are significant factors when absorbing energy. The specimens were tested against a live hemispherical blast from a 1000 kg TNT charge located at a stand-off distance of 20 m. Figure 2-2 is a photograph of the setup with two B-100 slabs; the specimen on the left was protected with aluminum foam.

The blast wave propagation and slab responses were modeled using Dytran and LS-Dyna however the specimen displayed a complex behaviour due to the lack of restraint from the support. From the experiment, it was observed that the displacement of the B-100 specimen was lesser than for the B-30 specimen due to the strength and composition of the slabs. Aluminum foams were found capable of reducing the acceleration and damage magnitude of the protected concrete panels. It was also noted that the foams were more effective on the regular strength specimen as it should be generally more sensitive to energy dissipation. Additional testing was recommended to further evaluate the efficiency and practicality of aluminum foams.

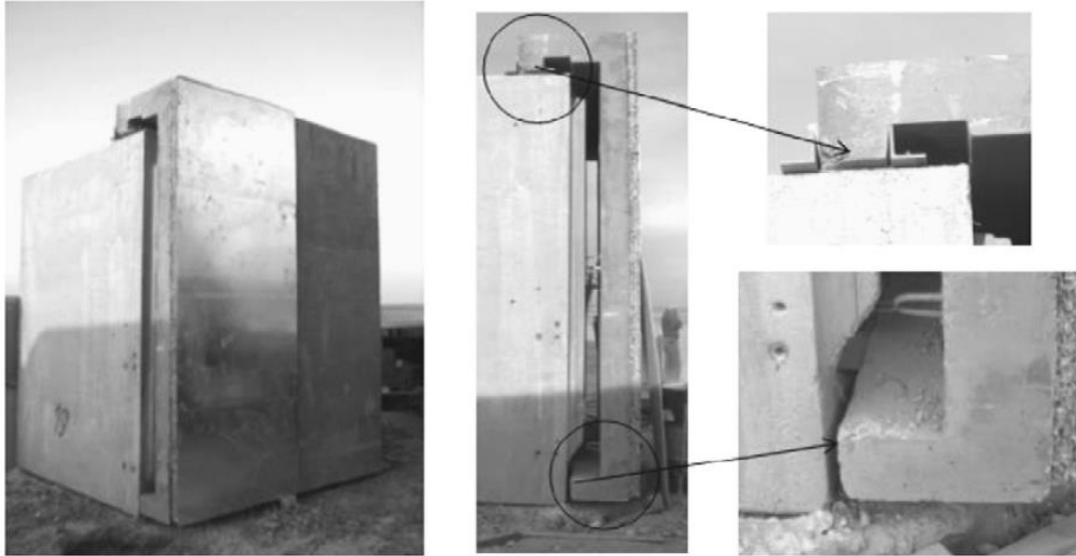


Figure 2-2 Two B-100 specimens mounted side-by-side (Schenker et al. 2008)

E. Jacques (2011)

The purpose of this research was to observe the performance benefits of retrofitting one-way and two-way reinforced concrete flexural members using externally bonded carbon fibre reinforced polymer (CFRP), to develop analytical models and propose design guidelines for this type of retrofit.

For this research, a total of thirteen simply-supported one-way and fully-fixed two-way reinforced concrete slabs were cast and distributed between five distinctive companion sets to be tested at the University of Ottawa Shock-Tube Facility. These specimens were cast with regular concrete covering either an area of 2440 mm x 440 mm or 2440 mm x 2440 mm, and their widths would vary between 80 mm or 120 mm. Reinforcement was composed of 6.3 mm non-deformed steel bars. In addition, most of the specimens were retrofitted using CFRP.

The experimental program consisted of sixty simulated far-field blast loads; specimens would be exposed to progressively increasing pressures and impulses leading to elastic and inelastic behaviours. A single-degree-of-freedom (SDOF) model using a lumped inelasticity approach was developed to compare analytical displacements with the experimental responses. This method was implemented into the software RCBlasT (Jacques 2014).

It was concluded that the use of fibre reinforced polymer (FRP) would increase the strength and stiffness of flexural members, significantly reducing the response of slabs under blast loads. The failure mode was altered from flexure to flexure-shear debonding; this could be delayed through the usage of FRP anchors. Finally, it was recommended not to use dynamic increase factors to CFRP systems as no apparent effect could be observed from the application of high strain rates during dynamic responses.

G. Thiagarajan et al. (2014)

The purpose of this research was to observe numerically and experimentally the response and behaviour of high strength concrete (HSC) and normal strength concrete (NSC) doubly reinforced slabs with low alloy vanadium reinforcement and normal strength reinforcement (NSR). More specifically, to study the possibility of using high-performance materials as a substitute of traditional materials when dealing with blast loads.

A total of four slabs, each a different combination of the aforementioned concrete and reinforcement available, were tested with the Blast Loading Simulator (BLS) at the U.S. Army Corps of Engineers – Engineer Research and Development Center in Vicksburg, MS. The slabs dimensions were 1652 mm x 857 mm x 101.6 mm. Finite element modelling was conducted using LS-DYNA and two concrete models.

It was noted that using HSC instead of NSC for normal strength reinforcement greatly reduced the deflections by 37%. Substituting NSR with high strength low alloy vanadium in NSC slabs also reduced the deflections by 45%. However, for HSC, the change in reinforcement only affected the response by 12%. Due to the limited number of specimens tested, the advantages perceived in this research for high-performance materials were to be taken only as observations.

2.3. PREVIOUS RESEARCH ON THE BLAST BEHAVIOUR OF ULTRA-HIGH PERFORMANCE FIBRE REINFORCED CONCRETE PANELS

Several researches related to impact testing have demonstrated that ultra-high performance fibre reinforced concrete (UHPFRC) displays an enhanced performance against high-strain rates when compared to conventional materials. These studies have been conducted through drop-weight and drop hammer testing, as well as direct tension and Split Hopkinson Pressure Bar (SHPB). However, due to UHPFRC being a relatively new innovative material for construction, there are only a limited number of studies available on the subject of UHPFRC in blast research. A majority of the studies that can be found involve live explosives either in direct contact with specimens, or tested in near or far-ranges. Samples of the researches applicable to UHPFRC slabs and panels are listed in this section in chronological order.

B. Cavill, M. Rebentrost & V. Perry (2006)

This study was conducted in order to evaluate the performance of Ductal[®] against blast and impact loading. This reactive powder concrete (RPC) was developed by Bouygues and has properties such as high compressive strength (160-200 MPa), high tensile strength (30-40 MPa), and improved flexural toughness compared to traditional concrete.

Far-range blast tests were completed in 2004, in Woomera, South Australia. Panels had an area of 2.0 m x 1.0 m and a thickness varying between 50, 75 and 100 mm. A total of seven panels were designed with Ductal[®], five of which had high strength pre-stressed reinforcement while the other two had no reinforcement. A control panel was also designed as a conventional

reinforced concrete slab. Finally, the blasts consisted of a six tonnes of TNT equivalent charge located at either 30, 40 or 50 m from the specimens, such as seen in Figure 2-3. From these tests, it was observed that the Ductal[®] panels showed improved ductility and no signs of fragmentation. It was also noted that the pre-stressed panels were able to absorb the energy from the blast without fractures or residual deformations.



Figure 2-3 Woomera blast trials (Cavill et al. 2006)

Other tests with positive outcomes were conducted on this material such as close-range charge tests, impact tests using a Split Hopkinson Pressure Bar; fragment simulated projectile loading, and ballistic tests. It was concluded that this material had the ability of resisting blast and impact tests with a smaller thickness than conventional concrete and with a lower risk of fragmentation.

T. Ngo et al. (2007)

This study investigated the blast performance of ultra-high strength concrete (UHSC) panels constructed with reactive powder concrete (RPC). Three concrete panels were designed with pre-stressed reinforcement, RPC and a thickness of either 75 or 100 mm. A fourth panel with regular reinforcement, normal strength concrete (NSC) and a thickness of 100 mm was built as a control specimen. The dimensions of the panels face were 2 m x 1 m. The compressive strength of RPC was evaluated at 164.2 MPa and its tensile strength at 22.28 MPa, while the compressive strength of NSC was tested at 39.8 MPa.

Live tests were conducted in Woomera, South Australia. Specimens were exposed to explosives equivalent to six tonnes of TNT at a stand-off distance of either 30 or 40 m. The thicker UHSC panels had no permanent deformation and only minor damage such as hairline cracks, while the thinner UHSC had a small permanent inward deformation and wider tensile cracks. The control specimen failed from severe damage and the concrete was breached. It was concluded that the ultra-high strength pre-stressed panels were capable of resisting the high deflections due to dynamic blast loads while eliminating the risk of injuries from fragmentation.

In addition, a high strain rate model was tested and a non-linear dynamic analysis with the finite-element model software RC-IMPULSIVE was conducted to obtain accurate comparative results with the experiment.

Wu, C. et al. (2009)

This study's purpose was to investigate the blast performance of different slab configurations under near-range live blast tests. A total of six specimens were designed for this experiment. Two control specimens were designed with 39.5 MPa concrete and two way steel reinforcement. Two more normal strength concrete specimens were retrofitted with fibre reinforced polymer (FRP) on the compression face. Two slabs were also cast using ultra-high performance fibre concrete (UHPFRC) with 151.6 MPa in compressive strength and 30.2 MPa in tensile strength. One of the UHPFRC slabs was designed with steel reinforcement similar to the previously mentioned conventional slabs while the second one was prepared without steel reinforcement. The dimensions of the specimens were 2000 mm x 2000 mm x 100 mm and they were installed on the fixture depicted in Figure 2-4, assuming idealized simply supported conditions.

A total of eight blast tests were conducted on the specimens with explosives varying from 1 to 20 kg of TNT equivalent materials. The slabs were tested at near ranges with standoff distances between 0.75 m and 3 m. The blast load was noted to be non-uniform due to the type of explosive used for testing. A single-degree-of-freedom analysis was also conducted to compare the results.

From the experiment, it was observed that the plain UHPFRC would demonstrate a lower amount of damage than the conventional reinforced concrete slabs and that the reinforced UHPFRC slab was greatly superior to the other slabs tested in this study. It was also noted that the externally bonded FRP improved the blast performance of slabs however it couldn't be quantified in this research.

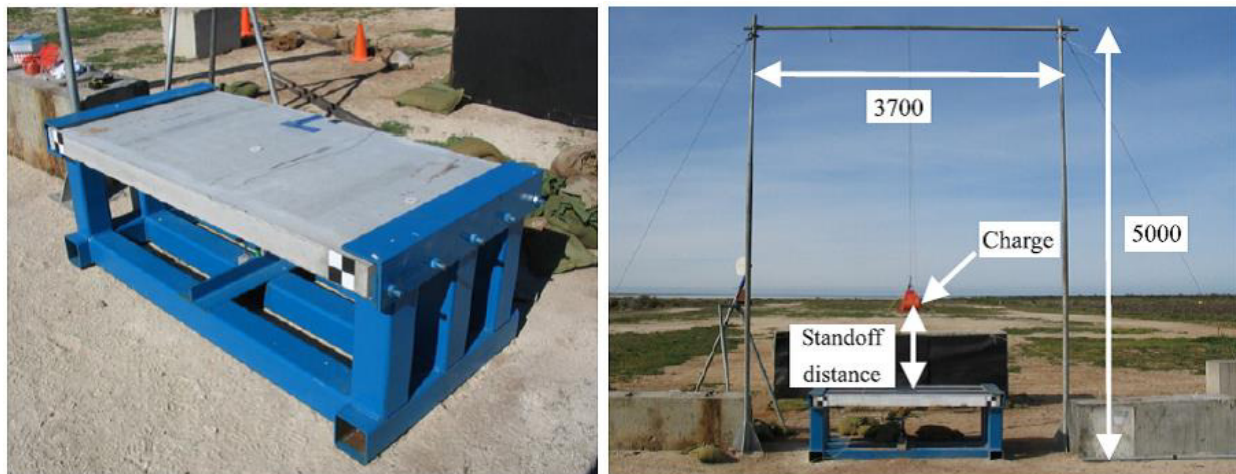


Figure 2-4 Test fixture and charge support frame (Wu et al. 2009)

S.J. Barnett et al. (2010)

The purpose of this research was to investigate the potential resistance of ultra-high performance fibre reinforced concrete (UHPFRC) against impact and blast loads in order to apply this material to anti-terrorism solutions.

Live blast tests were conducted in the UK on a series of four full-scale UHPFRC panels built with dimensions of 3.5 m x 1.3 m x 100 mm, specified with different fibres and steel reinforcement ratios. These specimens were tested at near and far ranges using 100kg TNT-equivalent explosions distanced at appropriate stand-off distances. These distances were obtained from single-degree-of-freedom models and Autodyn. Two identical slabs with 2% straight fibre content and steel reinforcement were placed at different stand-off distances, 7 m and 9 m. Two additional non-reinforced UHPFRC slabs with varied fibre content were placed at 12 m from the blast.

The closer panels both survived the blast with permanent deflections and minor cracks while the further non-reinforced slabs deflected and kept these deformations along with slab-deep horizontal cracks. The weakest of the non-reinforced slabs had the lesser fibre content ratio. It was also stated that no debris were observed during the test. It was concluded that UHPFRC had adequate properties to resist explosions but the detailing of the panel in terms of fibre content and reinforcement steel had major effects on its performance.

N.-H. Yi et al. (2012)

The purpose of this study was to investigate the capacity of ultra-high strength concrete (UHSC) and reactive powder concrete (RPC) to resist blast loads. For this experiment, the compressive strengths of normal strength concrete (NSC), UHSC and RPC were measured as 25.6 MPa, 202.1 MPa and 202.9 MPa respectively, while the split tensile strengths were found to be 2.2 MPa, 9.2 MPa and 21.4 MPa.

Live tests were conducted at the Agency for Defense Development of Korea test site, located near the Military Demarcation Line. Specimens were designed with a 1000 mm x 1000 mm area and a thickness of 150 mm and clamped to a buried steel frame in order to limit uplifting and blast wave reflection (Figure 2-5). Reinforcement in NSC and UHSC panels was composed of two layers of D10 mesh reinforcement spaced at 82 mm. Short steel fibres were incorporated to the RPC mix in a 2% volume ratio. Preliminary tests determined that the experiment would be conducted using 15.88 kg TT charges at 1.5 m from the specimens.

The UHSC and RPC panels generally had less cracking than the NSC but still failed in a quasi-brittle manner due to the material composition. They are however capable of resisting greater loads than the NSC panel; this was based from displacement, strain and acceleration measurements. The steel fibres from the RPC specimen increased the tensile strength as well as

controlled the spread of macro-cracks during blast. Finally, steel reinforcement and fibres were found to increase ductility and counter the brittle nature of the high strength materials.

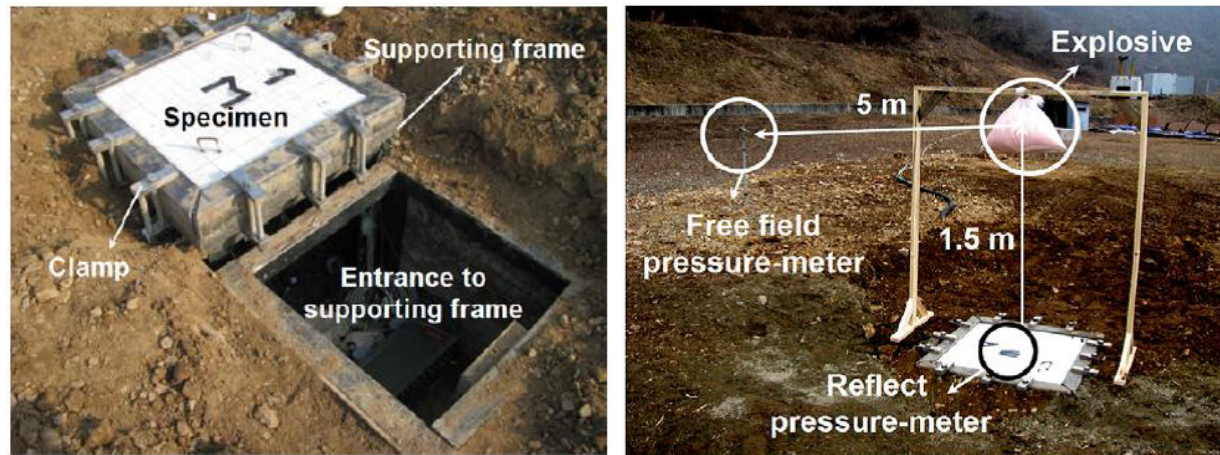


Figure 2-5 Support frame and test setup (Yi et al. 2012)

L. Mao et al. (2013)

This research's purpose was to conduct a numerical investigation of the blast performance of ultra-high performance fibre reinforced concrete (UHPFRC). Previous attempts at predicting the response of UHPFRC under blast loading using single-degree-of-freedom (SDOF) models, Autodyn or LS-Dyna were unsuccessful. This was attributed to the UHPFRC stress-strain model as its behaviour is still not fully understood, especially for the softening phase, which involves the effects of steel fibres, and the effects of high strain rates. Hence, this numerical study was an attempt at modelling UHPFRC panels using the non-linear finite element software LS-DYNA and a modified Karagozian and Case (K&C) concrete model which would account for these effects. Dynamic increase factors followed the CEB models.

This study reused data from Barnett et al. (2010). Full scale blast tests were conducted at GL Industrial Services, Spadeacam, Cumbria, England. Four panels were built with dimensions of 3.5 m x 1.3 m x 100 mm, specified with different fibres and steel reinforcement ratios. These specimens were tested at near and far ranges using 100kg TNT-equivalent explosions distanced at appropriate stand-off distances. During testing, it was noted that the blast loading was non-uniform due to the ground reflection of the blast wave leading to non-symmetrical panel fractures.

With revised models, it was possible to accurately predict peak blast pressure and impulse as well as obtaining maximum and residual displacements within a reasonable range of the experimental values. It was suggested that the fibre reinforced concrete model could be improved to further account the crack restraining effects of fibres.

B.D. Ellis et al. (2014)

The goal of this study was to conduct an experimental investigation of the reflected impulse limits of un-reinforced ultra-high performance fibre reinforced concrete (UHPFRC) panels and to develop a multiscale model based on the properties and mesostructure of the materials.

Four panels were designed with dimensions of 1626 mm x 864 mm x 50.4 mm. The UHPFRC mix was tested for an average compressive strength of 201 MPa. Steel fibres were included in the mix; these were 14 mm long, 0.185 mm in diameter, and had a tensile strength of 2160 MPa. No steel longitudinal reinforcement was used and one specimen had five holes drilled at symmetric locations to observe the effects of these openings on blast performance. The fibres were mixed in the concrete for a 2% volume ratio. Tests were conducted using a blast load simulator (BLS) at the U.S. Army Corps of Engineers' Engineer Research and Development Center, in Vicksburg, MS. The boundary conditions were similar to a simply supported flexural member.

Most specimens were tested once for fracture except panel #3 which survived two tests. Negligible debris resulted from the fractures and it was found that the critical impulse for the UHPFRC panels was situated between 0.97 and 1.47 MPa-ms. The multiscale model from this study was validated using experimental data including the critical specific impulse, fracture patterns and mid-span displacements. Following this validation, the model was used for a parametric study which deemed that fibres had an important role in increasing energy dissipation; factors would include the aspect ratio, the packing and the volume ratio of the fibres.

A. Stolz et al. (2014)

The purpose of this study was to investigate the performance of ductile concrete plates when subjected to far-field blast loads. Materials included in this research include DUCON[®] ductile concrete, a self-compacting high performance concrete reinforced with Micromat[®], a micro reinforcement mesh. The reinforcement mesh serves as an alternative to randomly distributed steel fibres.

Three configurations of ductile concrete plates were investigated with varied plate thickness, mesh size and reinforcement volume. A static four-point bending test was first completed to obtain properties such as the tensile bending strength, the ductility ratio and the cracking patterns of each configuration. Dynamic testing was conducted using the Shock-tube of the Fraunhofer EMI in Freiburg. The plate specimens covered an area of 2.41 m by 1.14 m with a thickness of either 60 or 100 mm. These were installed on single point bearing supports for a clear span of 2.31 m. Specimens were tested against loading levels from 35 kPa up to 250 kPa in order to determine the elastic range of each configuration, the cracking pattern and the ultimate bending deformation.

A single-degree-of-freedom model was defined to produce a dynamic analysis of the plate configuration and derive iso-damage curves, also known as pressure-impulse diagrams. The performance of ductile concrete was then assessed using non-dimensional iso-damage curves in comparison with previously tested normal strength reinforced concrete and ultra-high performance fibre reinforced concrete. While it was observed that the ductile concrete was an improvement over both, it was noted that the UHPFRC specimen had a smaller reinforcement ratio than the DUCON configuration.

2.4. PREVIOUS RESEARCH ON THE EFFECTS OF HIGH STRAIN RATES

Depending on the type of load, materials are subjected to different magnitudes of strain rates such as shown in Figure 2-6. When analyzing the behaviour of materials under impact or blast loads, it has been observed that the high strain rates occurring from these dynamic loads influence the capacity of materials to resist the said loads.

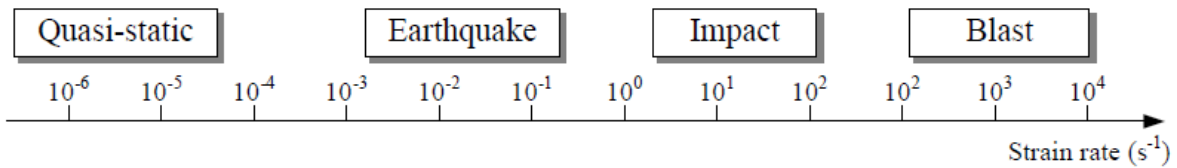


Figure 2-6 Strain rates in function of the loading type (Ngo et al. 2007)

Indeed, a dynamic increase factor (DIF), generally estimated from the strain rate, can increase the compressive and tensile strength of both concrete and steel components. Few models are well-established regarding this behaviour. These include the *Comité euro-international du béton* (CEB) Model Code 90 (1990) and the U.S. Department of Defense's United Facilities Criteria UFC 03-340-02 (2008). It should be noted that the UFC model is mostly used for the design of blast-resistant structures and may be more conservative as it proposes dynamic increase factors without taking into account the variation of the strain rate. The typical stress-strain curves shown in Figure 2-7 provide an overview of the effects of high strain rates on the behaviour of concrete and steel reinforcement components.

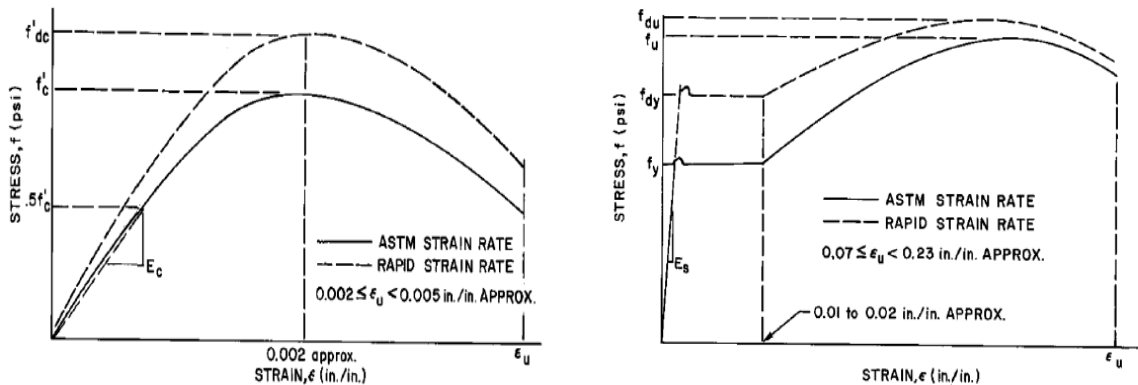


Figure 2-7 Typical stress-strain models for quasi-static and dynamic strain rates (UFC 03-340-02 2008)

To further improve the understanding of materials under dynamic loads, several researchers have conducted studies on the effects of high strain rates and the importance of dynamic increase factors.

L.J. Malvar, and J.E. Crawford (1998)

This study's purpose was to review existing high strain rate data in order to revise the dynamic increase factor concrete model recommended by the CEB. For concrete in tension, similar trends could be found in data with strain rates above 1s^{-1} ; a change of slope could be observed around this particular strain rate instead of the CEB suggested 30s^{-1} . Based on the DIF versus strain rate relationship initially proposed by Ross (1992), the tensile DIF equations were modified to accommodate lower and higher strain rates with a pivot point at 1s^{-1} and a static strain rate value of 10^{-6}s^{-1} ; under this strain rate, no increases in concrete tensile strength could be observed.

The equations for this particular study were included in Chapter 6 of this thesis under equations (6-34) and (6-35).

L. Zhang & S. Mindness (2011)

This study was conducted in order to investigate the behaviour of steel fibre reinforced concrete (SFRC) under compressive impact loads. Various specimens with different compressive strength, including 50 MPa, 90 MPa and 110 MPa, were tested using static tests, instrumented drop weight impact machines and a Split Hopkinson Pressure Bar. The data obtained from the impact tests were then compared to the CEB model. This model was considered to be inaccurate to determine dynamic increase factors for high performance concrete and a new model was proposed: the RCM model. Based on the CEB model, it also suggested a new turning point between both equations which would vary in function of the concrete strength. The RCM model was evaluated against Zhang's (2011) drop weight DIF data and was found to be fairly accurate with a variance between -3% and 19%.

The equations for this study were included in Chapter 6 under equations (6-32) and (6-33).

L.J. Malvar (1998)

The purpose of this study was to review existing data on the static and dynamic properties of ASTM graded steel rebar in order to propose new expressions to define the dynamic increase factor in function of the strain rate and the yielding stress.

Through this research, it was observed that the dynamic increase factor for a similar strain rate would be greater at yield than at the ultimate stress. It was also noted that the increase due to high strain rates was less significant as the steel reinforcement reaches higher grades. Finally, the dynamic increase factor would follow a linear function when plotted against the logarithm of the strain rate.

Based on these observations, a formula was proposed for steel reinforcement with yield strengths between 290 and 710 MPa and strain rates between 10^{-4} and $10s^{-1}$. The expression by Saatcioglu et al. (2011) was however preferred for the present thesis.

M. Saatcioglu et al. (2011)

This project was initiated due to the lack of design clauses for blast loads in the National Building Code of Canada (NBCC 2010). This research would include but not limit itself to the development of concrete, steel and timber models incorporating the effects of high strain rates such as dynamic increase factors. Following an extensive review of existing models and data sets, multiple material models were proposed by Saatcioglu et al. (2011).

The concrete and steel models were verified by testing eighteen concrete column specimens at the University of Ottawa Shock-Tube Facility. The experimental displacements were compared with analytical values from a single-degree-of-freedom (SDOF) model; it was capable of predicting with reasonable accuracy the response of the columns.

The dynamic increase factor expressions for Grade 400 steel were included in Chapter 6 of this thesis under equation (6-36). It was also reported that there was approximately a 15% difference between concrete cylinder strength and unconfined concrete strength in structural elements due to the size, shape and casting method.

2.5. SUMMARY OF LITERATURE REVIEW

A few observations can be extracted from the previous studies contained within this literature review. Several alternatives were considered in attempt to improve the blast performance of reinforced concrete slabs. Such approaches described in the studies from Section 2.2 include:

- applying externally bonded FRP on the tension or compression faces of the panels;
- protecting the concrete slabs with aluminum foams;
- replacing conventional concrete with high strength concrete (HSC);
- replacing conventional steel reinforcement with high strength low alloy vanadium bars;
- adding of steel fibres in the concrete mix;

Only a limited number of studies have examined the effect of steel fibers on the blast performance of conventional concrete slabs and further data is needed, in particular in the case of SFRC slabs tested at far range. Furthermore the performance of slabs reinforced with high-performance steels warrants further study. It is also noted that there is no data in the literature on the performance of slabs constructed with SFRC and high-performance steel reinforcement.

Most studies on the effect of ultra-high performance fibre reinforced concrete on the blast performance of concrete panels were conducted using live blast tests at close ranges. From these studies, it was noted that:

- steel fibres had a significant impact on the ductility and energy-dissipation properties of the concrete materials; the effects would be subjective to the aspect ratio, the packing and the volume ratio of the fibres;
- UHPFRC panels demonstrated lower amounts of damage than their counterparts, significantly reducing peak and residual displacements as well as negating the propagation of debris.

Test data on slabs tested at far range or using blast load simulations is limited as shown in Section 2.3. Some specimens were tested without reinforcement or with pre-stressed reinforcement; however, none were listed to have high strength or high ductility steel reinforcement bars and this topic requires further investigation. The behaviour of specimens was analyzed using either single-degree-of-freedom modelling or complex finite element modelling; however, there is a further need to examine the effect of various modelling parameters such as influence of material models and dynamic increase factors.

CHAPTER 3. EXPERIMENTAL PROGRAM

3.1. CHAPTER OVERVIEW

This research study examines the blast performance of reinforced concrete slabs constructed with high performance concrete and steel materials. The experimental program includes fourteen one-way slab specimens tested under simulated blast loads using the University of Ottawa Shock-tube Facility. The performance of the slabs is studied by varying parameters such as the type of concrete mix, the type and volume ratio of steel fibres, the type of steel reinforcing bars and the depth of the slab.

This chapter summarizes the details of the experimental program and provides information related to specimen descriptions, material parameters, specimen preparation, the description of the experimental setup with its related equipment, and explanations pertaining to the experimental procedure.

3.2. SPECIMEN SPECIFICATIONS

A total of fourteen one-way reinforced concrete slab specimens were designed for this research. The test program includes three specimens constructed with normal-strength self-consolidating concrete (SCC), five slabs constructed with steel fibre reinforced concrete (SFRC) and six slabs constructed with ultra-high performance fibre reinforced concrete (UHPFRC). Among these specimens, four are built with high-performance steel reinforcement.

Each specimen was constructed with a height of 2440 mm, a width of 440 mm, and a depth of either 80 mm or 100 mm. Longitudinal steel reinforcement was composed of four top and four bottom bars with 180° hooks, spaced at approximately 125 mm from each other. Clear cover was restricted to 6 mm on the tension and compression side. The specified dimensions are summarized in Figure 3-1.

These specimens can be divided into three categories in function of their concrete mix and fibre ratio: the CONTROL series, the SFRC series and the UHPFRC series. A summary of their composition can be found in Table 3-1 including their series, specimen ID, concrete mix type, fiber type and volume ratio (v_f), steel rebar type and slab depth. Nomenclature for the specimen ID follows this simple logic:

CONCRETE TYPE – FIBER % and TYPE – STEEL REBAR SIZE and TYPE – DEPTH

For example, SCC-1.2%ZP-#3NS-100 would be the specimen ID for a 100 mm deep SFRC slab cast with self-consolidating concrete, ZP fibres at a volumetric ratio of 1.2% and #3 U.S. size normal strength (NS) rebar. The HS or SS in the specimen nomenclature designates the use of high strength (HS) steel or high ductility stainless steel (SS) reinforcement in these slabs.

Table 3-1 Specimen matrix

Series	Specimen ID	Concrete Type	Fibre Type	v_f (%)	Steel Type	Depth (mm)
CONTROL Series	SCC-0%-6.3NS-80	SCC	None	0.0%	NS	80
	SCC-0%-#3NS-100				HS	100
	SCC-0%-#3HS-100					
SFRC Series	SCC-1.2%ZP-6.3NS-80		ZP	1.2%	NS	80
	SCC-0.6%BP-#3NS-100		BP	0.6%		100
	SCC-0.6%ZP-#3NS-100		ZP			
	SCC-1.2%ZP-#3NS-100					
	SCC-1.2%ZP-#3HS-100		HS			
UHPFRC Series	CRC-2%B-6.3NS-80		CRC	B	2.0%	NS
	CRC-2%B-#3NS-80	A		HS		100
	CRC-2%B-#3NS-100					
	CRC-2%A-#3NS-100					
	CRC-2%A-#3HS-100					
	CRC-2%A-#3SS-100					

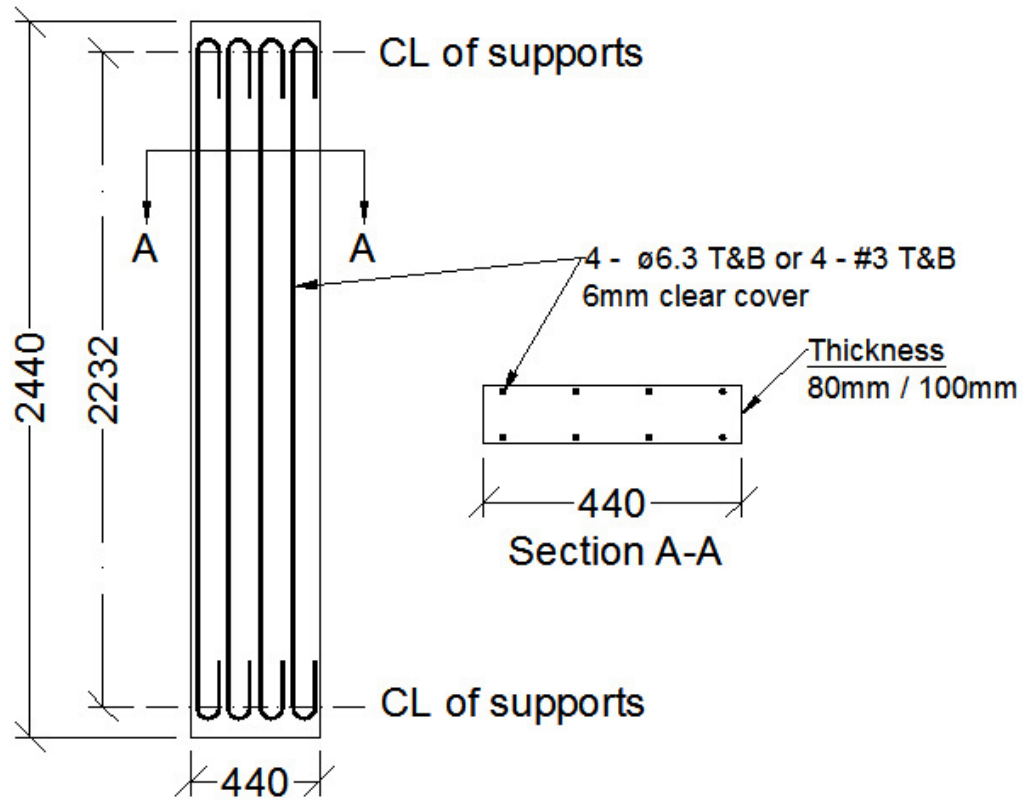


Figure 3-1 Specimen dimensions

3.2.1. CONTROL SERIES

The control series serves as a reference point when comparing results between different specimens. A total of three specimens were included in this group: SCC-0%-6.3NS-80, SCC-0%-#3NS-100 and SCC-0%-#3HS-100. The common criteria for these specimens were that they are all composed of self-consolidating concrete, also referred here as normal strength concrete, and that no steel fibres were added to their mix.

SCC-0%-6.3NS-80 was designed to be the control specimen for the thinner specimens and is composed of 6.3 mm non-deformed steel bars as reinforcement. SCC-0%-#3NS-100 was the main control specimen with which every specimen can be compared; it had regular #3 sized deformed steel bars as reinforcement and a depth of 100 mm. SCC-0%-#3HS-100 was a secondary control specimen, built to observe improvements associated with the use of high strength steel reinforcement in conventional reinforced concrete slabs.

3.2.2. SFRC SERIES

The Steel Fibre Reinforced Concrete (SFRC) series consists of five normal strength concrete slabs with structural steel fibres. In this research, the combined use of SCC and fibres is selected to improve the workability of SFRC. This series includes the following specimens: SCC-1.2%ZP-6.3NS-80, SCC-0.6%ZP-#3NS-100, SCC-1.2%ZP-#3NS-100, SCC-0.6%BP-#3NS-100 and SCC-1.2%ZP-#3HS-100. The fibre types used for these slabs include ZP305 (ZP) and BP80/30 (BP) deformed fibres. The fibre content varies between 0.6% and 1.2% of the total volume of concrete. One slab in this series is built with high-strength steel reinforcement.

3.2.3. UHPFRC SERIES

The Ultra High Performance Fibre Reinforced Concrete (UHPFRC) series consists of six high strength fibre reinforced concrete slabs built with *compact reinforced composite* (CRC). This series includes the following specimens: CRC-2%B-6.3NS-80, CRC-2%B-#3NS-80, CRC-2%B-#3NS-100, CRC-2%A-#3NS-100, CRC-2%A-#3HS-100 and CRC-2%A-#3SS-100. The fibre types used for these slabs include OL13/0.20 (A) and BEL M-0.30/13-3150 (B). The fibre content was fixed at 2% of the total volume of concrete. Two slabs in this series are built with high-performance steel reinforcement (one with HS and one with SS).

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 considered for this research; *self-consolidating concrete* (SCC) by KING Packaged Materials Company, and *compact reinforced composite* (CRC) imported from CRC Technology in Denmark.

Eight of fourteen specimens were cast using self-consolidating concrete, including the three control specimens. Specified for a nominal compressive strength of 40 MPa, the pre-packaged

mix “KING SCC” provided high workability even with the addition of steel fibres for SFRC slabs. Properties provided by the manufacturer for this mix are summarized in Table 3-2. The maximum aggregate size was specified as 10 mm. Admixtures were incorporated in the mix in the form of dry powder including an air-entrained admixture, superplasticizer and a viscosity modifying admixture.

The other six specimens were cast using a compact reinforced composite, a technology that was first developed in 1986 in the Cement & Concrete Laboratories of Aalborg Portland. “CRC Joint Cast” is a UHPFRC pre-packaged mix developed by CRC Technology capable of reaching high compressive strengths of over 150 MPa. With a maximum aggregate size of 4 mm, this mix is exempt of coarse aggregates. It also includes a binder with about 20-25% microsilica, mixed with Portland cement and superplasticizer. Steel fibres are typically added in a 2-6% ratio for ductility and the water to binder ratio ranges from 0.16-0.20. An example of the content of the CRC mix for a 2% steel fibre ratio is described in Table 3-3.

Table 3-2 KING SCC properties

Component	Content
HSF Cement	500 kg/m ³ of concrete
Water-Cement Ratio	0.42
Coarse Aggregate	765 kg/m ³ of concrete
Fine Aggregate	915 kg/m ³ of concrete
Ratio Fine/Total Aggregate	0.55
Mass Density	2300 kg/m ³
Air Content	7 %
Steel fibres ($V_f = 0.6$ to 1.2%)	46.8-93.6 kg/m ³ of concrete

Table 3-3 CRC Joint Cast properties

Ingredient	Fraction by Weight (%)	Density (kg/m ³)	Content (kg/m ³ of concrete)
Densit Binder	39	Cement: 3150 Microsilica: 2250	1146
Quartz Sand			
0-0.25 mm	8	2640	236
0.25 – 1 mm	14	2640	404
1 – 4 mm	27	2740	809
Water	7	1000	202
Water-binder ratio	N/A	N/A	0.18
Steel fibres ($V_f = 2\%$)	5	7500	156

3.3.2. STEEL REINFORCEMENT PARAMETERS

Four types of steel reinforcement were considered for this research, mostly of U.S. size #3 unless otherwise noted. This includes 6.35 mm normal strength non-deformed steel bars, normal strength deformed steel bars, high strength steel rebar and high ductility stainless steel

rebar. The properties of these steel bars are summarized in Table 3-4, including identification (ID) for nomenclature purposes, the type of steel reinforcement, the rebar diameter and cross-sectional area, the yield strain and strength, and the ultimate strain and strength. The steel coupons were tested at the University of Ottawa Structures Laboratory with a GALDABINI SUN 60 Universal Floor Standing Testing Machine to determine the stress-strain relationships shown in Figure 3-2; the setup is shown in Figure 3-3. When stress-strain response did not show a well-defined yield plateau, yield strength and strain was determined using the 0.2% offset method.

The normal strength non-deformed rebar (6.3NS) was used in three of the four 80 mm deep specimens. It has a diameter of 6.35 mm, a yield strength reaching 610 MPa at a strain of 0.0054 mm/mm, and an ultimate strength of 660 MPa at a strain of 0.064 mm/mm.

The normal strength deformed rebar (#3NS) was used in seven of the fourteen specimens analyzed in this research. The bars tested had a diameter of 9.525 mm, a yield strength measured as 435 MPa at a strain of 0.002 mm/mm, and an ultimate strength of 625 MPa at a strain of 0.16 mm/mm.

The high strength steel rebar (#3HS) was used in three specimens, one per series. Microcomposite multistructural formable steel *MMFX₂* is manufactured by MMFX Technologies Corporation. Described as an uncoated, corrosion-resistant, low-carbon chromium-steel alloy, it has a much higher tensile strength than standard steel rebar; however, ductility is reduced. The bars tested had a diameter of 9.525 mm, a yield strength measured as 1000 MPa at a strain of 0.0078 mm/mm, and an ultimate strength of 1200 MPa at a strain of 0.048 mm/mm.

The high ductility stainless steel rebar (#3SS) was used in one specimen of the CRC series. *EnduraMet 32* (XM-28) is a product manufactured by Carpenter Technology Corporation. It consists of a type of stainless steel rebar with a high percentage of chromium capable of higher strengths than typical carbon steel rebar and much greater elongation limits. The high amount of chromium allows it to be corrosion-resistant due to the formation of a chromium oxide protection cover. It also has a longer lifetime than other steels, estimated at over 100 years. The bars tested had a diameter of 9.525 mm, a yield strength measured as 670 MPa at a strain of 0.0086 mm/mm, and an ultimate strength of 940 MPa at a strain of 0.37 mm/mm.

Chemical constituents of the MMFX2 and EnduraMet 32 products are included in Table 3-5. These were obtained from product guides and technical data sheets available on MMFX Technologies Corporation and Carpenter Technology Corporation's respective websites.

Table 3-4 Steel reinforcement mechanical properties

ID	Steel Reinforcement	Bar Diameter d_b (mm)	Bar Area A_b (mm ²)	Yield Strain ϵ_y	Yield Strength f_y (MPa)	Ultimate Strain ϵ_u	Ultimate Strength f_u (MPa)
6.3NS	Non-deformed	6.350	32	0.0054	610	0.064	660
#3NS	Regular	9.525	71	0.0020	435	0.16	625
#3HS	MMFX ₂	9.525	71	0.0078	1000	0.048	1200
#3SS	EnduraMet 32 (XM-28)	9.525	71	0.0086	670	0.37	940

Table 3-5 Steel reinforcement chemical properties

Element	MMFX ₂	EnduraMet 32
Carbon	0.07%	0.06% (Maximum)
Chromium	9.3%	16.50 – 19.00%
Manganese	0.6%	11.00 – 14.00%
Nitrogen	0.02%	0.20 – 0.45%
Phosphorus	0.01%	0.06% (Maximum)
Sulfur	0.014%	0.03% (Maximum)
Silicon	0.14%	1.00% (Maximum)
Nickel	-	0.50 – 2.50%

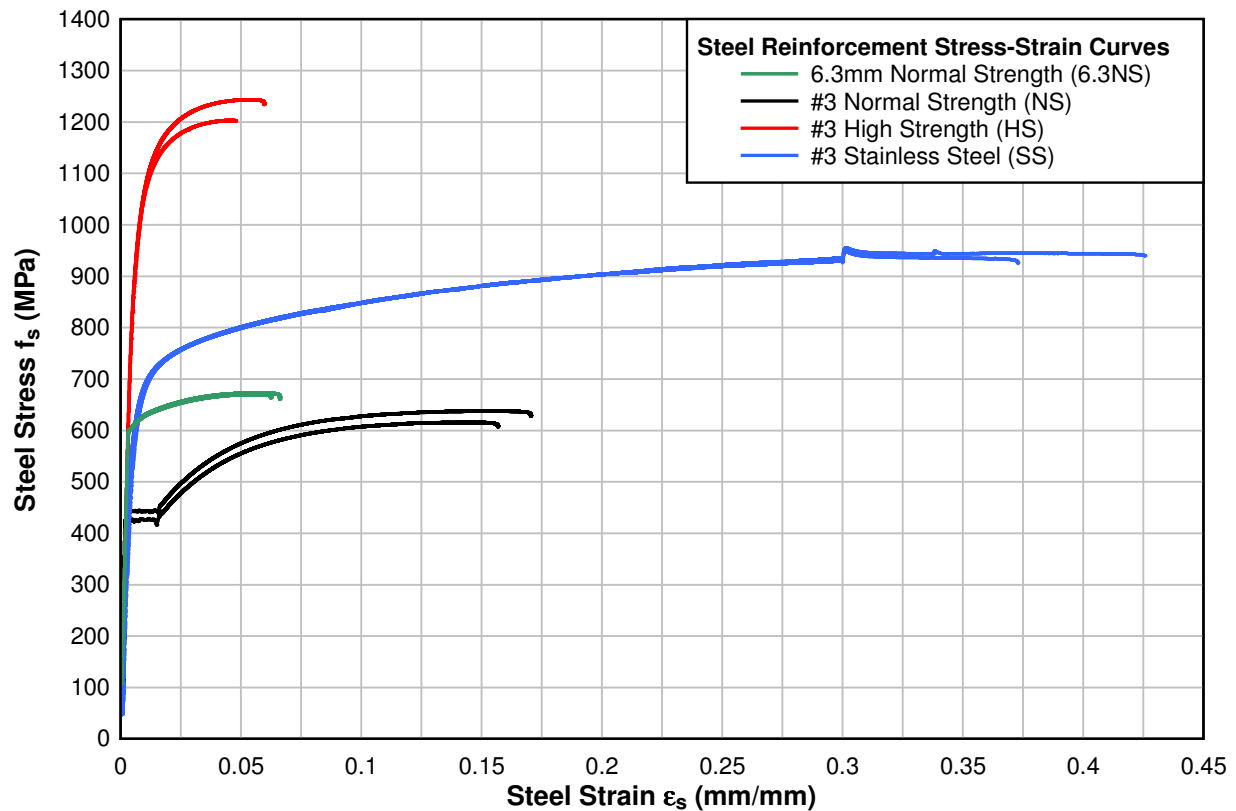


Figure 3-2 Steel reinforcement stress-strain relationships



Figure 3-3 GALDABINI Universal Floor Standing Testing Machine; testing steel rebar

3.3.3. STEEL FIBRES PARAMETERS

Four types of steel fibre reinforcement were considered for this experimental program. The summary of each can be found in Table 3-6, followed up by photographs of the fibres.

In terms of the deformed structural steel fibres, *BAKAERT Dramix® ZP305* (Figure 3-4(a)) was used in four SFRC specimens and consists of hooked-end steel wires with an aspect ratio of 55 and a tensile strength estimated at 1350 MPa. Similarly, used for one SFRC specimen, the brass-colored *BAKAERT Dramix® BP 80/30* (Figure 3-4(b)) offers a higher aspect ratio of 79 and a greater tensile strength of 3070 MPa.

For micro-fibres, a category of straight, thinner fibres with no hooked-ends, the *BEL M-0.30/13-3150* (Figure 3-4(c)) was used in three of the CRC specimens, has an aspect ratio of 43 and a tensile strength of 3150 MPa. The last three CRC specimens were cast using *BAKAERT Dramix® OL 13/20* (Figure 3-4(d)) with a higher aspect ratio of 62 but a lower tensile strength of 2750 MPa.

Table 3-6 Steel fibres properties

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
BP	BP 80/30	30	0.38	79	3070
A	OL 13/.20	13	0.21	62	2750
B	BEL M-0.30/13-3150	13	0.30	43	3150



a) ZP 305 fibres



b) BP 80/30 fibres



d) BEL M-0.30/13-3150 "B" fibres



c) OL 13/.20 "A" fibres

Figure 3-4 Steel fibres photographs

3.4. CONSTRUCTION OF TEST SPECIMENS

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

3.4.1. PREPARATION AND CASTING

Formworks were constructed from 1220 x 2440 mm sheets of 19 mm thick plywood and 38 x 64 mm wood studs for solidification. Additional holes were cut where the top of the

concrete specimen would be located to install hooks for lifting. Motor oil was spread on the bottom and sides of the formwork to ease the removal of the specimens and allow the formwork to be reused. Non-deformed steel bars were used to maintain a clear cover of 6 mm on the tension side of the specimens and steel chairs were built to keep the compression side in place.

Three meter long steel reinforcement bars were bent in the laboratory to form 180° hooks with 63 mm diameters and an approximate extension length of 150 mm at both ends of the bars. A hand held belt grinder was used to smooth a small area near the mid-span of the steel rebar to install strain gauges. Afterwards, the steel rebar were placed in the formworks and attached to the steel chairs with wires to restrict movement during casting. Photographs of the formwork and different steel reinforcement bars are included in Figure 3-5.

Concrete was prepared at the University of Ottawa Structures Laboratories using a 420V pan mixer shown in Figure 3-6 and pre-packaged concrete mixes. After pouring and levelling the slabs, wet burlap sheets and plastic sheets were used to cure the slabs for 7 days. When this period was finished, the curing sheets were removed and the specimens were air-cured until testing occurred.



a) Regular steel reinforcement



b) High strength / Stainless steel reinforcement

Figure 3-5 Preparation of specimens



Figure 3-6 Concrete pan mixer

3.4.2. FRESH STATE PROPERTIES

Slump tests were conducted in accordance with ASTM C1611 “Standard Test Method for Slump Flow of Self-Consolidating Concrete” (ASTM International 2014). The concrete mix is placed in a conical mold and released on a flat, unconfined surface. This allows a measurement of the flowability and workability of each concrete mix; results were tabulated in Table 3-7 and photographs were included in Figure 3-7. A higher slump means the mix has a greater flow potential and should require less vibration during placement.

Table 3-7 Concrete fresh state properties

Specimen ID	Concrete Mix	Fibre Ratio	Fibre Type	Slump (mm)	Average Slump (mm)
SCC-0%-6.3NS-80	SCC	0.0%	N/A	580/580	580
SCC-0%-#3NS-100	SCC	0.0%	N/A	460/480	470
SCC-0%-#3HS-100	SCC	0.0%	N/A	690/670	680
SCC-1.2%ZP-6.3NS-80	SCC	1.2%	ZP	530/530	530
SCC-0.6%ZP-#3NS-100	SCC	0.6%	ZP	605/610	608
SCC-1.2%ZP-#3NS-100	SCC	1.2%	ZP	580/610	595
SCC-0.6%BP-#3NS-100	SCC	0.6%	BP	600/650	625
SCC-1.2%ZP-#3HS-100	SCC	1.2%	ZP	610/600	605
CRC-2%B-6.3NS-80	CRC	2.0%	B	470/470	470
CRC-2%B-#3NS-80	CRC	2.0%	B	360/360	360
CRC-2%B-#3NS-100	CRC	2.0%	B	480/510	495
CRC-2%A-#3NS-100	CRC	2.0%	A	330/320	325
CRC-2%A-#3HS-100	CRC	2.0%	A	420/460	440
CRC-2%A-#3SS-100	CRC	2.0%	A	440/430	435

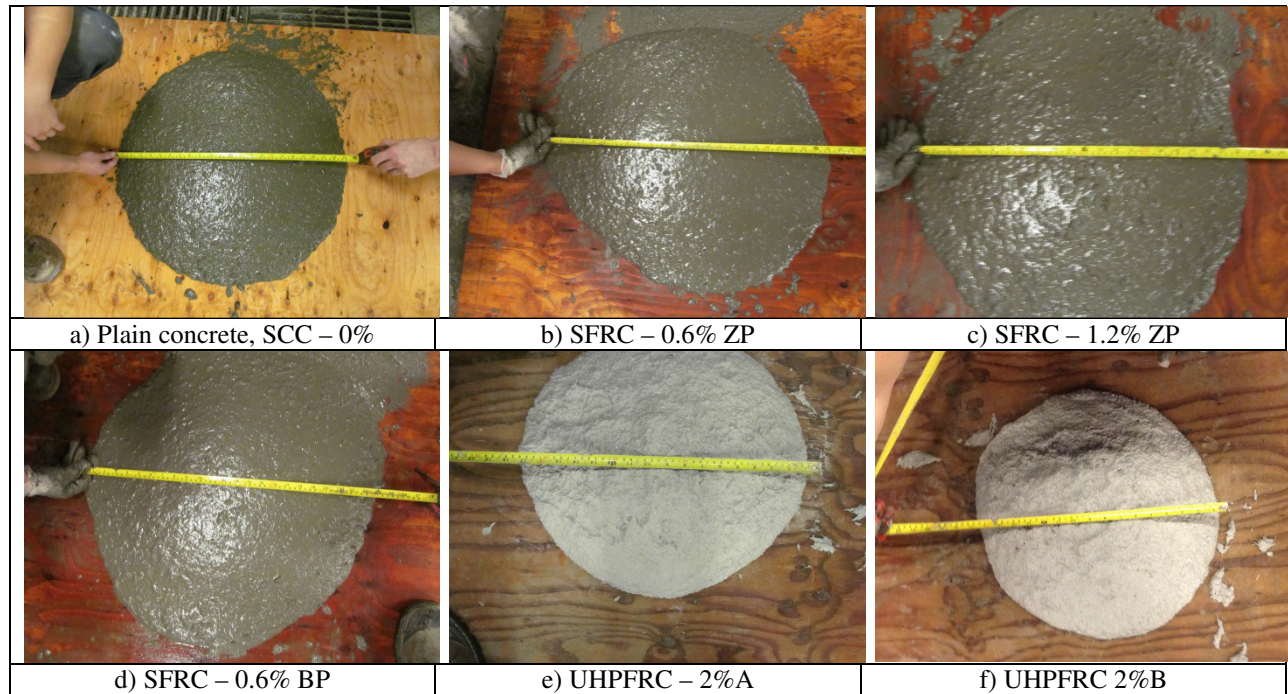


Figure 3-7 Typical concrete slump test results

3.4.3. HARDENED STATE PROPERTIES

Several concrete cylinders and flexural concrete beams were prepared during casting to obtain the hardened state properties of the different concrete mixes.

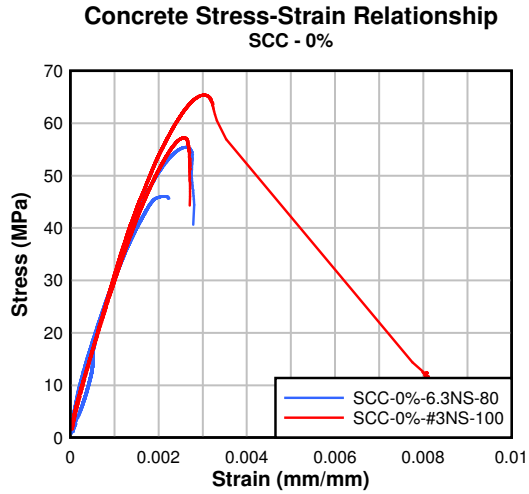
The compressive strength and stress-strain curves were measured from cylinders specimens with a diameter of 100 mm and a height of 200 mm. The cylinder testing machine seen on the left of Figure 3-8 was connected to a computer to record the force applied on the specimen. An extensometer was also attached to the cylinder and connected to the computer to measure axial displacements. The resulting compressive strengths were tabulated in Table 3-8 while samples of the stress-strain curves are included in Figure 3-9. Sample pictures of failed cylinders can be seen in Figure 3-10. Both the stress-strain curves and the sample pictures show that mixes with fibres have greater toughness than plain concrete and that they would elongate further prior to failure, hence the higher concentration of cracks on the SFRC and UHPFRC cylinders. With UHPFRC, a characteristic plateau could be observed during stress-strain tests. Finally, despite their high-strength, with smaller steel fibres in greater concentration, UHPFRC mixes would also hold the concrete together better than the SFRC mixes during cracking; this is noticeable when comparing Figure 3-10(b), Figure 3-10(b) and Figure 3-10(d) photographs against Figure 3-10(e) and Figure 3-10(f).



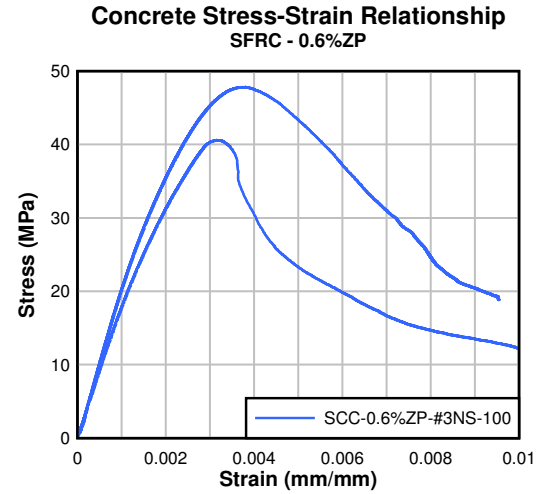
Figure 3-8 Cylinder testing setup

Table 3-8 Concrete compressive strength summary

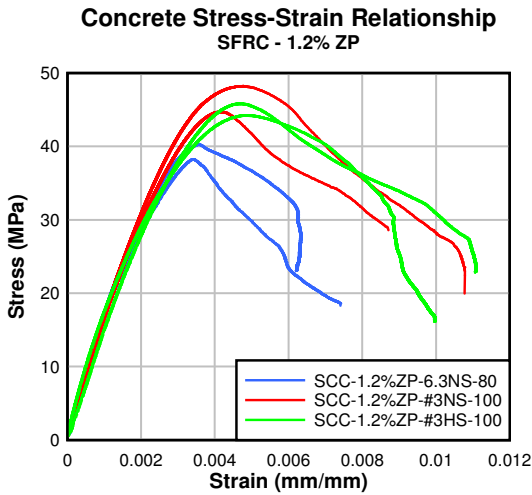
Specimen ID	Concrete Mix	Fibre Type	Fibre Ratio	Cylinder Test Compressive Strength (MPa)			
				#1	#2	#3	Avg.
SCC-0%-6.3NS-80	SCC	N/A	0.0%	46.08	55.47	-	50.78
SCC-0%-#3NS-100	SCC	N/A	0.0%	57.27	65.43	-	61.35
SCC-0%-#3HS-100	SCC	N/A	0.0%	58.61	-	-	58.61
SCC-1.2%ZP-6.3NS-80	SCC	ZP	1.2%	41.38	38.23	40.29	39.97
SCC-0.6%ZP-#3NS-100	SCC	ZP	0.6%	41.82	40.61	47.83	43.42
SCC-1.2%ZP-#3NS-100	SCC	ZP	1.2%	41.69	44.73	48.22	44.88
SCC-0.6%BP-#3NS-100	SCC	BP	0.6%	32.78	41.61	36.33	36.91
SCC-1.2%ZP-#3HS-100	SCC	ZP	1.2%	44.25	45.84	-	45.05
CRC-2%B-6.3NS-80	CRC	B	2.0%	149.38	148.01	-	148.70
CRC-2%B-#3NS-80	CRC	B	2.0%	161.24	144.54	-	152.89
CRC-2%B-#3NS-100	CRC	B	2.0%	140.29	137.90	143.36	140.51
CRC-2%A-#3NS-100	CRC	A	2.0%	148.46	150.58	149.61	149.55
CRC-2%A-#3HS-100	CRC	A	2.0%	146.54	128.37	133.14	136.02
CRC-2%A-#3SS-100	CRC	A	2.0%	146.30	-	-	146.30



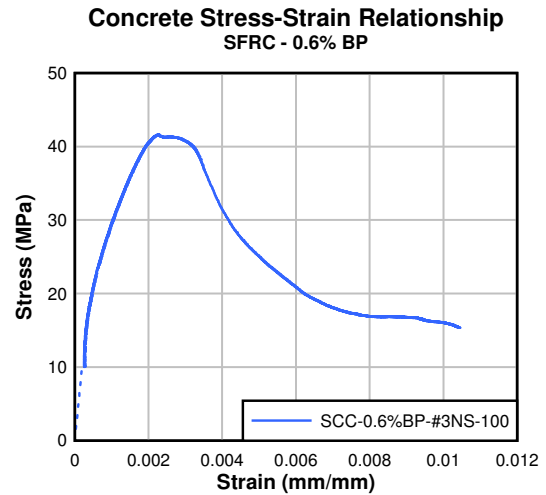
a) Plain concrete, SCC – 0%



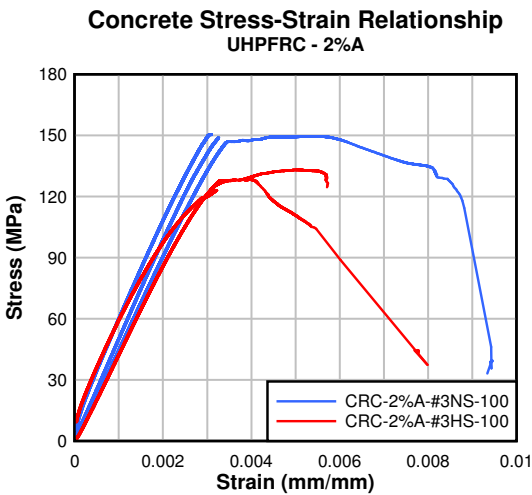
b) SFRC – 0.6% ZP



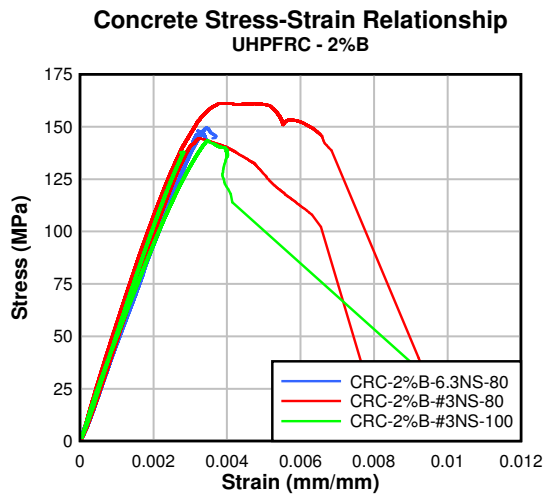
c) SFRC – 1.2% ZP



d) SFRC – 0.6% BP



e) UHPFRC – 2% A



f) UHPFRC – 2% B

Figure 3-9 Concrete stress-strain relationship samples

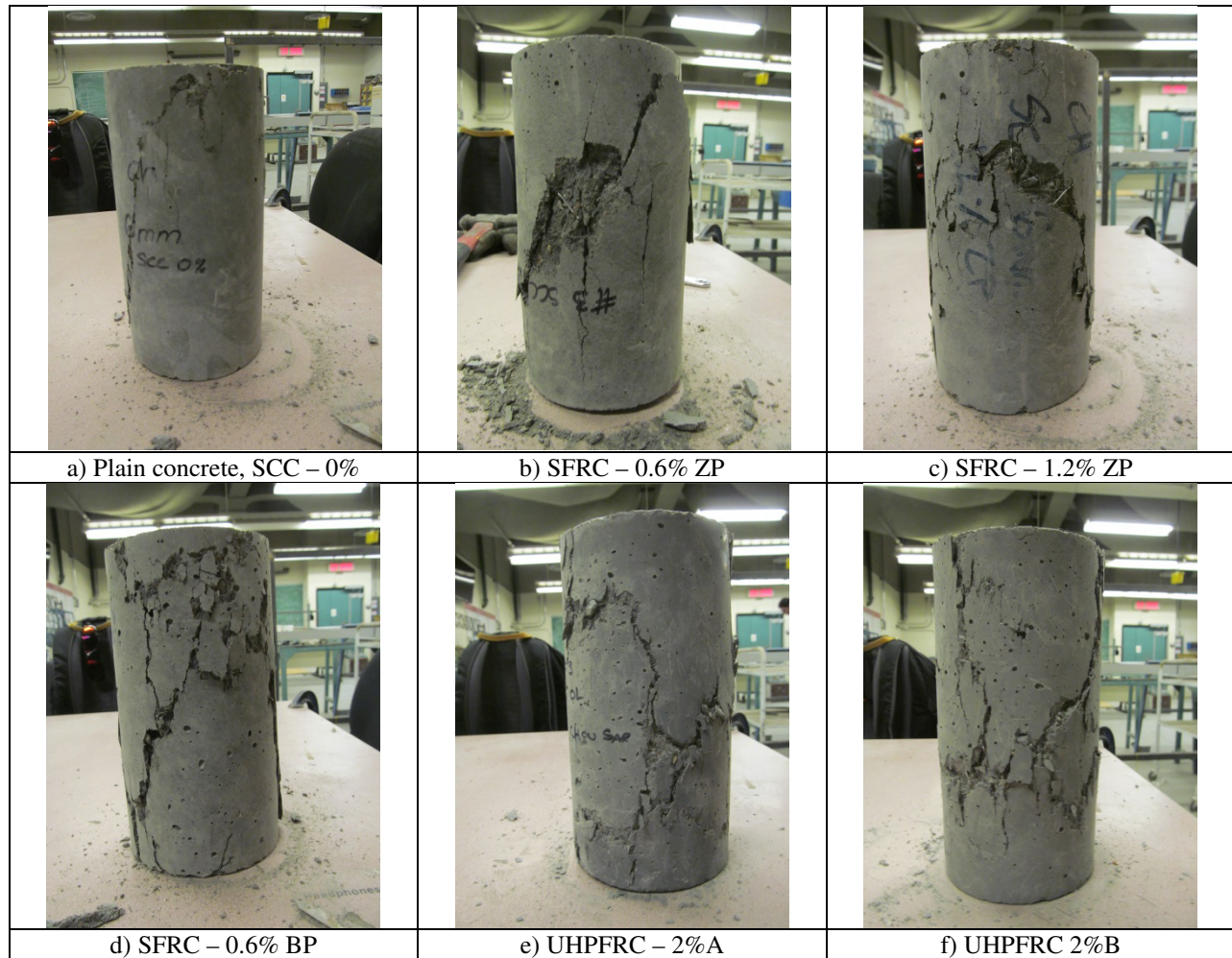


Figure 3-10 Sample concrete cylinder testing failures

To determine the flexural toughness of different concrete mixes, four-point bending tests were completed on several flexural concrete beams in accordance with the ASTM C1609 standard. For each concrete batch, at least one beam was cast with dimensions of 100 mm x 100 mm x 400 mm. As defined by the ASTM C1609 standard, the flexural beams were subjected to third-point loading over a span of 300 mm and deflections were measured at mid-span, as seen on Figure 3-11. The properties obtained from the ASTM C1609 tests are compiled in Table 3-9. Load-deformation curves are grouped in Figure 3-12.

Some batches demonstrate deflection hardening, such as SFRC-1.2%ZP and UHPFRC-2%B, where a maximum load is reached prior to the peak load. Other concrete mix load-deformation curves illustrate deflection softening, such as SFRC-0.6%ZP and UHPFRC-2%A, where the first peak is also the maximum peak load. For both SFRC and UHPFRC, the addition of steel fibres greatly increased flexural toughness and post-cracking ductility when compared to the plain concrete mix in Figure 3-12(a). UHPFRC also demonstrates impressive tensile properties, reaching over 17 MPa in peak flexural strength, nearly three times the plain concrete strength.

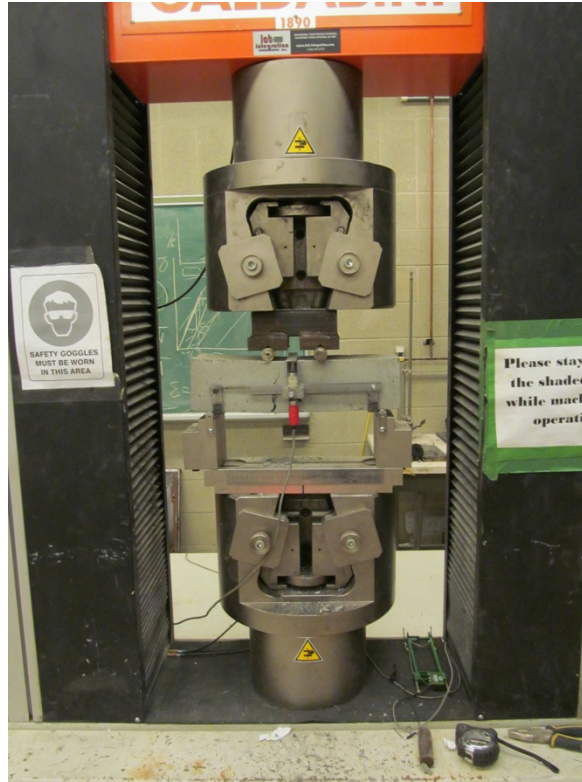
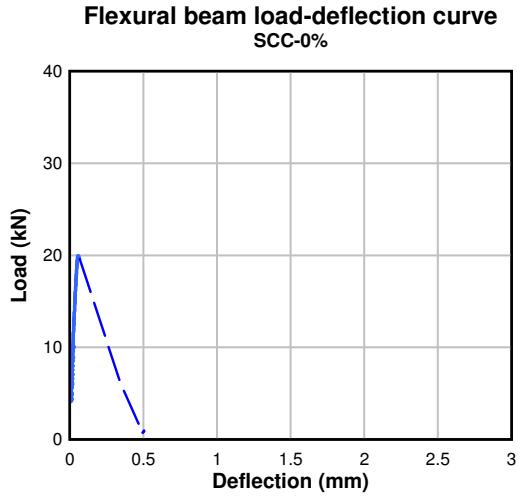


Figure 3-11 Flexural beam testing setup

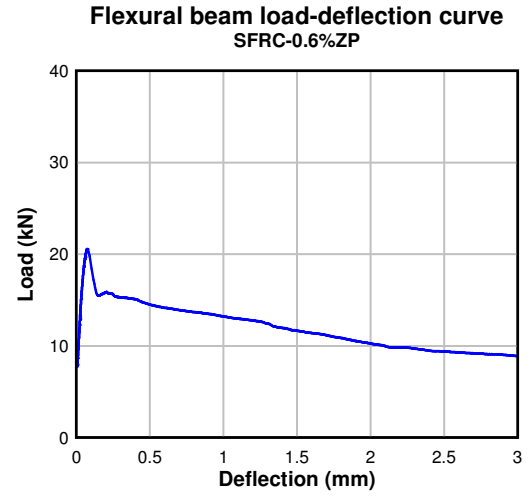
Table 3-9 Results from ASTM C1609 toughness tests

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%	-	-	19.94	0.055	-	5.98	-	-	-	-	-
SFRC-0.6%ZP	20.57	0.074	20.57	20.57	6.17	6.17	14.37	4.31	10.15	3.05	26.47
SFRC-1.2%ZP	19.15	0.313	19.70	0.628	5.75	5.91	19.58	5.87	13.09	3.93	33.97
SFRC-0.6%BP	-	-	12.98	0.196	-	3.89	11.83	3.55	7.06	2.12	18.82
UHPFRC-2%A	66.62	0.188	66.62	0.188	19.99	19.99	53.80	16.14	26.18	7.85	86.17
UHPFRC-2%B	53.91	0.128	56.73	0.261	16.17	17.02	51.55	15.47	22.06	6.62	77.65

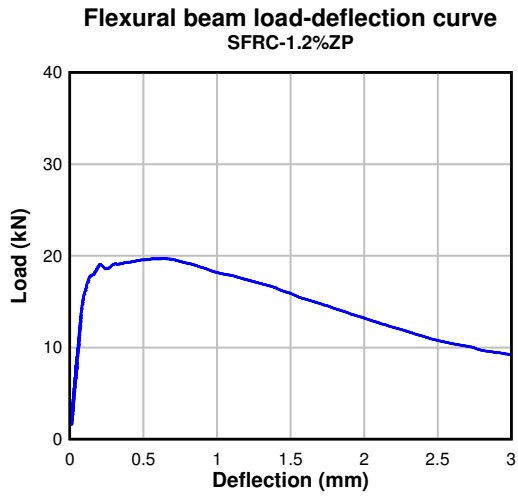
L = Span Length (300 mm), ($L/600 = 0.5$, $L/150 = 2$)
 P_1 = First-Peak Load (kN)
 δ_1 = Net Deflection at First-Peak Load (mm)
 P_p = Peak Load (kN)
 δ_p = Net Deflection at Peak Load (mm)
 f_1 = First-Peak Strength (MPa)
 f_p = Peak Strength (MPa)
 P_{600} = Residual Load at net deflection of $L/600$ (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 = $(T_{150} * L) / (L/150 * b * d^2)$



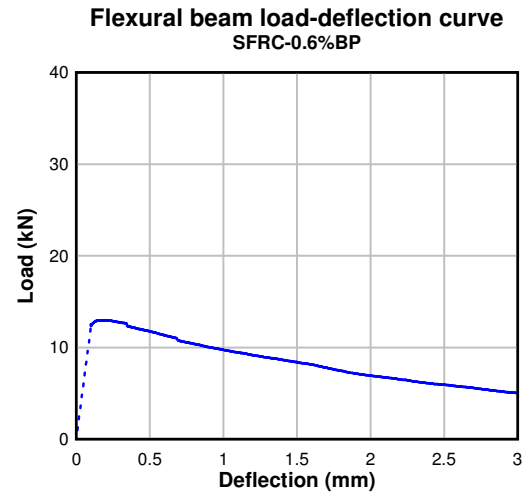
a) Plain concrete, SCC – 0%



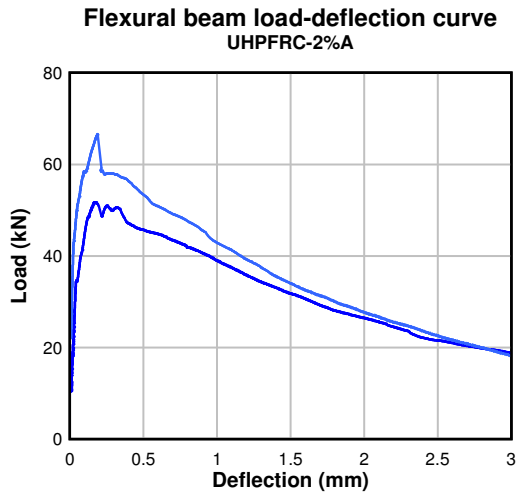
b) SFRC – 0.6% ZP



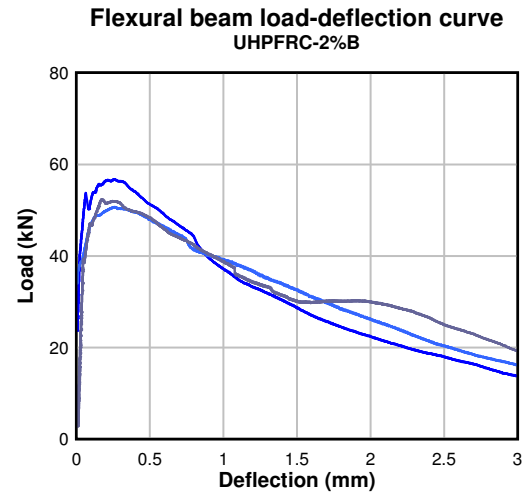
c) SFRC – 1.2% ZP



d) SFRC – 0.6% BP



e) UHPFRC – 2% A



f) UHPFRC – 2% B

Figure 3-12 Flexural beam load-deflection curve samples

3.5. EXPERIMENTAL SETUP

The following section describes the various elements composing the experimental setup such as the shock-tube, the load transfer device, and any related instrumentation including high speed cameras, LVDT, strain gauges and data acquisition equipment.

3.5.1. SHOCK-TUBE

The University of Ottawa shock-tube is an installation capable of simulating far-ranged explosion shock waves through the use of pressure differentials. It is divided in four sections: the variable length driver, the spool, the expansion chamber and the rigid end test frame. A summary of the shock-tube's operations is briefly described in this sub-section; for further details, it is suggested to refer to *Lloyd et al. (2010)*.

Air can be pumped in both the driver and the spool sections. Pressure is controlled using a double diaphragm system composed of two sets of aluminum sheets; one set is located between the driver and the spool, the second set is kept between the spool and the expansion bay. The differential of pressure on both sides of the sheets must be maintained under the aluminum sheets rupture limits until the driver reaches the desired pressure for the blast simulation. Once this pressure is achieved, the pressure in the spool section is released, introducing a differential higher than the limits of the first set of aluminum sheet which triggers a chain reaction. Both diaphragms are ruptured by the driver's pressure which then passes through the expansion chamber until it reaches the specimen located at the rigid end test frame. Excess pressure is then released by pressure relief vents on the sides of the frame.

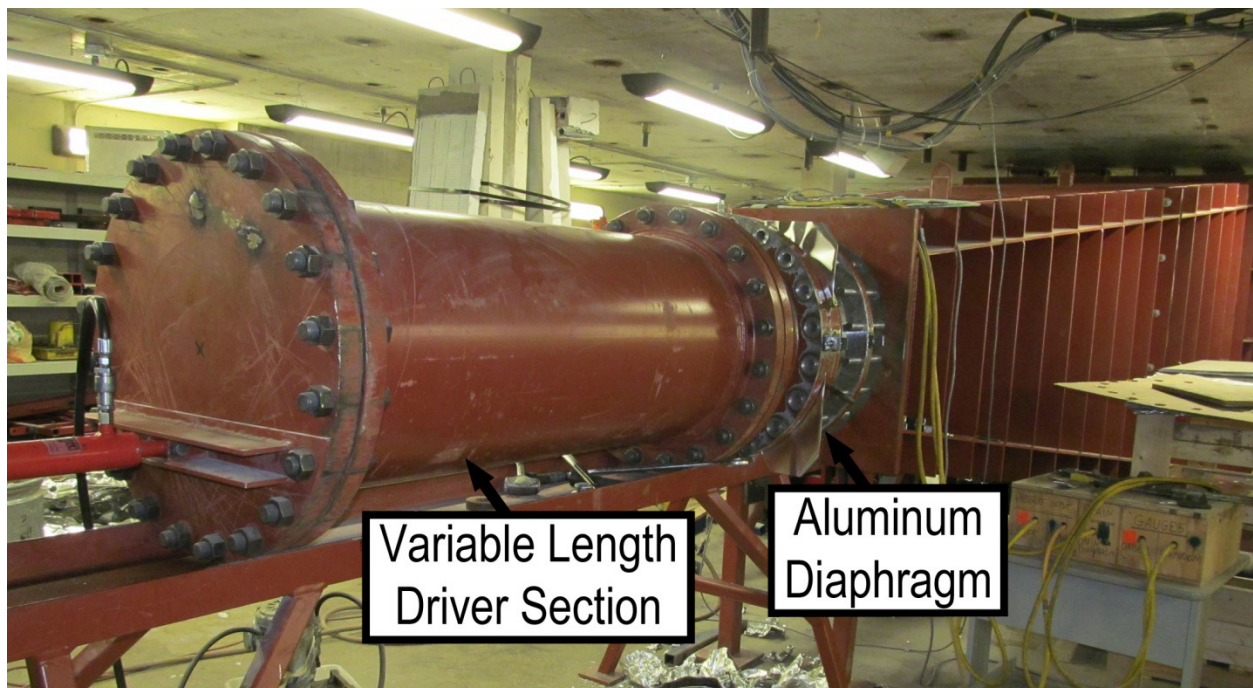


Figure 3-13 Shock-tube's driver section

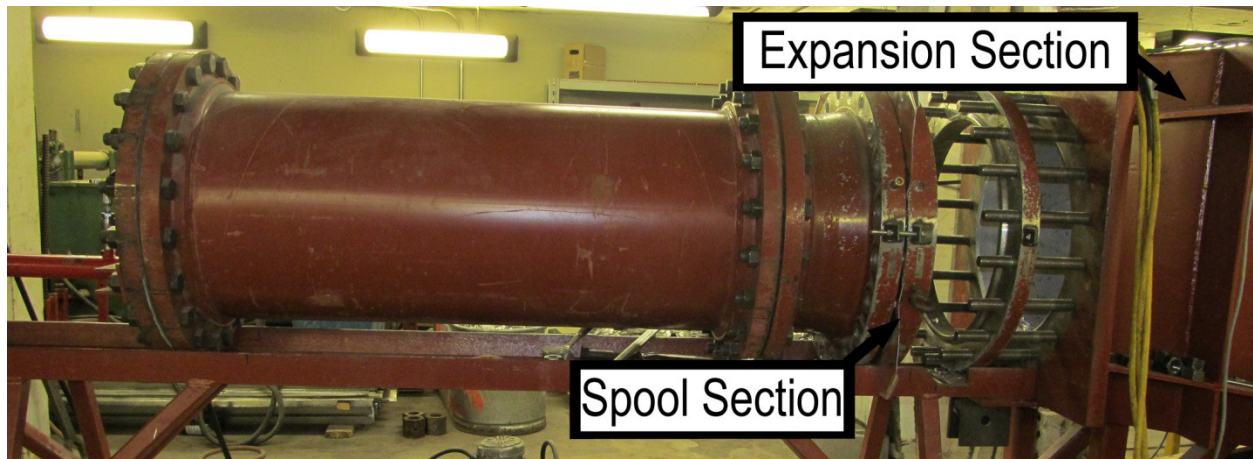


Figure 3-14 Shock-tube's spool section

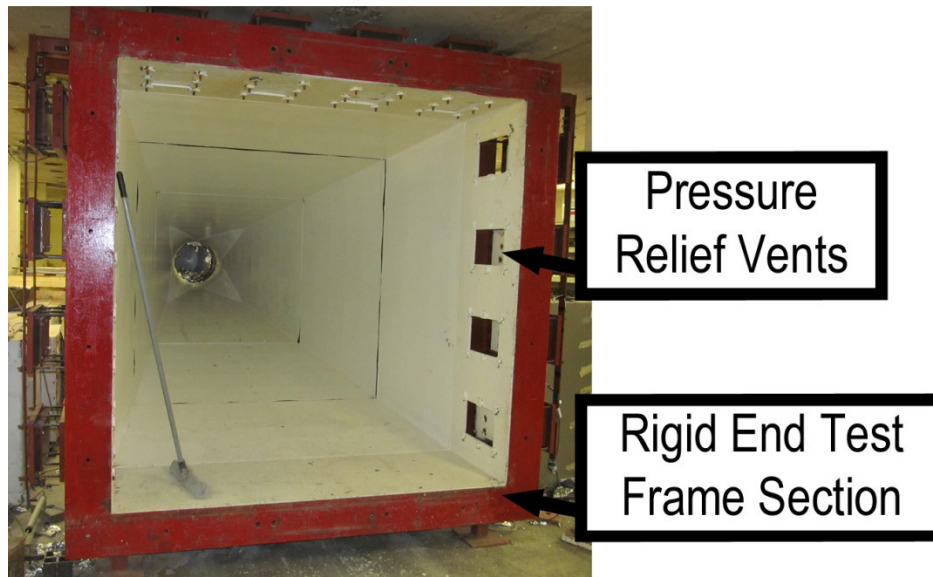


Figure 3-15 Shock-tube's rigid end test frame section

3.5.2. LATERAL LOAD TRANSFER DEVICE

In the case of non-planar structural elements or specimens which can't cover the entire opening of the shock-tube, a square area of 2032 mm in width, a lateral load transfer device (LTD) can be used to redirect the blast loads to the specimen. For this research, slab specimens were built shorter than the opening's width meaning they require the use of the load transfer device to be tested.

To simulate dynamic uniformly distributed loads, the load transfer device was constructed out of 22 gauge cold rolled steel sheets on which were attached eight 76 x 76 x 6.35 mm hollow steel sections (HSS). The top HSS was then attached to the top of the shock-tube's end test frame using threaded rods and steel chains; the bottom was free to move during the blast simulation. Refer to Figure 3-16 for a photograph depicting the setup.

The total mass of the LTD (210kg) and 440 mm wide slab system is estimated to be lower than the mass of a single one-way panel having the same size as the shock-tube's opening. The LTD also doesn't affect the stiffness of the specimen. However, only the positive phase of the shock wave will be transferred to the specimen.

3.5.3. SUPPORTS

The setup was configured to simulate a simply supported beam with a dynamic uniformly distributed load. For both supports, two pieces were constructed to allow rotation.

The first piece located on the end test frame was built from a 51 x 51 x 6.35 mm HSS with a length of 610 mm welded with a 19 mm cold rolled steel rod with a length of approximately 440 mm. One hole was drilled on each side to allow a 19 mm threaded rod to slide through.



Figure 3-16 Lateral load transfer device

The second piece located at the front of the specimen was built from a 152 x 152 x 6.35 mm HSS with a length of 1200 mm welded to a 19 mm cold rolled steel rod with a length of approximately 440 mm. An additional 102 mm wide, 6.35 mm thick plate was welded on the tension side of the HSS for additional stiffness. Four holes were drilled to allow 19 mm threaded rod to slide and connect the supports to the shock-tube.

The specimens were placed on wooden blocks and then the supports were tightened to hold the slab between the steel rods. The approximate distance between both supports was 2232 mm.

3.5.4. DATA ACQUISITION SYSTEM

Data is recorded with a Yokogawa SL1000 High Speed Data Acquisition Unit capable of recording 100,000 samples of data per second. These samples include strains, displacements and pressures. When testing, the unit is set to loop, record and constantly overwrite data until signalled by a pressure sensor that the blast has occurred, which is called the trigger. Once triggered, the unit saves data over a period of 3.0 seconds, including a 10% pre-trigger phase lasting 0.3 seconds. It can then be imported using XViewer to a .csv format for further analysis.

3.5.5. HIGH SPEED VIDEO CAMERA

The behaviour of the specimens during the blast simulations was recorded using a high speed camera, the AOS Technology X-PRI. It is a camera capable of recording full color images with 800x600 pixels resolution at a rate of 1000 frames per second.

The camera was installed in an angle to show both the front and side of the specimen and set to record 1.0 second at a rate of 500 frames per second. Similarly to the data acquisition unit, it runs in a loop until triggered by the said unit. Then, it records an uncompressed video file with a 10% pre-trigger phase. The file can then be modified to different rates depending on the required needs.

3.5.6. PRESSURE SENSORS

Two PCB Piezotronics Model #112A22 piezoelectric pressure sensors are located on the side and bottom of the shock-tube's end frame. These are connected to the data acquisition unit and serve two purposes: to obtain representative pressure-time histories from the blast simulation on the load transfer device, and to send the trigger signal to record data samples.

3.5.7. LINEAR VARIABLE DISPLACEMENT TRANSDUCERS (LVDT)

Two Celesco CLWG-300 linear variable displacement transducers limited were installed perpendicular with the specimen at mid-span and one third of the span. These allowed the data acquisition unit to obtain measurements of the lateral displacement of the slabs during and after the blast simulation. A third LVDT was also installed at the bottom of the shock-tube to subtract any displacement from the installation itself. The full stroke range for these particular LVDT is 300 mm.

3.5.8. STRAIN GAUGES

For each specimen, two FLA-6-350-11 strain gauges were installed at the mid-span of the specimen on the two middle reinforcement bars in tension as seen on Figure 3-17. These strain gauges have a length of 6 mm and a resistance of 350 ± 1.0 ohms.

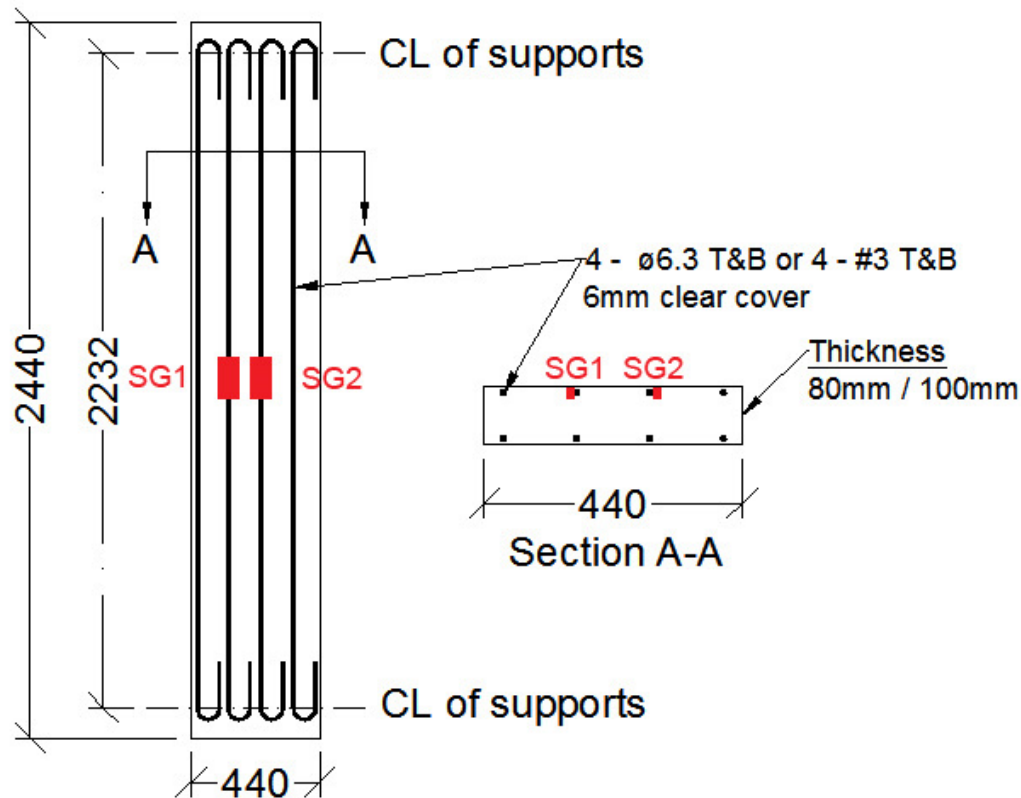


Figure 3-17 Strain gauges location

3.6. EXPERIMENTAL PROCEDURE

Each specimen was subjected to repeated blasts until failure. Failures include bar rupture, bar slippage, debonding, crushing, excessive tensile cracking and excessive residual displacement. These blasts increase gradually in pressure as seen in Table 3-10 and in the sample pressure time histories from Figure 3-18. The initial test is expected to be within the elastic range for most of the specimens while the second blast should reach past the yielding point for regular steel reinforcement. The selected driver length was 1830 mm and the selected driver pressures were increments of 103 kPa (15 psi); blast 6 had a slightly lower increment due to the limitations of the compressor that was used to pump air into the double diaphragm system.

Table 3-10 Blast test properties

Test #	Driver Pressure kPa (psi)	Driver Length mm (ft)	Avg. Reflected Pressure (P_r) kPa	Avg. Reflected Impulse (I_r) kPa-ms	Avg. Positive Phase Duration (t_p) ms
Blast 1	103 (15)	1830 (6)	19.5	137	13.5
Blast 2	207 (30)	1830 (6)	35.2	240	14.6
Blast 3	310 (45)	1830 (6)	46.5	319	15.1
Blast 4	414 (60)	1830 (6)	56.7	373	15.5
Blast 5	517 (75)	1830 (6)	66.5	437	16.0
Blast 6	607 (88)	1830 (6)	77.5	511	16.9

Sample Pressure Time Histories
(SCC 0% #3HS 100)

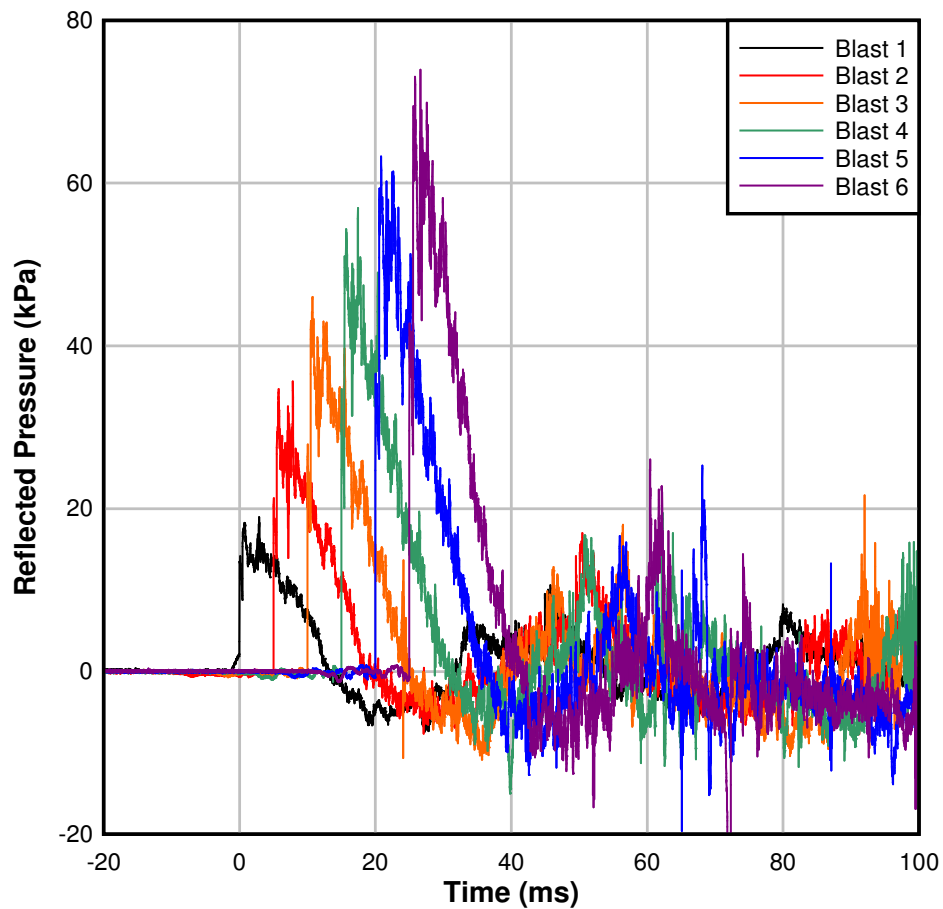


Figure 3-18 Sample pressure time histories for blasts 1-6

CHAPTER 4. EXPERIMENTAL DATA

4.1. CHAPTER OVERVIEW

The following chapter contains the various results obtained from the previously described experimental program. For each test of the fourteen concrete slab specimens, data has been provided; whether it's the reflected pressure or reflected impulse magnitude, the maximum or residual mid-span displacements, the strain in the tensile reinforcement steel bars, or the damage that occurred from the blast.

A summary of the results has been included in Section 4.2. Further explanations of each test results have been described within Sections 4.3 to 4.5, depending on the series. Additional graphics, such as reflected pressure and impulse time histories, or reflected pressure and mid-span displacement time histories, were compiled for Section 4.6.

4.2. SUMMARY OF EXPERIMENTAL RESULTS

The overall summary of the experimental results have been included in Table 4-1 to Table 4-3, starting with the control specimens, followed by the SFRC and then the UHPFRC slabs. The tables describe the shockwave properties of each test, including the shock-tube's driver length (L), the approximate driver pressure (P_d), the positive reflected pressure (P_r) and its phase duration (t_p), the reflected impulse (I_r), as well as the specimen's relative maximum and residual mid-span displacements ($\delta_{\max}$ and δ_{res}) and maximum relative support rotation ($\theta_{\max}$).

The reflected pressure values were recorded from a pressure transducer on the side of the shock-tube end test frame and represent accurately the pressure acting on the load transfer device. Each test's associated reflected impulse was calculated with the integral of the pressure-time history curve using the software DPlot by HydeSoft Computing, LLC. It should also be noted that mid-span displacements and support rotation values within the three aforementioned tables are not cumulative.

A minimum of three blasts were conducted on each specimen, incrementing the driver's pressure by about 103kPa (15psi) per shot. Specimens with a larger depth, higher strength concrete and steel could reach up to a sixth incremental blast prior to failure.

Table 4-1 Blast data summary (1/3)

Name		Driver Length	Approximate Driver Pressure	Reflected Pressure	Positive Phase Duration	Reflected Impulse	Max Deflection	Max Support Rotation	Time to Max	Residual Deflection
<i>Specimen ID</i>	<i>Blast #</i>	<i>L_d</i> mm	<i>P_d</i> kPa	<i>P_r</i> kPa	<i>t_p</i> ms	<i>I_r</i> kPa-ms	<i>δ_{max}</i> mm	<i>θ_{max}</i> degree	<i>t_{max}</i> ms	<i>δ_{res}</i> mm
SCC-0%-6.3NS-80	-1	1830	103	18.7	13.1	127	43.2	2.2	38.2	17.0
	-2	1830	207	32.5	13.8	210	100.6	5.2	68.1	46.6
	-3	1830	310	46.1	14.1	275	176.4	9.0	88.2	83.8
SCC-0%-#3NS-100	-1	1830	103	18.9	13.5	138	23.5	1.2	27.6	7.1
	-2	1830	207	34.6	14.8	238	52.8	2.7	37.2	29.3
	-3	1830	310	45.3	15.1	319	84.9	4.3	42.5	46.3
	-4	1830	414	53.2	15.2	351	110.2	5.6	46.7	58.1
	-5	1830	517	-	-	-	-	-	-	-
SCC-0%-#3HS-100	-1	1830	103	19.0	13.0	131	18.9	1.0	26.9	1.8
	-2	1830	207	35.7	13.3	222	36.3	1.9	23.9	2.7
	-3	1830	310	46.0	15.0	323	56.6	2.9	30.1	9.5
	-4	1830	414	57.0	15.9	382	76.1	3.9	29.8	15.5
	-5	1830	517	63.3	17.1	440	93.9	4.8	31.8	24.3
	-6	1830	604	73.9	16.5	498	-	-	-	-
SCC-1.2%ZP-6.3NS-80	-1	1830	117	24.7	14.0	156	47.1	2.4	33.9	14.6
	-2	1830	207	36.7	15.0	231	97.6	5.0	64.8	43.2
	-3	1830	310	48.8	15.0	308	152.8	7.8	65.4	68.2
	-4	1830	414	56.4	14.5	354	180.7	9.2	65.7	99.7
SCC-0.6%ZP-#3NS-100	-1	1830	103	20.1	13.3	138	20.7	1.1	23.4	3.6
	-2	1830	207	36.2	14.4	233	48.1	2.5	32.1	21.8
	-3	1830	310	45.8	14.7	303	76.2	3.9	34.9	41.2
	-4	1830	414	52.5	15.0	405	122.1	6.2	46.1	69.8
	-5	1830	517	64.9	14.6	397	181.6	9.2	60.3	136.0

Table 4-2 Blast data summary (2/3)

Name		Driver Length	Approximate Driver Pressure	Reflected Pressure	Positive Phase Duration	Reflected Impulse	Max Deflection	Max Support Rotation	Time to Max	Residual Deflection
<i>Specimen ID</i>	<i>Blast #</i>	<i>L_d</i> mm	<i>P_d</i> kPa	<i>P_r</i> kPa	<i>t_p</i> ms	<i>I_r</i> kPa-ms	<i>δ_{max}</i> mm	<i>θ_{max}</i> degree	<i>t_{max}</i> ms	<i>δ_{res}</i> mm
SCC-1.2%ZP-#3NS-100	-1	1830	103	19.7	13.3	136	23.9	1.2	24.9	3.8
	-2	1830	207	34.7	15.0	238	44.2	2.3	34.4	21.0
	-3	1830	310	46.4	15.0	323	72.9	3.7	37.1	38.1
	-4	1830	414	55.5	14.9	369	105.1	5.4	43.2	65.5
	-5	1830	517	62.9	14.9	418	155.9	8.0	47.3	98.3
SCC-0.6%BP-#3NS-100	-1	1830	103	-	-	-	22.6	1.2	-	4.6
	-2	1830	207	36.6	14.6	258	52.0	2.7	29.7	28.1
	-3	1830	310	46.6	15.1	318	73.1	3.7	39.1	41.8
	-4	1830	414	55.8	14.9	356	99.2	5.1	41.9	59.4
	-5	1830	517	75.0	14.6	398	158.6	8.1	47.6	95.9
SCC-1.2%ZP-#3HS-100	-1	1830	103	21.2	13.2	143	18.3	0.9	24.8	2.3
	-2	1830	207	33.9	13.8	253	38.0	2.0	28.2	1.8
	-3	1830	310	48.0	15.0	338	61.3	3.1	28.9	11.3
	-4	1830	414	62.7	16.2	408	80.9	4.1	30.3	20.1
	-5	1830	517	68.8	17.1	488	109.3	5.6	36.8	38.3
CRC-2%B-6.3NS-80	-1	1830	103	18.4	14.4	134	20.3	1.0	26.4	6.6
	-2	1830	207	32.4	15.2	231	44.8	2.3	32.8	18.4
	-3	1830	310	42.6	15.5	295	92.3	4.7	63.4	48.0
	-4	1830	414	51.3	15.3	320	111.2	5.7	63.2	48.0
CRC-2%B-#3NS-80	-1	1830	103	21.9	13.7	138	21.4	1.1	24.5	7.6
	-2	1830	207	34.6	15.2	241	45.3	2.3	31.3	13.7
	-3	1830	310	49.6	15.6	333	98.2	5.0	46.3	53.0
	-4	1830	414	56.0	14.2	381	194.3	9.9	65.4	158.7

Table 4-3 Blast data summary (3/3)

Name		Driver Length	Approximate Driver Pressure	Reflected Pressure	Positive Phase Duration	Reflected Impulse	Max Deflection	Max Support Rotation	Time to Max	Residual Deflection
<i>Specimen ID</i>	<i>Blast #</i>	<i>L_d</i> mm	<i>P_d</i> kPa	<i>P_r</i> kPa	<i>t_p</i> ms	<i>I_r</i> kPa-ms	<i>δ_{max}</i> mm	<i>θ_{max}</i> degree	<i>t_{max}</i> ms	<i>δ_{res}</i> mm
CRC-2%B-#3NS-100	-1	1830	103	18.7	13.4	142	15.9	0.8	20.6	1.3
	-2	1830	207	34.2	14.4	240	31.1	1.6	19.8	10.3
	-3	1830	310	46.7	15.4	320	-	-	-	23.0
	-4	1830	414	54.9	16.1	339	59.2	3.0	27.3	27.3
	-5	1830	517	64.3	16.2	419	104.8	5.4	38.7	72.1
CRC-2%A-#3NS-100	-1	1830	103	16.5	14.1	131	12.1	0.6	23.1	3.3
	-2	1830	207	39.5	15.2	267	33.9	1.7	23.9	12.4
	-3	1830	310	47.4	15.0	323	49.7	2.5	28.0	19.3
	-4	1830	414	56.5	16.4	365	67.3	3.5	33.3	38.4
	-5	1830	517	67.4	16.4	440	213.1	10.8	67.9	137.3
CRC-2%A-#3HS-100	-1	1830	103	17.3	14.0	134	13.0	0.7	21.0	2.5
	-2	1830	207	35.4	15.0	243	27.8	1.4	20.2	2.0
	-3	1830	310	46.7	15.4	350	45.1	2.3	23.7	6.5
	-4	1830	414	58.3	16.1	421	59.4	3.0	26.0	12.1
	-5	1830	517	68.5	17.1	469	71.9	3.7	26.2	14.7
	-6	1830	607	87.2	17.5	535	128.5	6.6	55.5	66.5
CRC-2%A-#3SS-100	-1	1830	103	18.6	13.1	134	14.5	0.7	21.2	4.0
	-2	1830	207	35.8	14.2	253	30.2	1.6	21.3	4.1
	-3	1830	310	45.5	15.6	340	45.8	2.4	24.6	8.9
	-4	1830	414	57.8	16.4	393	59.9	3.1	26.5	10.8
	-5	1830	517	63.2	16.4	461	81.8	4.2	29.1	25.3
	-6	1830	601	71.3	16.7	500	119.1	6.1	37.7	67.6

4.3. DESCRIPTION OF EXPERIMENTAL RESULTS – CONTROL SERIES

4.3.1. SCC - 0% - 6.3NS – 80

This specimen is the first of three slabs from the control series. It is an 80 mm deep slab composed of normal strength concrete with no fibre content, and 6.3 mm non-deformed steel bars. The shockwave and displacement data can be found in Table 4-1. Complete reflected pressure and impulse histories, as well as reflected pressure and mid-span displacement histories, can be found in Figure 4-1. Photographs detailing the effects of each blast on the specimen are included in Figure 4-2.

Blast 1 is meant to be within the elastic range for control specimens. The maximum and residual mid-span displacements were recorded at 43.2 mm and 17.0 mm respectively. The tensile strain gauges reached an average maximum strain of 0.00282 mm/mm ($\epsilon_{\text{res}} = 0.000532$ mm/mm). Hairline cracks were found at 170 mm, 700 mm and 1600 mm from the top support. Larger cracks were found near mid-span including one at 1080 mm from the top support with a width of 1.75 mm and another at 1340 mm with a width of 1.25 mm.

Blast 2 is meant to be within the plastic range for most control specimens. The maximum and residual mid-span displacements were measured as 100.6 mm and 46.6 mm respectively. Cumulative residual displacement adds up to 63.6 mm. The tensile strain gauges reached an average 0.00291 mm/mm ($\epsilon_{\text{res}} = 0.000144$ mm/mm). In terms of damage, the two major cracks at 1080 mm and 1340 mm from the top support reached a width of 5.5 mm and 2.5 mm. There was also a noticeable loss of concrete cover on the left edge of the specimen as shown on Figure 4-2(d).

Blast 3 resulted in the failure of the 80 mm deep control specimen. The maximum and residual mid-span displacements were recorded at 176.4 mm and 83.8 mm respectively. The final cumulative residual displacement was 147.4 mm. The tensile strain gauges at this point are not reliable anymore due to bar slippage. Indeed, both the far-left and the mid-right reinforcement bars have slipped within the concrete specimen, the latter provoking a 600 mm long vertical crack around mid-span with exposed rebar for 220 mm. The principal tensile crack at 1080 mm from the top support reached a width varying between 15 mm and 40 mm due to cover loss, as shown on Figure 4-2(e).

4.3.2. SCC - 0% - #3NS – 100

This slab is to be considered as the main control specimen for this research. It is composed of normal strength concrete and steel rebar, has no fibre content, and has a depth of 100 mm. The shockwave and displacement data can be found in Table 4-1. Complete reflected pressure and impulse histories, as well as reflected pressure and mid-span displacement histories, can be found in Figure 4-3 and Figure 4-4. Photographs detailing the effects of each blast on the specimen are included in Figure 4-5.

Blast 1 is meant to be within the elastic range for control specimens. The maximum and residual mid-span displacements were 23.5 mm and 7.1 mm respectively. The functional tensile strain gauge reached a maximum strain of 0.00425 mm/mm ($\epsilon_{\text{res}} = 0.00185$ mm/mm), which means the rebar has passed the elastic limit. Hairline cracks were found in a range within 420 mm and 1840 mm from the top support. The largest crack was near the mid-span of the slab and had a width of approximately 0.40 mm.

Blast 2 is meant to be within the plastic range for most control specimens. The maximum and residual mid-span displacements were 52.8 mm and 29.3 mm respectively. Cumulative residual displacement sums to 36.4 mm. The functional tensile strain gauge reached a maximum of 0.0179 mm/mm ($\epsilon_{\text{res}} = 0.00536$ mm/mm). The mid-span crack has increased to a width of 1.25 mm.

Blast 3 is the test at which the previously described 80 mm deep control specimen had failed. The maximum and residual mid-span displacements were 84.9 mm and 46.3 mm respectively. Cumulative residual displacement totals for 82.7 mm. The tensile strain gauge seems to now be unreliable as it provided a negative strain. The mid-span crack has increased to a width of 2.00 mm along with a minimal loss of concrete cover following the crack's length.

For blast 4, the maximum and residual mid-span displacements were 110.2 mm and 58.1 mm respectively. Cumulative residual displacement is calculated as 140.8 mm. The mid-span crack now varies between 4 mm and 20 mm and the concrete cover has been lost over three of the tensile rebar on a range of 200 mm in height as seen in Figure 4-5(f).

Data for the reflected pressure and mid-span displacements could not be extracted for blast 5. This blast caused the specimen to fail in flexure due to cracking, crushing and bar slippage as shown in Figure 4-5(g) and Figure 4-5(h). The concrete cover was lost on a height of 680 mm and crack widths varied between 15 mm and 30 mm. It should also be noted that secondary fragmentation occurred during the test due to bar slippage.

4.3.3. SCC - 0% - #3HS - 100

This slab serves as a comparison between the effects of high strength steel against regular steel reinforcement in the case of normal strength concrete with no fibre content. The size and composition of this specimen is identical to the main control specimen previously described except for the normal strength rebar being replaced with high strength steel. The shockwave and displacement data can be found in Table 4-1. Complete reflected pressure and impulse histories, as well as reflected pressure and mid-span displacement histories, can be found in Figure 4-6 and Figure 4-7. Photographs detailing the effects of each blast on the specimen are included in Figure 4-8.

Blast 1 is meant to be within the elastic range for control specimens. The maximum and residual mid-span displacements were 18.9 mm and 1.8 mm respectively. The tensile strain gauges reached an average of 0.00234 mm/mm ($\epsilon_{\text{res}} = 0.000303$ mm/mm), which signifies the steel bars

shouldn't be yielding. Few hairline cracks were visible but no major tensile cracking was recorded.

Blast 2 is meant to be past the yielding point for most control specimens built with conventional reinforcement steel. The maximum and residual mid-span displacements were 36.3 mm and 2.7 mm respectively. Cumulative residual displacement was calculated as 4.5 mm. The tensile strain gauges reached an average of 0.00505 mm/mm ($\epsilon_{res} = 0.000755$ mm/mm), which means the steel bars are still within the elastic range. Hairline cracks are spreading however no major tensile cracking has occurred.

For blast 3, the maximum and residual mid-span displacements were measured as 56.6 mm and 9.5 mm respectively. Cumulative residual displacement adds up to 14.0 mm. The tensile strain gauges reached an average of 0.0109 mm/mm ($\epsilon_{res} = 0.00449$ mm/mm), which means the high strength steel bars have started yielding. In terms of damage, there is an 850 mm long and 0.30 mm wide vertical crack formed at the location of the mid-right rebar. Four noticeable horizontal tensile cracks appeared in a 200 mm range near mid-span; these also have a width of 0.30 mm.

For blast 4, the maximum and residual mid-span displacements were recorded as 76.1 mm and 15.5 mm respectively. Cumulative residual displacement sums up to 29.5 mm. The tensile strain gauges reached an average of 0.0101 mm/mm ($\epsilon_{res} = -0.000935$ mm/mm); however, one of the strain gauges showed negative residual values leading to believe these are not as reliable as in the previous tests. Vertical cracking on the mid-right rebar extended to a length of 1830 mm, with the rebar itself being exposed on 240 mm with a 22 mm wide opening. Horizontal tensile cracks continued to be developed and now vary between 0.50 mm and 1.00 mm. The widest crack occurred at 1200 mm from the top support.

With blast 5, the maximum and residual mid-span displacements reached 93.9 mm and 24.3 mm respectively. Cumulative residual displacement was measured as 53.8 mm. As for damage, the previously mentioned vertical crack exposed the mid-right rebar for the full length of the specimen, with an opening width varying between 10 mm and 30 mm due to cover loss; this can be seen on Figure 4-8(g). The major tensile crack at 1200 mm from the top support was 4 mm wide.

No data was recovered for displacements during blast 6 due to the nature of the specimen failure. Secondary fragmentation occurred as seen on Figure 4-8(h), releasing chunks of concrete on several meters and weakening the middle section of the slab. The top part of the specimen collapsed due to the lack of strength in the middle section and bended the steel rebar; this resulted in the release of the specimen as seen on Figure 4-8(i).

4.4. DESCRIPTION OF EXPERIMENTAL RESULTS – SFRC SERIES

4.4.1. SCC – 1.2%ZP - 6.3NS – 80

This slab is the only specimen of the SFRC series with a depth of 80 mm. It is meant to observe the effects of structural steel fibres against blast loads when used with normal strength concrete. The composition of this specimen is similar to the first control “SCC-0%-6.3NS-80” except for the addition of ZP fibres to the concrete mix. The shockwave and displacement data can be found in Table 4-1. Complete reflected pressure and impulse histories, as well as reflected pressure and mid-span displacement histories, can be found in Figure 4-9 and Figure 4-10. Photographs detailing the effects of each blast on the specimen are included in Figure 4-11.

Blast 1 is meant to be within the elastic range for control specimens built with conventional materials. The maximum and residual mid-span displacements were 47.1 mm and 14.6 mm respectively. The tensile strain gauges reached an average of 0.00340 mm/mm ($\epsilon_{res} = 0.000232$ mm/mm), which means the steel bars are still within elastic range. Hairline cracks are visible near the middle section of the slab along with a 0.40 mm wide crack at 1150 mm from the top support.

Blast 2 is meant to be past the yielding point for most control specimens. The maximum and residual mid-span displacements were 97.6 mm and 43.2 mm respectively. Cumulative residual displacement sums up as 57.8 mm. The functional tensile strain gauge reached 0.0119 mm/mm ($\epsilon_{res} = 0.00939$ mm/mm), which means the steel bars have yielded. The principle tensile crack at 1150 mm from the top support has a width varying between 6 mm and 10 mm while other adjacent cracks vary between hairline and 0.40 mm.

Blast 3 was the test at which the similar control specimen had failed. The maximum and residual mid-span displacements were 152.8 mm and 68.2 mm. Cumulative residual displacement was calculated as 126.0 mm. The strain gauges were both unable to provide reliable data for this blast. In terms of damage, the major crack has continued to widen and now varies between 13 mm and 25 mm. The far-right rebar slipped and has been exposed over 100 mm as seen on Figure 4-11(e).

The specimen was considered as failed after blast 4. The maximum and residual mid-span displacements were measured as 180.7 mm and 99.7 mm respectively. Cumulative residual displacement totaled for 225.7 mm. The principal tensile crack reached a width varying between 25 mm and 33 mm, which is greater than the length of the steel fibres and such these cannot control the cracking anymore. The far-right tensile rebar continued slipping and has been exposed over 120 mm due to a loss in concrete cover as seen on Figure 4-11(f).

4.4.2. SCC – 0.6%ZP - #3NS – 100

This slab is the first specimen of the SFRC series with a depth of 100 mm. Similar to the main control specimen “SCC-0%-#3NS-100” except for the added ZP steel fibres, it is meant to show the effects of fibres against blast loads when using normal strength concrete. The

shockwave and displacement data can be found in Table 4-1. Complete reflected pressure and impulse histories, as well as reflected pressure and mid-span displacement histories, can be found in Figure 4-12 and Figure 4-13. Photographs detailing the effects of each blast on the specimen are included in Figure 4-14.

Blast 1 is meant to be within the elastic range for control specimens built with conventional materials. The maximum and residual mid-span displacements were measured as 20.7 mm and 3.6 mm respectively. Tensile strain gauges reached an average of 0.00243 mm/mm ($\epsilon_{res} = 0.000508$ mm/mm), which means the steel rebar were close to the yielding point. Hairline cracks were present around the midsection of the specimen; no major crack was revealed.

Blast 2 is meant to be past the yielding point for most control specimens. The maximum and residual mid-span displacements were measured as 48.1 mm and 21.8 mm respectively. Cumulative residual displacement was calculated as 25.4 mm. Tensile strain gauges for this test didn't follow similar magnitudes as one reached 0.0187 mm/mm ($\epsilon_{res} = 0.0148$ mm/mm) and the other 0.00516 mm/mm ($\epsilon_{res} = 0.000238$ mm/mm). While the reliability of these values can be questioned, both show that the steel has yielded. In terms of damage, two major cracks were noted at 1100 mm and 1170 mm from the top support with widths of 1.25 mm and 0.50 mm.

For blast 3, the maximum and residual mid-span displacements were measured as 76.2 mm and 41.2 mm respectively. Cumulative residual displacement adds up to 66.6 mm. Strain gauges were not reliable for this test and the following ones. The main tensile cracks were at 800 mm, 1100 mm and 170 mm from the top support with widths of 1.50 mm, 2.00 mm and 1.00 mm.

For blast 4, the maximum and residual mid-span displacements were measured as 122.1 mm and 69.8 mm. Cumulative residual displacement sums up to 136.4 mm. Vertical cracking occurred at the location of the far-left tensile rebar, exposing the steel on a distance of 360 mm as seen on Figure 4-14(f). Secondary fragmentation at that location was retained by the steel fibres. The principle tensile crack at 1100 mm from the top support expanded to a width varying between 9 mm and 13 mm.

The specimen was considered as failed after blast 5. The maximum and residual displacements were measured as 181.6 mm and 136.0 mm respectively. Cumulative residual displacement reached a total of 272.4 mm. Concrete was lost over all four tensile rebar due to bar slippage, exposing the bars on lengths varying between 100 mm and 540 mm as seen on Figure 4-14(g). The principal crack near mid-span increased to a width varying between 25 mm and 35 mm.

4.4.3. SCC - 1.2%ZP - #3NS - 100

This slab is similar to the previous "SCC-0.6%ZP-#3NS-100" specimen except it has a greater concentration of ZP steel fibres. The purpose behind this specimen is to observe the effects of varying the ratio of fibres in a slab. The shockwave and displacement data can be found in Table 4-2. Complete reflected pressure and impulse histories, as well as reflected

pressure and mid-span displacement histories, can be found in Figure 4-15 and Figure 4-16. Photographs detailing the effects of each blast on the specimen are included in Figure 4-17.

Blast 1 is meant to be within the elastic range for control specimens reinforced with conventional reinforcement steel. The maximum and residual mid-span displacements were measured as 23.9 mm and 3.8 mm respectively. The tensile strain gauges reached an average of 0.00263 mm/mm ($\epsilon_{res} = 0.000481$ mm/mm), which is around the steel's yielding point. Hairline cracks were present around the midsection of the specimen; no major crack was revealed.

Blast 2 is meant to be past the yielding point for most control specimens. The maximum and residual mid-span displacements were measured as 44.2 mm and 21.0 mm respectively. Cumulative residual displacement was calculated as 24.8 mm. Both tensile strain gauges reached a peak invalid value meaning they were damaged during the test. Major cracks were found at 1070 mm and 1250 mm from the top support with a crack width of 0.50 mm and 1.00 mm.

For blast 3, the maximum and residual mid-span displacements were measured as 72.9 mm and 38.1 mm respectively. Cumulative residual displacement adds up to 62.9 mm. Major cracks at 1070 mm and 1250 mm from the top support expanded to a width of 2.50 mm and 2.00 mm.

For blast 4, the maximum and residual mid-span displacements were measured as 105.1 mm and 65.5 mm respectively. Cumulative residual displacement sums up to 128.4 mm. The principal crack at 1070 mm from the top support reached a width of 12.5 mm.

The specimen has failed in flexure during blast 5. The maximum and residual displacements were measured as 155.9 mm and 98.3 mm respectively. Cumulative residual displacement reached a total of 226.7 mm. The width of the principal crack varied between 25 mm and 45 mm. Cover was lost over multiple tensile rebar as seen on Figure 4-17(g) and Figure 4-17(h). However, it should be noted that no important secondary fragmentation occurred because of the effects of steel fibres.

4.4.4. SCC - 0.6%BP - #3NS - 100

As its predecessor, this slab is similar to "SCC-0.6%ZP-#3NS-100" except for a change in steel fibre type; this specimen uses the BP fibres which have a higher tensile strength and greater aspect ratio than the ZP fibres. The shockwave and displacement data can be found in Table 4-2. Complete reflected pressure and impulse histories, as well as reflected pressure and mid-span displacement histories, can be found in Figure 4-18 and Figure 4-19. Photographs detailing the effects of each blast on the specimen were included in Figure 4-20.

Blast 1 is meant to be within the elastic range for control specimens reinforced with conventional reinforcement steel. The maximum and residual mid-span displacements were measured as 22.6 mm and 4.6 mm respectively. Both tensile strain gauges did not record any data during this test. Hairline cracks appeared as a result of the blast but not major crack was visible.

Blast 2 is meant to be past the yielding point for most control specimens. The maximum and residual mid-span displacements were measured as 52.0 mm and 28.1 mm respectively. Cumulative residual displacement was calculated as 32.7 mm. The tensile strain gauges reached an average of 0.00379 mm/mm ($\epsilon_{\text{res}} = 0.000548$ mm/mm), which means the steel has yielded. A major crack was formed near mid-span with a width of 0.50 mm.

For blast 3, the maximum and residual mid-span displacements were measured as 73.1 mm and 41.8 mm respectively. Cumulative residual displacement sums up to 74.5 mm. The tensile strain gauges reached an average of 0.0182 mm/mm ($\epsilon_{\text{res}} = 0.00752$ mm/mm). The mid-span crack width expanded to 1.75 mm with minor losses of concrete cover.

For blast 4, the maximum and residual mid-span displacements were measured as 99.2 mm and 59.4 mm respectively. Cumulative residual displacement adds up to 133.9 mm. The tensile strain gauges reached an average of 0.00581 mm ($\epsilon_{\text{res}} = -0.000590$ mm/mm), however, this may be unreliable due to the fact that residual strain reaches negative values. The mid-span crack has grown to a width of 12 mm and can be seen on Figure 4-20(f).

The specimen was considered to have failed on blast 5. The maximum and residual mid-span displacements reached 158.6 mm and 95.9 mm respectively. Cumulative residual displacement adds up to a total of 229.8 mm. The mid-span crack expanded to a width of 25 mm and concrete cover was lost over the tensile rebar locations as seen on Figure 4-20(g). There was also noticeable concrete crushing visible when the specimen was removed on Figure 4-20(h).

4.4.5. SCC - 1.2%ZP - #3HS - 100

This slab is similar to “SCC-1.2%ZP-#3-100” except for a change in steel reinforcement; this specimen was built using high strength steel bars instead of regular steel ones. It is meant to show the effects of changing steel type in a normal strength concrete slab with fibres. The shockwave and displacement data can be found in Table 4-2. Complete reflected pressure and impulse histories, as well as reflected pressure and mid-span displacement histories, can be found in Figure 4-21 and Figure 4-22. Photographs detailing the effects of each blast on the specimen were included in Figure 4-23.

Blast 1 is meant to be within the elastic range for control specimens built with conventional materials. The maximum and residual mid-span displacements were measured as 18.3 mm and 2.3 mm respectively. The tensile strain gauges reached an average of 0.00236 mm/mm ($\epsilon_{\text{res}} = 0.000433$ mm/mm), which means the steel rebar did not yield. The only noticeable hairline cracks were due to the construction of the slab.

Blast 2 is meant to be past the yielding point for most control specimens. The maximum and residual mid-span displacements were measured as 38.0 mm and 1.8 mm respectively. Cumulative residual displacement was calculated as 4.1 mm. The tensile strain gauges reached an average of 0.00557 mm/mm ($\epsilon_{\text{res}} = 0.000934$ mm/mm), which means the steel rebar is still

within the elastic range. Hairline cracks have multiplied around the middle section; however, no major crack was visible.

For blast 3, the maximum and residual mid-span displacements were measured as 61.3 mm and 11.3 mm respectively. Cumulative residual displacement sums up to 15.4 mm. The tensile strain gauges reached an average of 0.0107 mm/mm ($\epsilon_{\text{res}} = 0.00419$ mm/mm), which is past the high strength steel's yielding point. Some of the previously mentioned hairline cracks expanded to a width of 0.30 mm with minimal cover losses. Small vertical cracks have begun to appear at the location of the reinforcement bars; bar slippage is to be expected.

For blast 4, the maximum and residual mid-span displacements were measured as 80.9 mm and 20.1 mm respectively. Cumulative residual displacement adds up to 35.5 mm. The tensile strain gauges reached an average of 0.0116 mm/mm. The vertical cracks expanded over the mid-right tensile rebar to a width of 0.40 mm and a length of 1150 mm. Tensile horizontal cracks continued growing to a width of 0.50 mm in the midsection area as seen on Figure 4-23(f). A major crack was detected at 1170 mm from the top support with a width varying between 0.75 mm and 1.50 mm.

It was decided that blast 5 was the final test for this specimen in order to minimize damages to the setup, such as what happened with the high strength control "SCC-0%-#3HS-100". The maximum and residual mid-span displacements reached 109.3 mm and 38.3 mm respectively. Cumulative residual displacement was calculated as 73.8 mm. The major crack at 1170 mm from the support expanded to a width varying between 4 mm and 10 mm. Vertical cracking continued over each tensile rebar. The far-left rebar was exposed over 210 mm and a maximum crack width of 42 mm. The two middle rebar were only slightly exposed over 30 mm. The far-right rebar was exposed over a length of 280 mm and a maximum crack width of 33 mm. Refer to Figure 4-23(g) and Figure 4-23(h) for photographs of the damage at blast 5.

4.5. DESCRIPTION OF EXPERIMENTAL RESULTS – UHPFRC SERIES

4.5.1. CRC – 2%B - 6.3NS – 80

This slab is one of two specimens with an 80 mm depth and ultra-high performance fibre reinforced concrete. It is similar to the 80 mm control "SCC-0%-6.3NS-80" except for the use of CRC with fibres. The purpose of this specimen is to observe the effectiveness of high strength fiber reinforced concrete over the conventional normal strength concrete. The shockwave and displacement data can be found in Table 4-2. Complete reflected pressure and impulse histories, as well as reflected pressure and mid-span displacement histories, can be found in Figure 4-24 and Figure 4-25. Photographs detailing the effects of each blast were included in Figure 4-26.

Blast 1 is meant to be within the elastic range for control specimens built with conventional reinforcement steel. The maximum and residual mid-span displacements were measured as 20.3 mm and 6.6 mm respectively. The tensile strain gauges reached an average of 0.00199 mm/mm ($\epsilon_{\text{res}} = 0.000750$ mm/mm), which is within the elastic range for this type of

steel bars. Multiple hairline cracks were visible over the span of the specimen; no major cracks were detected after this test.

Blast 2 is meant to be past the yielding point for most control specimens. The maximum and residual mid-span displacements were measured as 44.8 mm and 18.4 mm respectively. Cumulative residual displacement was calculated as 25.0 mm. The tensile strain gauges reached an average of 0.00349 mm/mm ($\epsilon_{\text{res}} = 0.000934$ mm/mm), which means the steel bars had yet to yield. Major cracks appeared at two locations. The first one was at 770 mm from the top support with a width of 1.00 mm, while the second one was at 1470 mm from the top support with a width of 2.5 mm.

For blast 3, the maximum and residual mid-span displacements were measured as 92.3 mm and 48.0 mm respectively. Cumulative residual displacement sums up to 73.0 mm. The tensile strain gauges were not considered reliable due to the fact that they were both showing negative values, compression. The two previously mentioned major cracks reached a width of 2.00 mm and 11 mm. The location of these cracks change the dynamic shape of the slab during the tests as the middle section becomes a solid body with hinges at the crack locations. Refer to Figure 4-26(e) for clarifications.

Blast 4 was considered as the final test for this specimen as the fibres had pulled out at the critical crack locations and could not contribute further to the slab. The maximum and residual mid-span displacements were measured as 111.2 mm and 48.0 mm. Cumulative residual displacement reached a total of 121.0 mm. The two major cracks expanded to a width of 6 mm and 18 mm as seen on Figure 4-26(f) and Figure 4-26(g). The actual maximum displacement was not located at the mid-span due to the formation of these cracks. Located at the bottom crack, its residual was about 15 mm further than mid-span.

4.5.2. CRC - 2%B - #3NS - 80

This slab is similar to the previous specimen "CRC-2%B-6.3NS-80" except for the change in steel reinforcement from non-deformed 6.3 mm bars to deformed #3 steel rebar. The shockwave and displacement data can be found in Table 4-2. Complete reflected pressure and impulse histories, as well as reflected pressure and mid-span displacement histories, can be found in Figure 4-27 and Figure 4-28. Photographs detailing the effects of each blast are included in Figure 4-29.

Blast 1 is meant to be within the elastic range for control specimens built with conventional materials. The maximum and residual mid-span displacements were measured as 21.4 mm and 7.6 mm respectively. Both strain gauges were not considered to be reliable for this specimen. Cracking was considered as negligible.

Blast 2 is meant to be past the yielding point for most control specimens. The maximum and residual mid-span displacements were measured as 45.3 mm and 13.7 mm respectively. Cumulative residual displacement was calculated as 21.3 mm. Hairline cracking occurred in the

middle section of the specimen. Major cracks were observed at 850 mm and 1070 mm from the top support; both were 0.40 mm wide. Refer to Figure 4-29(e) for crack locations.

For blast 3, the maximum and residual mid-span displacements were measured as 98.2 mm and 53.0 mm respectively. Cumulative residual displacement adds up to 74.3 mm. Three major cracks were observed at 850 mm, 1070 mm and 1620 mm from the top support, each with 0.75 mm, 9 mm and 2 mm crack widths. Refer to Figure 4-29(f) for a photograph of the largest of these cracks.

The specimen was considered as failed after blast 4. The maximum and residual mid-span displacements reached 194.3 mm and 158.7 mm respectively. Cumulative residual displacement totaled for 233.0 mm. Tensile steel rebar ruptured and the principal crack reached a width varying between 22 mm and 28 mm as seen in Figure 4-29(g).

4.5.3. CRC - 2%B - #3NS - 100

This slab is the first of four CRC specimens with a depth of 100 mm. It is similar to “CRC-2%B-#3NS-80” except for the change in depth. It can also be compared to the main control specimen for the effects of UHPFRC and fibres. The shockwave and displacement data can be found in Table 4-3. Complete reflected pressure and impulse histories, as well as reflected pressure and mid-span displacement histories, can be found in Figure 4-30 and Figure 4-31. Photographs detailing the effects of each blast are included in Figure 4-32.

Blast 1 is meant to be within the elastic range for conventional specimens. The maximum and residual mid-span displacements were measured as 15.9 mm and 1.3 mm respectively. The functional tensile strain gauge reached 0.00222 mm/mm ($\epsilon_{res} = 0.000778$ mm/mm), which is close to the yielding point for regular steel rebar. Cracking was considered as negligible.

Blast 2 is meant to be past the yielding point for most control specimens. The maximum and residual mid-span displacements were measured as 31.1 mm and 10.3 mm respectively. Cumulative residual displacement was calculated as 11.6 mm. The functional tensile strain gauge reached 0.00312 mm/mm ($\epsilon_{res} = 0.000264$ mm/mm), which is past the yielding point for regular steel rebar. Hairline cracks have propagated on the middle section of the slab. Two major cracks were noted at 920 mm and 1240 mm from the top support with widths of 0.40 mm and 1.00 mm. Note these locations were relative to the right side of the specimen as the cracks were not horizontal; refer to Figure 4-32(f) for clarifications.

For blast 3, no data was recorded for maximum displacements due the LVDT unscrewing during the test. Residual mid-span displacement was estimated at 23.0 mm. Cumulative residual displacement adds up to 34.6 mm. The functional tensile strain gauge reached 0.0150 mm/mm ($\epsilon_{res} = 0.0101$ mm/mm). The two major cracks expanded to widths of 1.25 mm and a variation between 3 mm and 7 mm.

For blast 4, the maximum and residual mid-span displacements were measured as 59.2 mm and 27.3 mm respectively. Cumulative residual displacement sums up to 61.9 mm. The functional tensile strain gauge reached 0.00826 mm/mm ($\epsilon_{\text{res}} = 0.00336$ mm/mm). As seen on Figure 4-32(g), the two major cracks widths expanded to a variation between 3 mm and 6 mm for the one located at 920 mm, and a variation between 5 mm and 9 mm for the second crack.

Blast 5 was the final test for this specimen as the fibres had pulled out at the critical crack locations and could not contribute further to the slab. The maximum and residual mid-span displacements reached 104.8 mm and 72.1 mm respectively. Cumulative residual displacement totalled for 134.0 mm. As seen on Figure 4-32(h), the two major cracks widths expanded to a variation between 8 mm and 15 mm for the one located at 920 mm from the top support, and 12 mm for the second one.

4.5.4. CRC – 2%A - #3NS – 100

Except for a change in fibre type, this specimen is nearly identical to the previous “CRC-2%-#3NS-100” slab, except for the change in fibre type. It can also be compared with the main control specimen to observe the effects of UHPFRC and fibres, as well as to examine of fibre type on UHPFRC slab performance. The shockwave and displacement data can be found in Table 4-3. Complete reflected pressure and impulse histories, as well as reflected pressure and mid-span displacement histories, can be found in Figure 4-33 and Figure 4-34. Photographs detailing the effects of each blast are included in Figure 4-35.

Blast 1 is meant to be within the elastic range for control specimens built with conventional materials. The maximum and residual mid-span displacements were measured as 12.1 mm and 3.3 mm respectively. The tensile strain gauges reached an average of 0.00224 mm/mm ($\epsilon_{\text{res}} = 0.000724$ mm/mm) which is close to the yielding point of regular steel. Few hairline cracks were visible in the middle section of the slab; no major cracks were observed.

Blast 2 is meant to be past the yielding point for most control specimens. The maximum and residual mid-span displacements were measured as 33.9 mm and 12.4 mm respectively. Cumulative residual displacement was calculated as 15.7 mm. The tensile strain gauges reached an average of 0.00744 mm/mm ($\epsilon_{\text{res}} = 0.00425$ mm/mm), which indicates that the steel rebar has yielded. Major tensile cracks were observed at four locations. Two non-horizontal cracks, similar to the previous specimen, were noted at 1090 mm and 1400 mm from the top support with crack widths of 0.30 mm and 0.75 mm. Two other cracks were noted at 1170 mm and 1540 mm from the top support with crack widths of 0.30 mm and 0.75 mm.

For blast 3, the maximum and residual mid-span displacements were measured as 49.7 mm and 19.3 mm respectively. Cumulative residual displacement adds up to 35.0 mm. The strain gauges were not reliable enough to obtain data for this test. A few cracks were observed among the hairline cracks with widths varying between 0.40 mm and 1.00 mm. The two major non-

horizontal cracks located at 1090 mm and 1280 mm from the top support expanded to widths of 1.25 mm and 2.5 mm.

For blast 4, the maximum and residual mid-span displacements were measured as 67.3 mm and 38.4 mm respectively. Cumulative residual displacement sums up to 73.4 mm. The two diagonal cracks mentioned earlier expanded to 1.50 mm for the first one, and a variation between 6 mm and 10 mm for the second one. These cracks can be observed on Figure 4-35(f).

Blast 5 was the final test for this specimen due to bar rupture, as seen on Figure 4-35(h). The maximum and residual mid-span displacements reached 213.1 mm and 137.3 mm respectively. Cumulative residual displacement totalled for 210.7 mm. The principal crack located at 1400 mm from the top support expanded to a width of 33 mm, as shown on Figure 4-35(g).

4.5.5. CRC - 2%A - #3HS - 100

This slab is similar to “CRC-2%A-#3NS-100” except for a change in steel type from normal strength rebar to high strength steel deformed bars. The shockwave and displacement data can be found in Table 4-3. Complete reflected pressure and impulse histories, as well as reflected pressure and mid-span displacement histories, can be found in Figure 4-36 and Figure 4-37. Photographs detailing the effects of each blast are included in Figure 4-38.

Blast 1 is meant to be within the elastic range for most control specimens reinforced with conventional reinforcement steel. The maximum and residual mid-span displacements were measured as 13.0 mm and 2.5 mm respectively. The tensile strain gauges reached an average of 0.00185 mm/mm ($\epsilon_{res} = 0.000664$ mm/mm), which means the high strength steel is still within the elastic region. Hairline cracks were visible on the middle section of the specimen.

Blast 2 is meant to be past the yielding point for most control specimens. The maximum and residual mid-span displacements were measured as 27.8 mm and 2.0 mm respectively. Cumulative residual displacement was calculated as 4.5 mm. The tensile strain gauges reached an average of 0.00421 mm/mm ($\epsilon_{res} = 0.000731$ mm/mm), which is still under yielding for high strength steel. Hairline propagated further on the specimen.

For blast 3, the maximum and residual mid-span displacements were measured as 45.1 mm and 6.5 mm respectively. Cumulative residual displacement adds up to 11.0 mm. The tensile strain gauges reached an average of 0.00770 mm/mm ($\epsilon_{res} = 0.00217$ mm/mm), which is close to the yielding point for high strength steel. A major tensile crack was observed at 1050 mm from the top support with a width of 0.40 mm.

For blast 4, the maximum and residual mid-span displacements were measured as 59.4 mm and 12.1 mm respectively. Cumulative residual displacement sums up to 23.1 mm. The tensile strain gauges reached an average of 0.0148 mm/mm ($\epsilon_{res} = 0.00351$ mm/mm), which is past the yielding point of high strength steel. The hairline crack zone expanded from the middle section.

Major cracks were found at 960 mm, 1090 mm, 1050 mm and 1140 mm from the top support with widths of 0.40 mm, 0.50 mm, 0.75 mm and 0.40 mm respectively.

For blast 5, the maximum and residual mid-span displacements were measured as 71.9 mm and 14.7 mm respectively. Cumulative residual displacement was calculated as 37.8 mm. The strain gauges were not reliable enough to obtain data for this test. The hairline crack zone has intensified as seen on Figure 4-38(g). Major cracks at 960 mm, 1090 mm, 1050 mm and 1140 mm from the top support expanded to widths of 1.25 mm, 1.25 mm, 2.00 mm and 1.00 mm respectively.

Blast 6 was considered as the final test for this specimen as the rebar ruptured; this failure can be observed in Figure 4-38(i). The maximum and residual mid-span displacements were measured as 128.5 mm and 66.5 mm respectively. Cumulative residual displacement totalled for 104.3 mm. Located at 1050 mm from the top support, the principle tensile crack shown on Figure 4-38(h) reached a width varying between 7 mm and 10 mm.

4.5.6. CRC - 2%A - #3SS - 100

This slab is similar to “CRC-2%A-#3NS-100” and “CRC-2%A-#3HS-100” except it was designed with high ductility stainless steel reinforcement bars. This stainless steel reinforcement has high strain capacity (rupture strain in the range of 30-40%); therefore, this test aimed at examining the potential of using this reinforcement type to delay rupture of tension rebar in UHPFRC slabs. The shockwave and displacement data can be found in Table 4-3. Complete reflected pressure and impulse histories, as well as reflected pressure and mid-span displacement histories, can be found in Figure 4-39 and Figure 4-40. Photographs detailing the effects of each blast are included in Figure 4-41.

Blast 1 is meant to be within the elastic range for control specimens with conventional steel reinforcement. The maximum and residual mid-span displacements were measured as 14.5 mm and 4.0 mm respectively. The tensile strain gauges reached an average of 0.00233 mm/mm ($\epsilon_{res} = 0.000789$ mm/mm), which means the stainless steel rebar is within the elastic range. Hairline cracks were visible on the middle section of the specimen; no major crack was observed.

Blast 2 is meant to be past the yielding point for most control specimens. The maximum and residual mid-span displacements were measured as 30.2 mm and 4.1 mm respectively. Cumulative residual displacement was calculated as 8.1 mm. The tensile strain gauges reached an average of 0.00603 mm/mm ($\epsilon_{res} = 0.00189$ mm/mm), which is still under the yielding point for stainless steel rebar. No major cracks were formed during this test.

For blast 3, the maximum and residual mid-span displacements were measured as 45.8 mm and 8.9 mm respectively. Cumulative residual displacement adds up to 17.0 mm. The tensile strain gauges reached an average of 0.0132 mm/mm ($\epsilon_{res} = 0.00563$ mm/mm), which means the stainless steel has yielded. The hairline crack range increased from the previous blast. Major

tensile cracks were observed at 760 mm, 1040 mm, 1080 mm and 1500 mm from the top support with widths of 0.30 mm, 0.30 mm, 0.40 mm and 0.30 mm respectively.

For blast 4, the maximum and residual mid-span displacements were measured as 59.9 mm and 10.8 mm respectively. Cumulative residual displacement sums up to 27.8 mm. The strain gauges were not reliable enough to obtain data for this test. The previously observed major tensile cracks expanded. Observed at 760 mm, 1040 mm, 1080 mm and 1500 mm from the top support, their widths increased to 0.75 mm, 0.50 mm, 1.00 mm and 0.50 mm respectively.

For blast 5, the maximum and residual mid-span displacements were measured as 81.8 mm and 25.3 mm respectively. Cumulative residual displacement reached 53.1 mm. The major tensile cracks at 760 mm, 1040 mm, 1080 mm and 1500 mm from the top support increased in width to 0.75 mm, 1.50 mm, 4 mm and 1.50 mm respectively. The cracking pattern can be seen on Figure 4-41(g).

Blast 6 was the final test for this specimen. The maximum and residual mid-span displacements reached 119.1 mm and 67.6 mm. Cumulative residual displacement totaled for 120.7 mm. The major tensile cracks at 760 mm, 1040 mm, 1080 mm and 1500 mm from the top support expanded to 1.50 mm, 1.50 mm, 13 mm and 3 mm respectively. The largest of these cracks can be seen on Figure 4-41(h) and Figure 4-41(i). It was noted that the high ductility stainless steel rebar did not rupture but the steel fibres would have no effect in resisting another blast due to the dimensions of the crack opening.

4.6. PRESSURE, IMPULSE, AND DISPLACEMENT TIME HISTORIES & SELECTED PHOTOGRAPHS

4.6.1. SCC - 0% - 6.3NS - 80

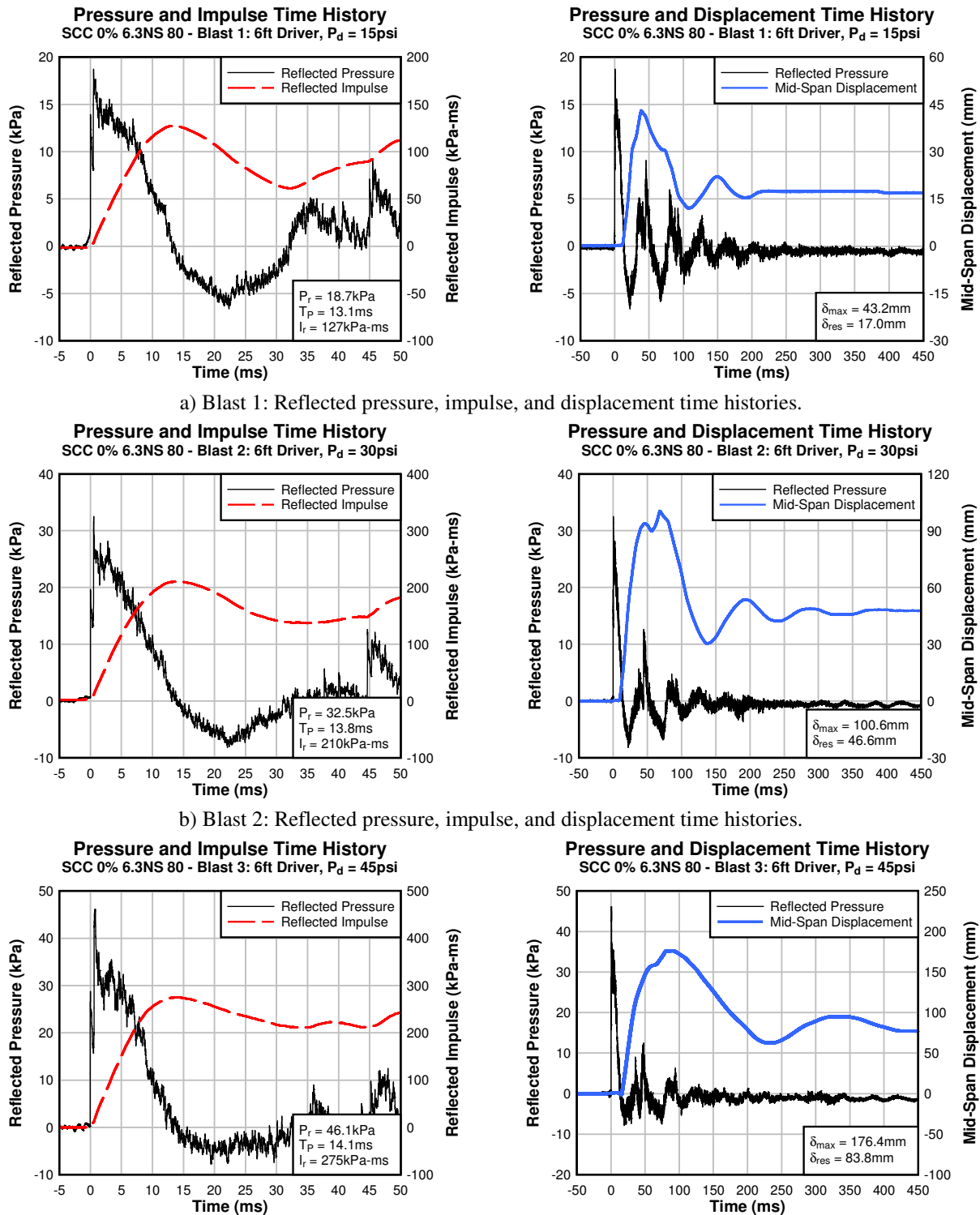
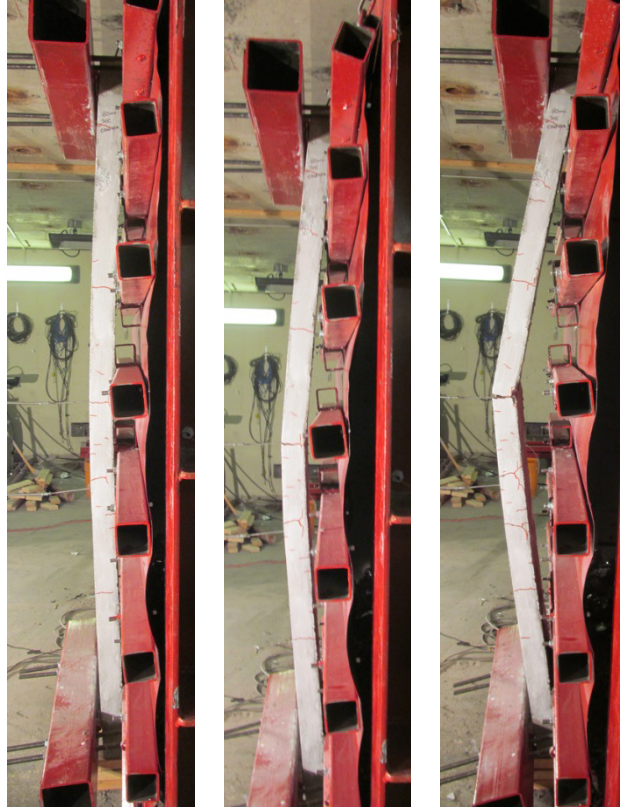


Figure 4-1 SCC-0%-6.3NS-80, recorded reflected pressure, impulse and displacement for Blast 1-3



a) Blast 1

b) Blast 2

c) Blast 3



d) Mid-span cracking at Blast 2



e) Mid-span cracking at Blast 3



f) Mid-span crushing at Blast 3

Figure 4-2 SCC-0% -6.3NS-80, photographs at the end of Blast 1-3

4.6.2. SCC - 0% - #3NS - 100

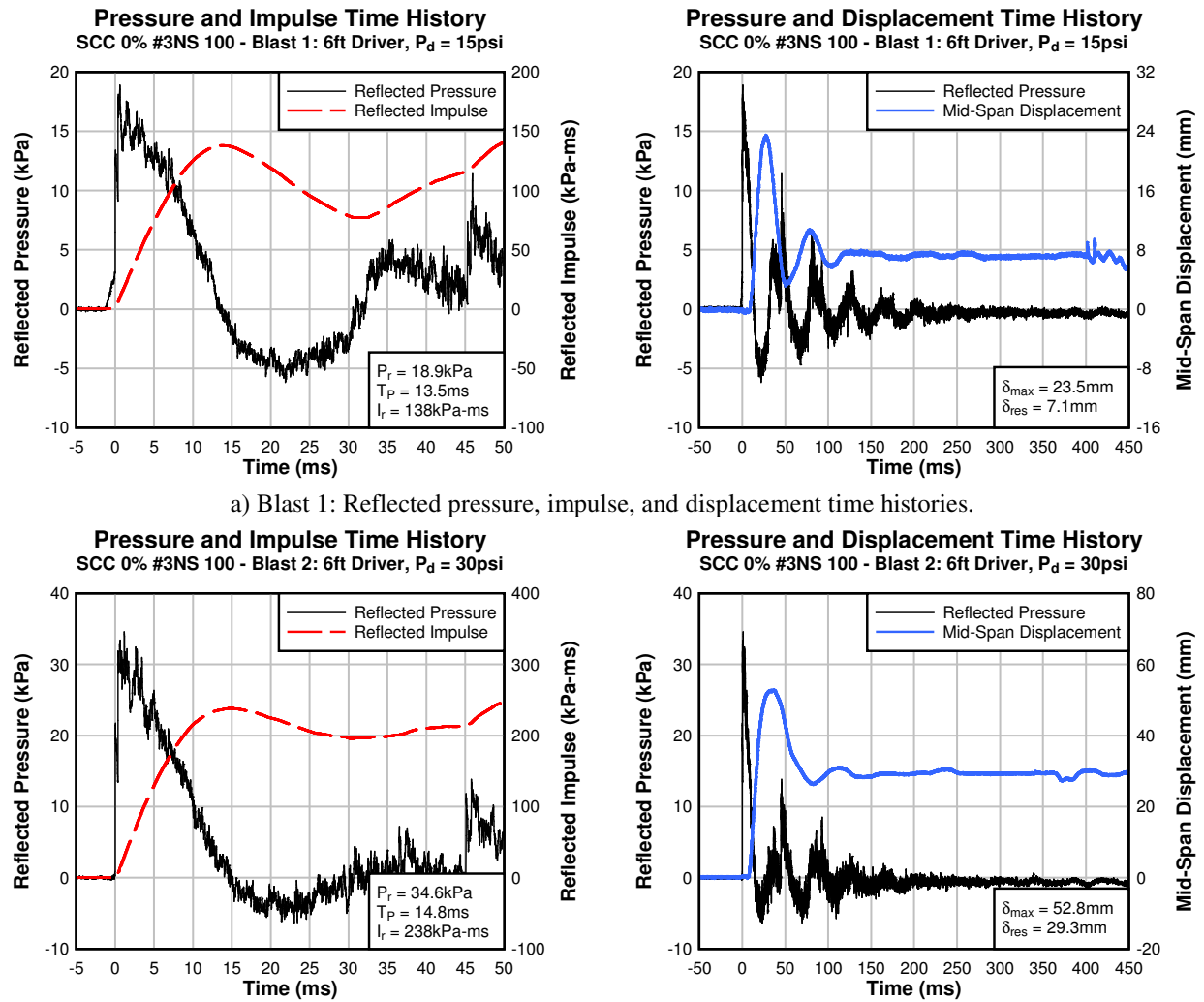
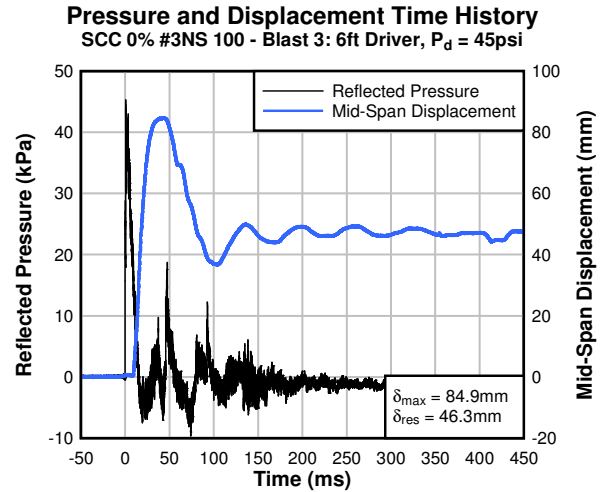
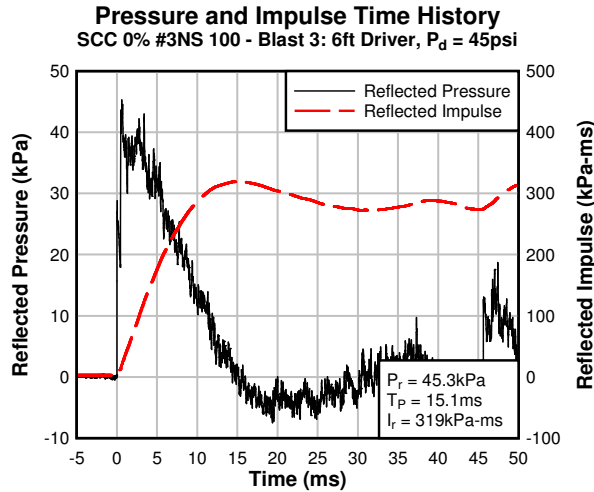
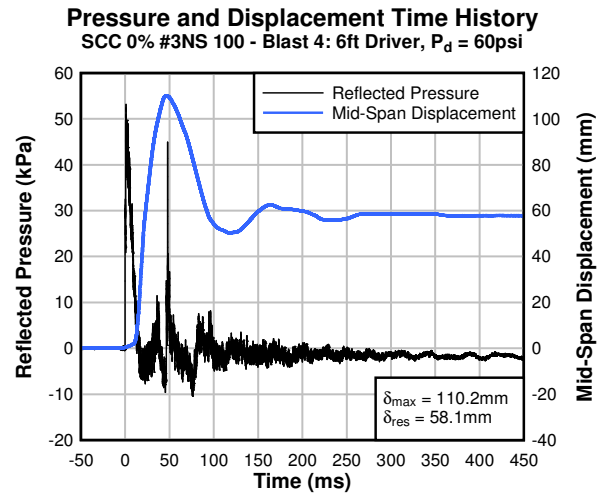
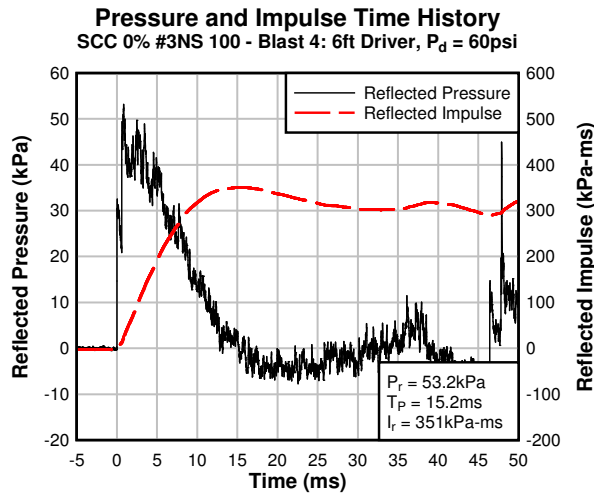


Figure 4-3 SCC-0%-#3NS-100, recorded reflected pressure, impulse and displacement for Blast 1-2

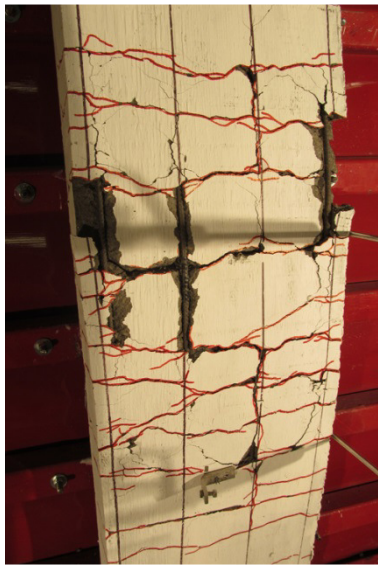
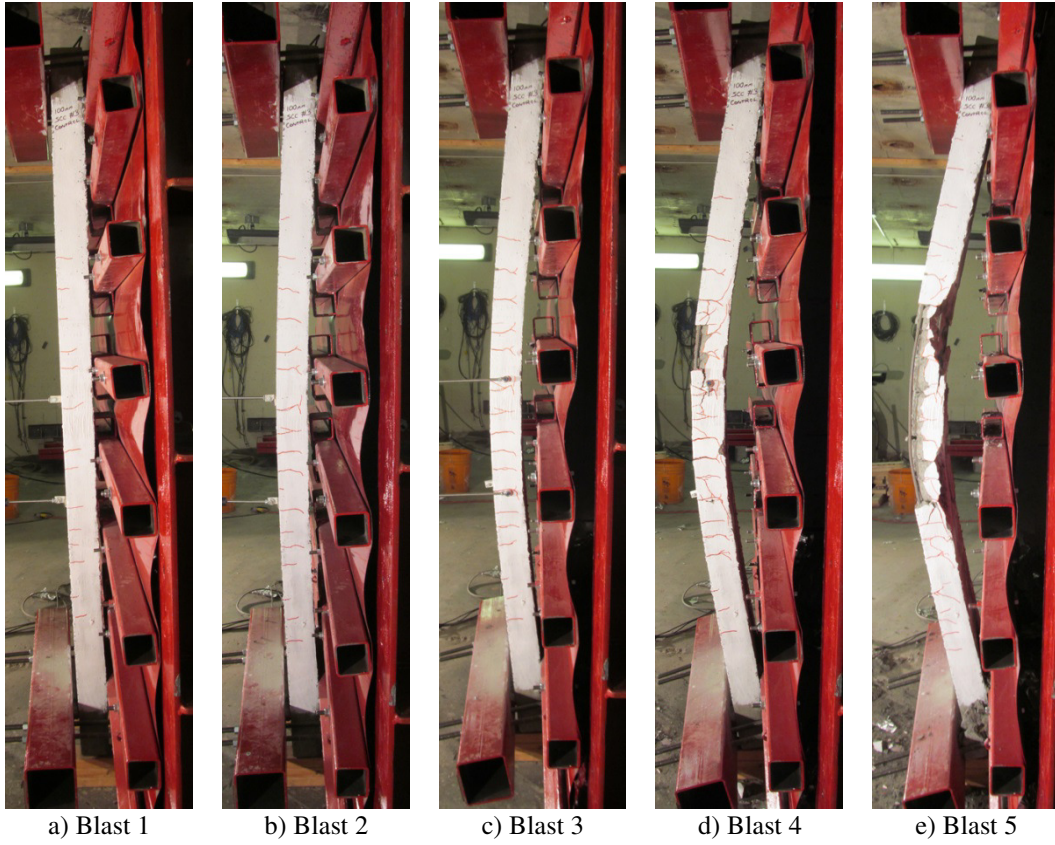


a) Blast 3: Reflected pressure, impulse, and displacement time histories.



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

Figure 4-4 SCC-0%-#3NS-100, recorded reflected pressure, impulse and displacement for Blast 3-4



f) Mid-span cracking at Blast 4



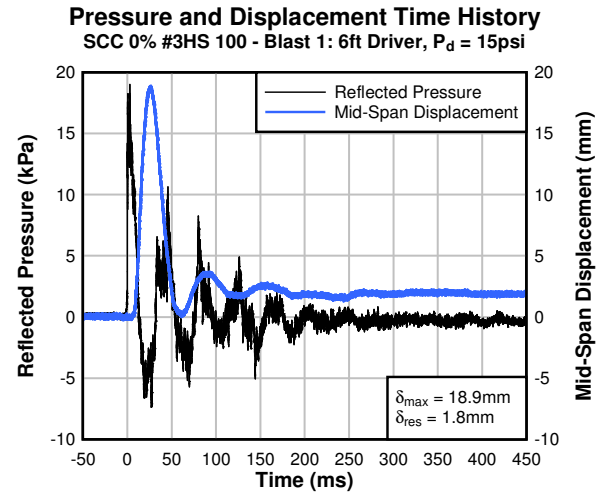
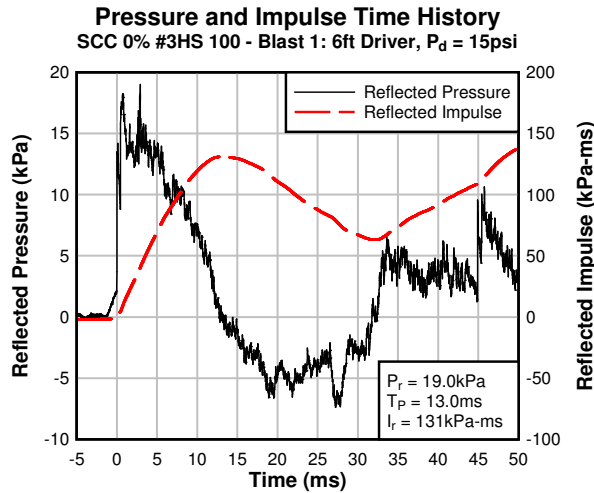
g) Bar slippage and fragmentation at Blast 5



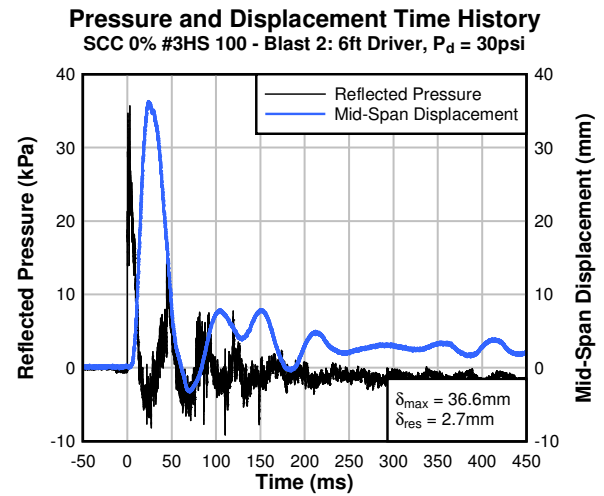
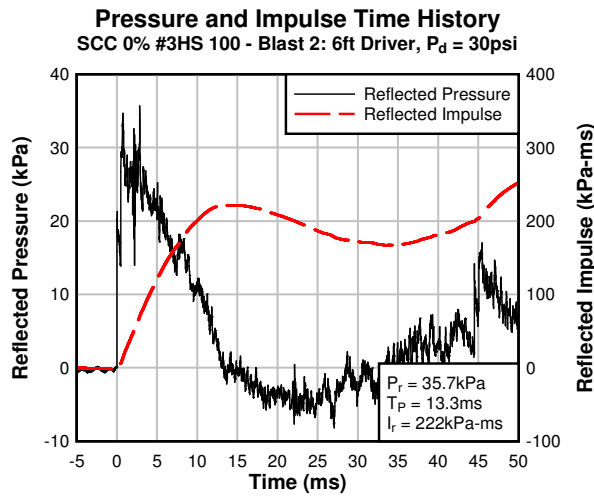
h) Exposed compression steel at Blast 5

Figure 4-5 SCC-0%-#3NS-100, photographs at the end of Blast 1-5

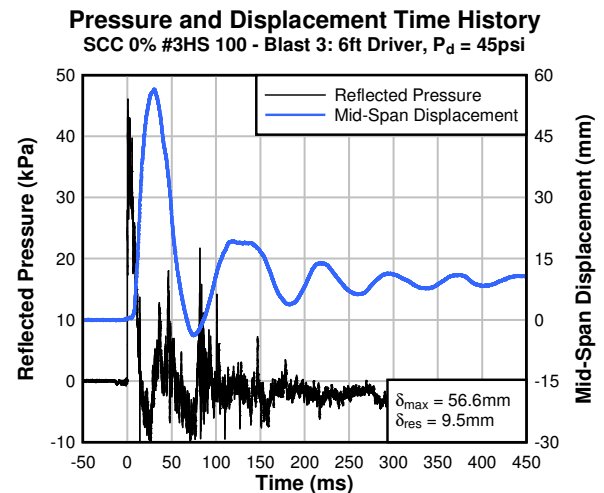
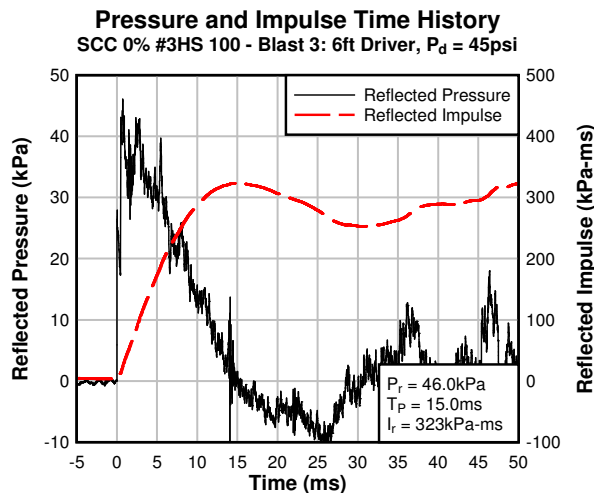
4.6.3. SCC - 0% - #3HS - 100



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

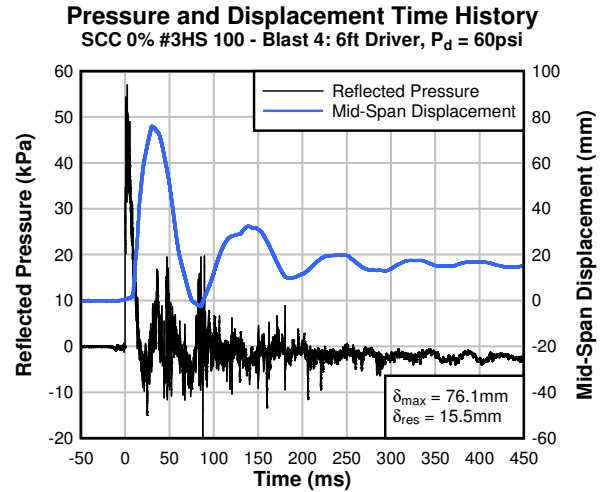
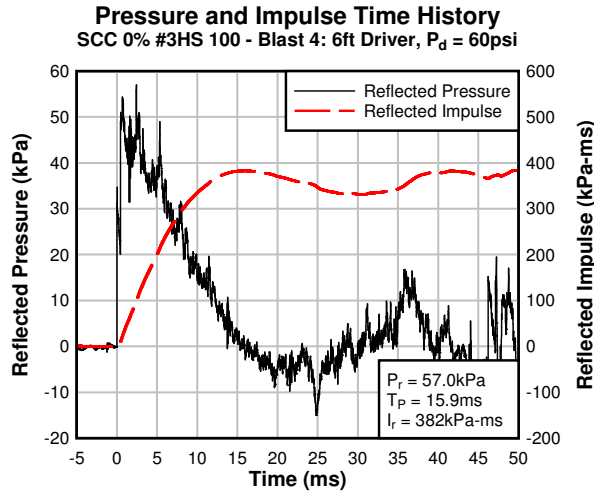


b) Blast 2: Reflected pressure, impulse, and displacement time histories.

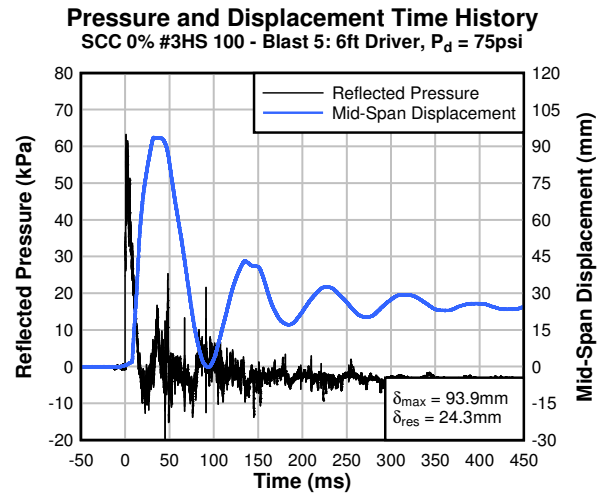
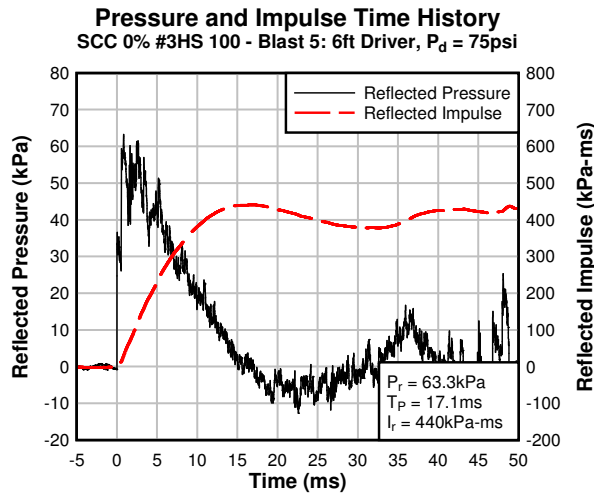


c) Blast 3: Reflected pressure, impulse, and displacement time histories.

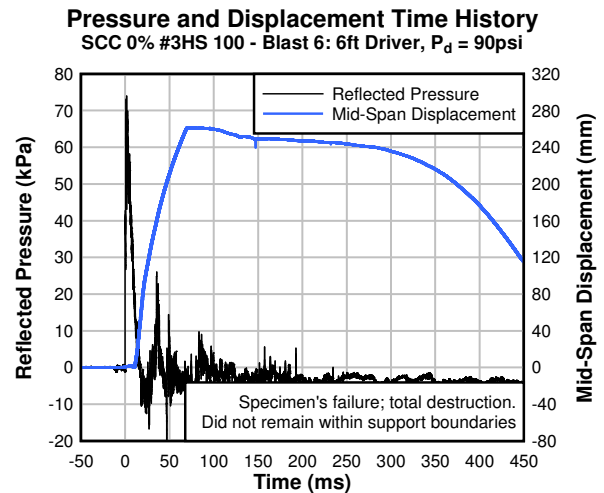
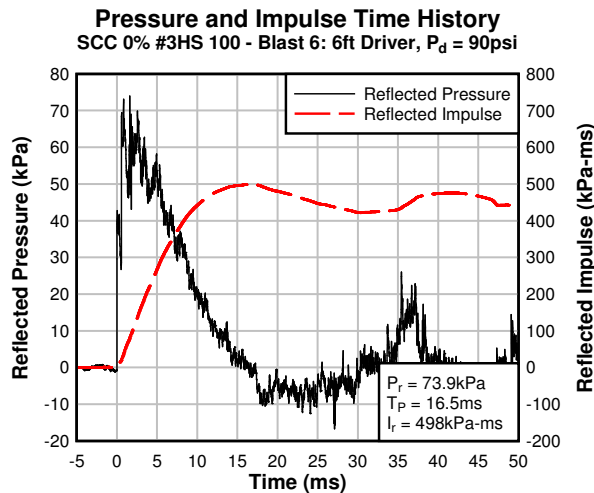
Figure 4-6 SCC-0% - #3HS-100, recorded reflected pressure, impulse and displacement for Blast 1-3



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



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



c) Blast 6: Reflected pressure, impulse, and displacement time histories.

Figure 4-7 SCC-0% #3HS-100, recorded reflected pressure, impulse and displacement for Blast 4-6



a) Blast 1

b) Blast 2

c) Blast 3

d) Blast 4

e) Blast 5

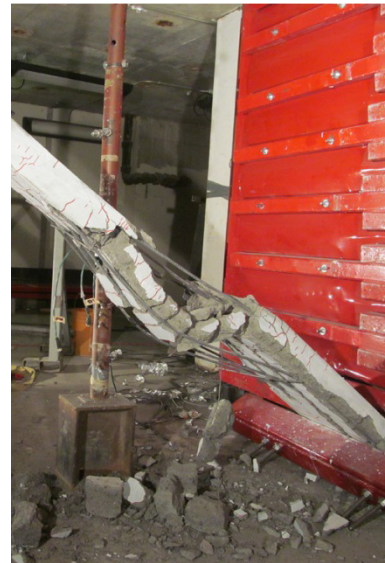
f) Blast 6



g) Vertical cracking at Blast 5



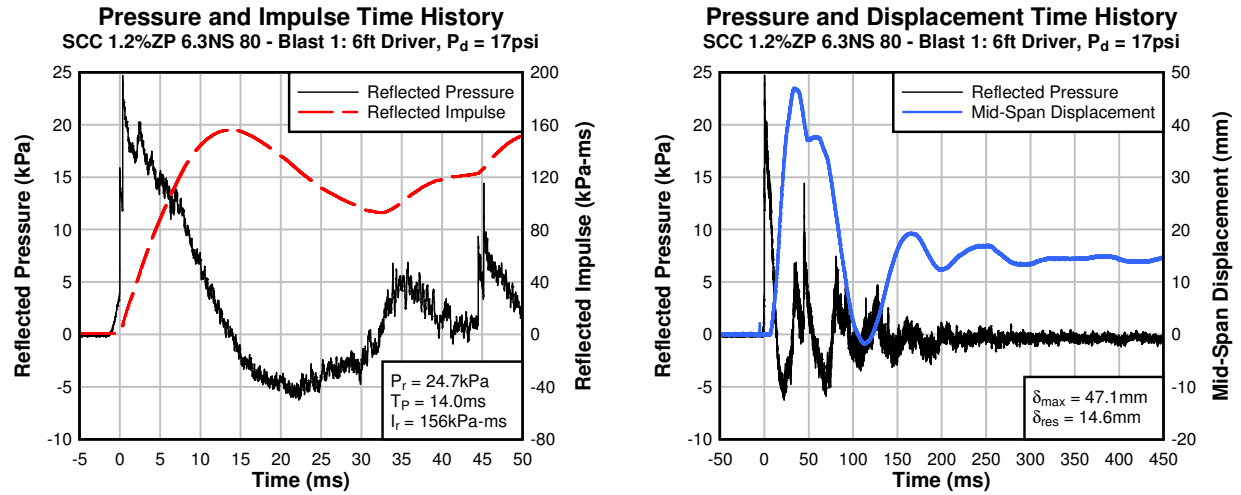
h) Secondary fragmentation during Blast 6



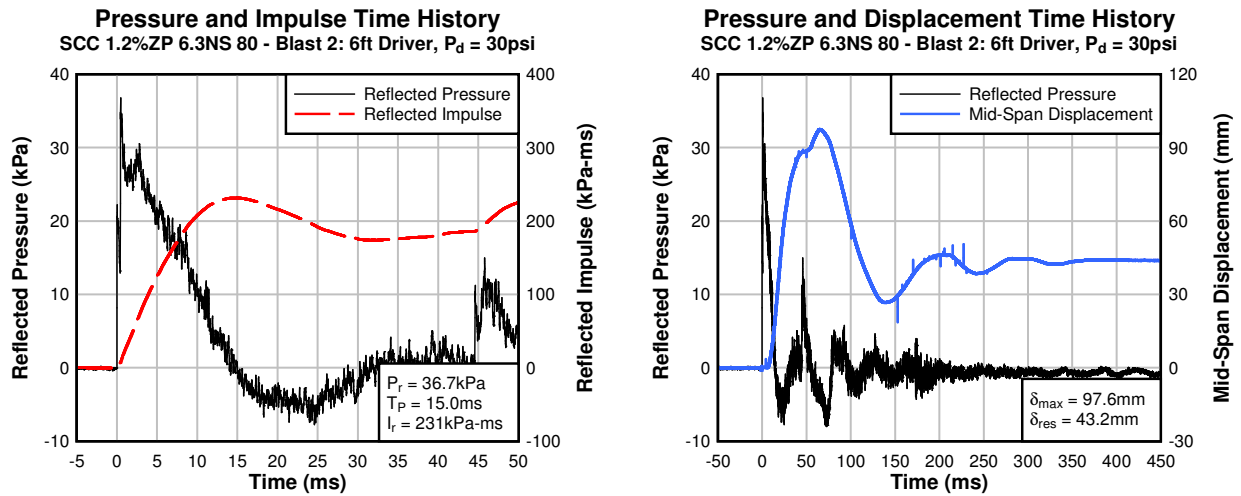
i) Specimen failure at Blast 6

Figure 4-8 SCC-0%-#3HS-100, photographs at the end of Blast 1-6

4.6.4. SCC - 1.2%ZP - 6.3NS - 80

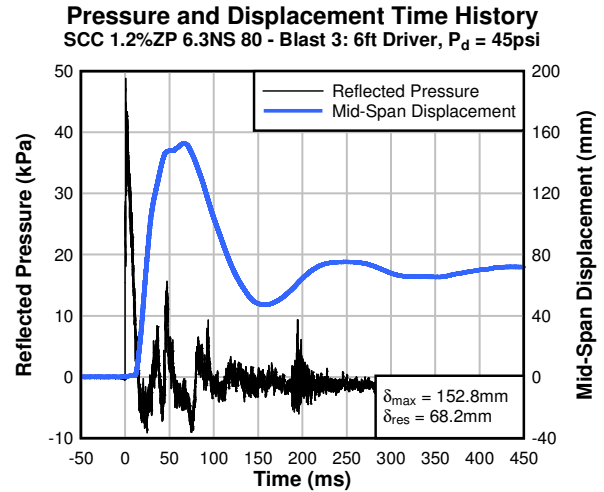
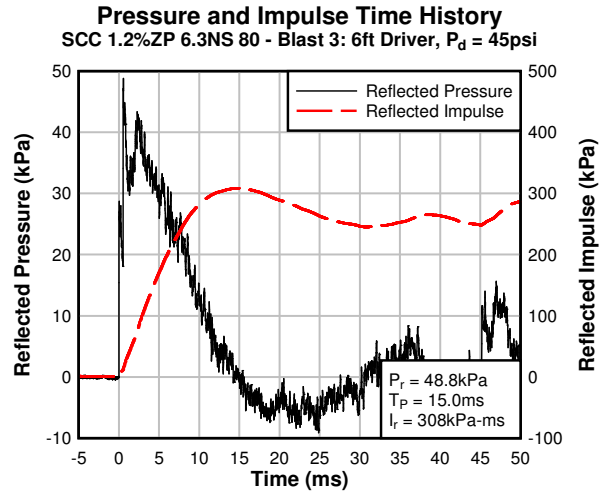


a) Blast 1: Reflected pressure, impulse, and displacement time histories.

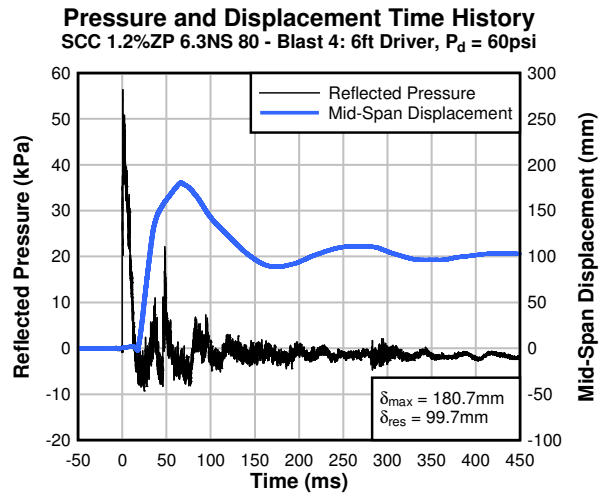
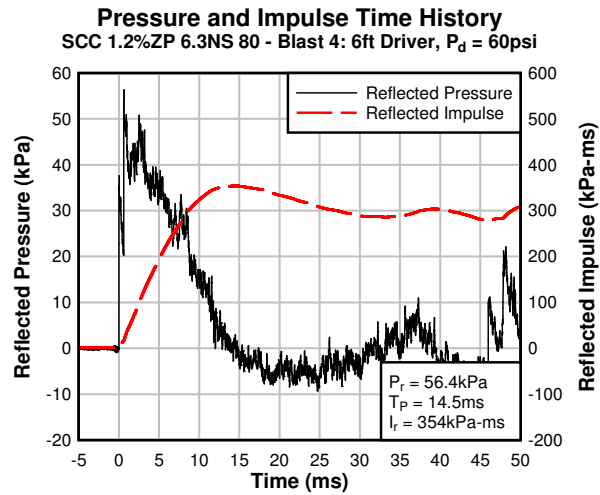


b) Blast 2: Reflected pressure, impulse, and displacement time histories.

Figure 4-9 SCC-1.2% ZP-6.3NS-80, recorded reflected pressure, impulse and displacement for Blast 1-2

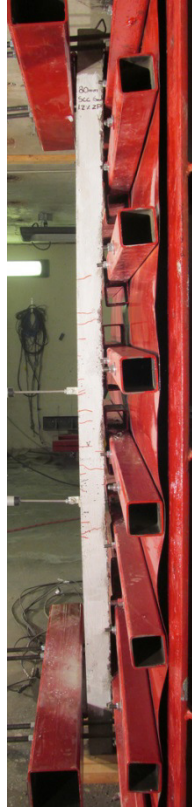


a) Blast 3: Reflected pressure, impulse, and displacement time histories.

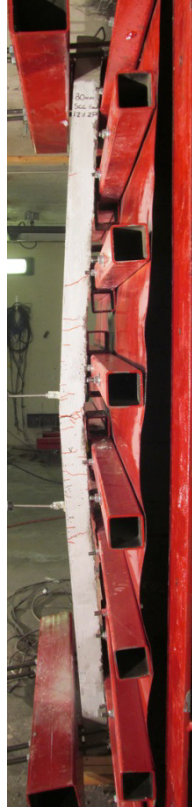


b) Blast 4: Reflected pressure, impulse, and displacement time histories.

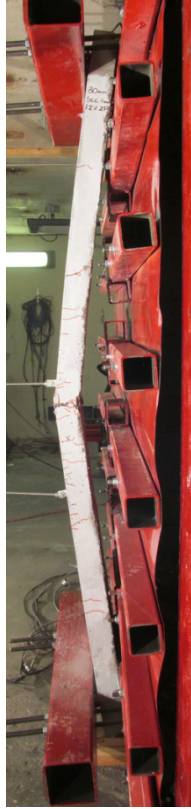
Figure 4-10 SCC-1.2%ZP-6.3NS-80, recorded reflected pressure, impulse and displacement for Blast 3-4



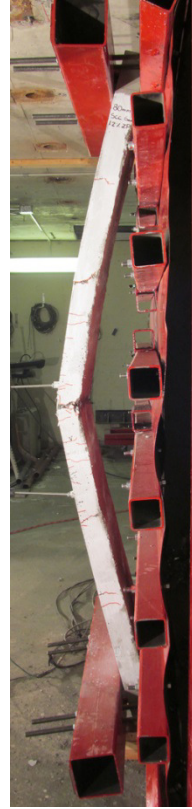
a) Blast 1



b) Blast 2



c) Blast 3



d) Blast 4



e) Mid-span cracking at Blast 3



f) Bar slippage at Blast 4



g) Concrete crushing at Blast 4

Figure 4-11 SCC-1.2% ZP-6.3NS-80, photographs at the end of Blast 1-4

4.6.5. SCC - 0.6%ZP - #3NS - 100

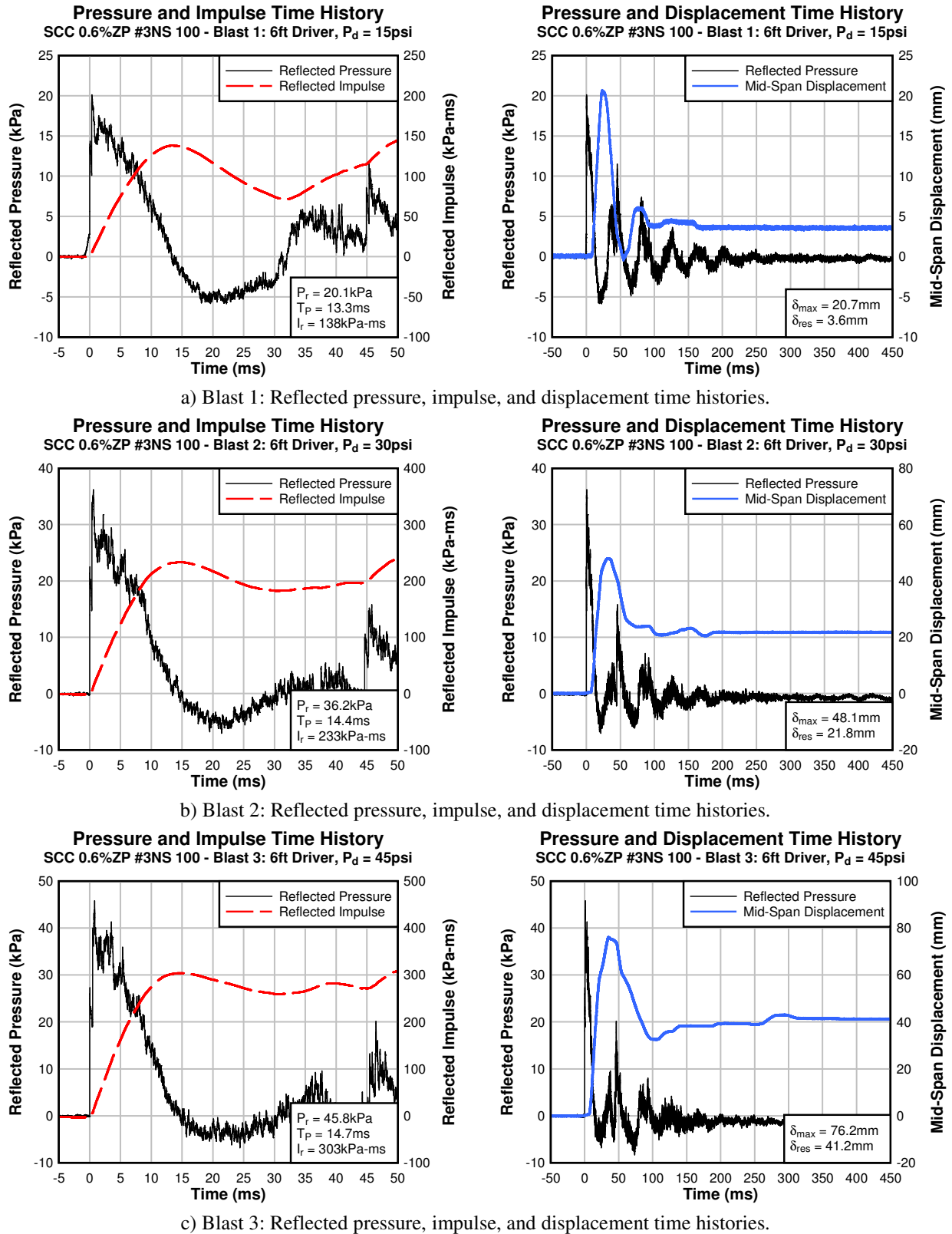
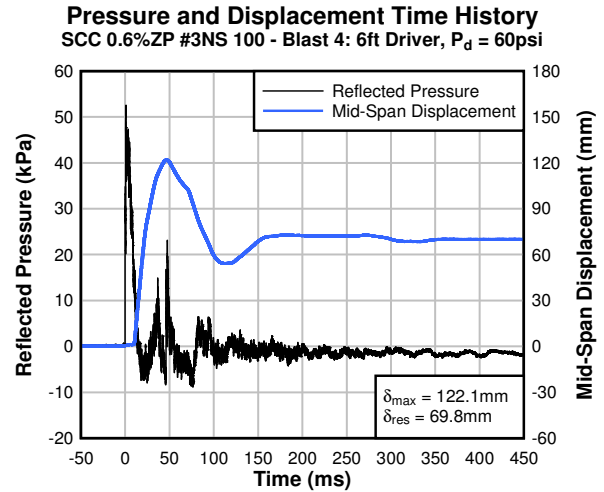
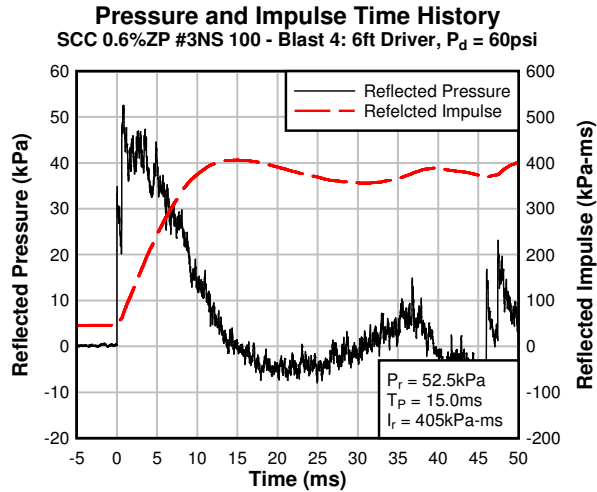
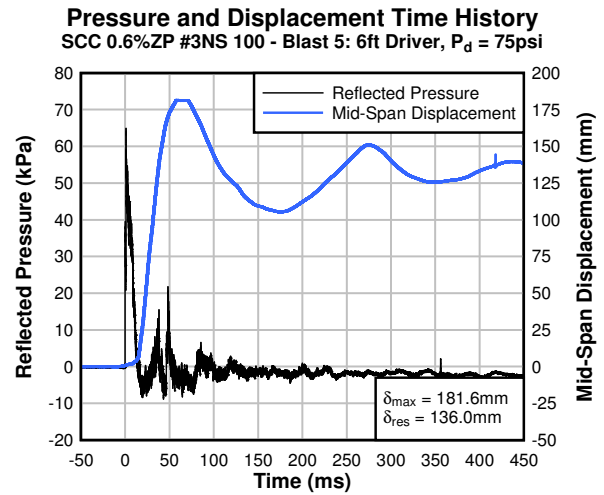
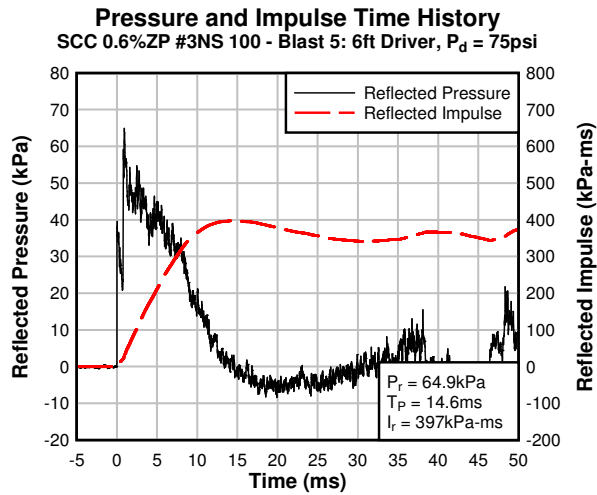


Figure 4-12 SCC-0.6%ZP-#3NS-100, recorded reflected pressure, impulse and displacement for Blast 1-3

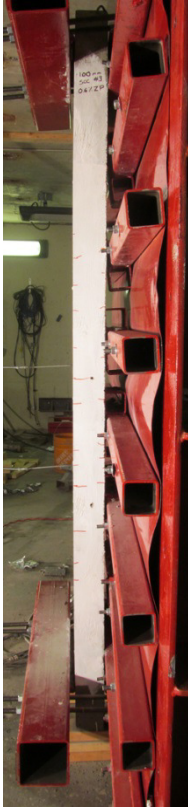


a) Blast 4: Reflected pressure, impulse, and displacement time histories.



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

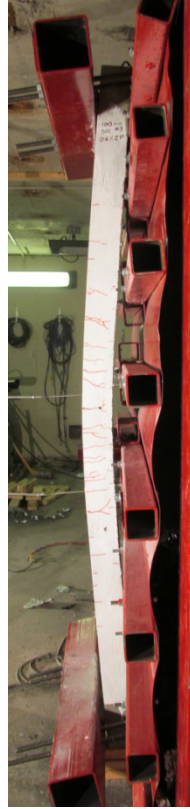
Figure 4-13 SCC-0.6%ZP-#3NS-100, recorded reflected pressure, impulse and displacement for Blast 4-5



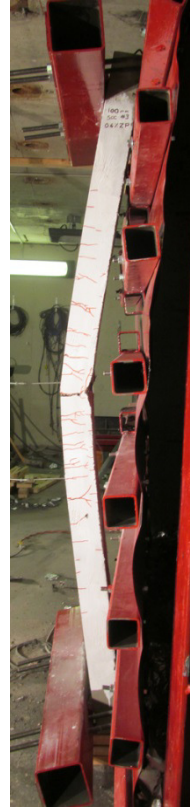
a) Blast 1



b) Blast 2



c) Blast 3



d) Blast 4



e) Blast 5



f) Mid-span cracking at Blast 4



g) Mid-span cracking at Blast 5



h) Concrete crushing at Blast 5

Figure 4-14 SCC-0.6%ZP-#3NS-100, photographs at the end of Blast 1-5

4.6.6. SCC - 1.2%ZP - #3NS - 100

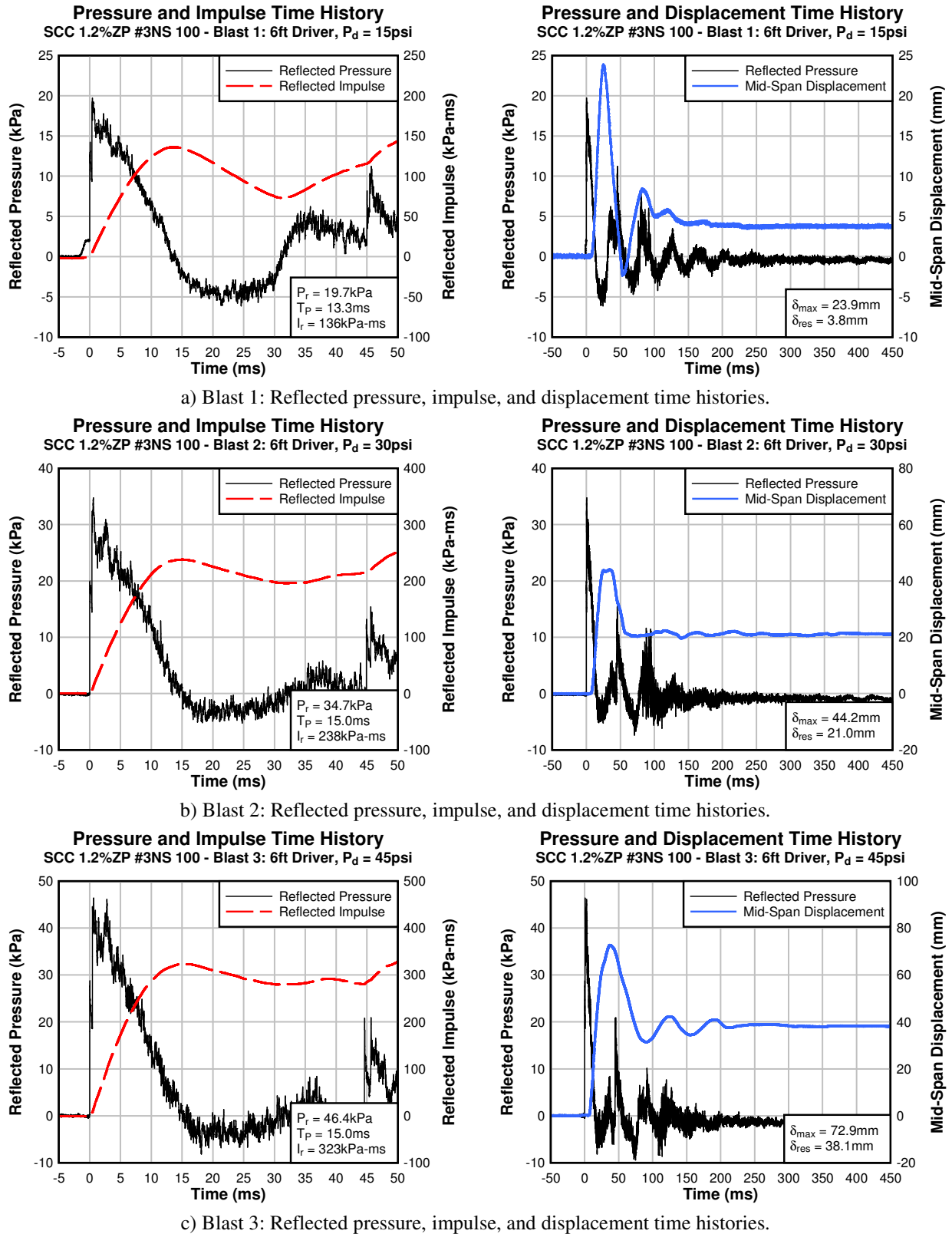
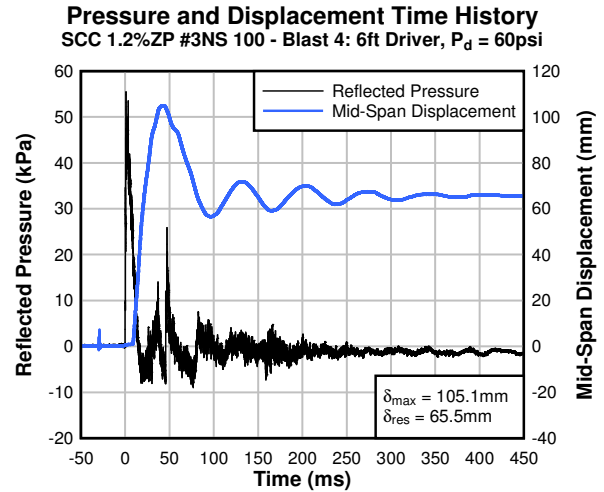
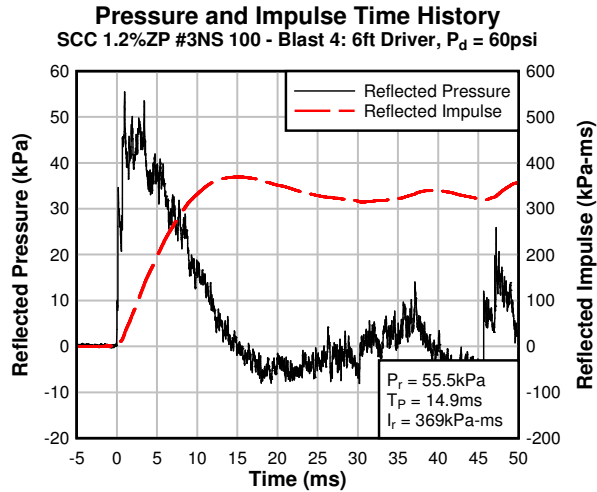
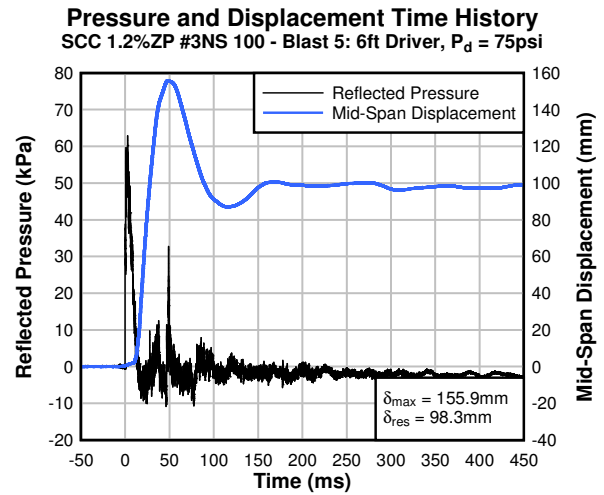
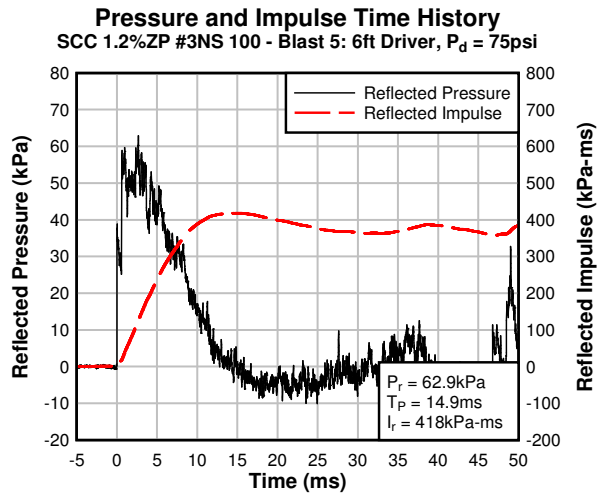


Figure 4-15 SCC-1.2%ZP-#3NS-100, recorded reflected pressure, impulse and displacement for Blast 1-3

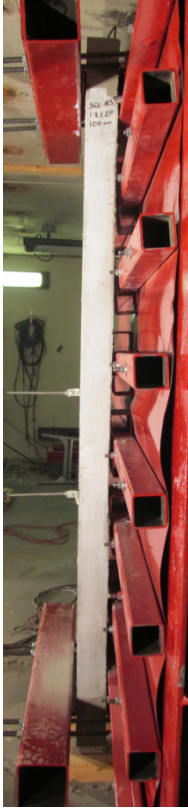


a) Blast 4: Reflected pressure, impulse, and displacement time histories.



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

Figure 4-16 SCC-1.2%ZP-#3NS-100, recorded reflected pressure, impulse and displacement for Blast 4-5



a) Blast 1



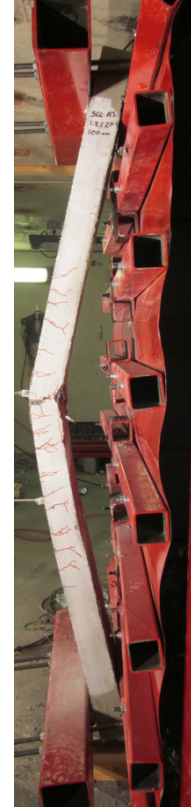
b) Blast 2



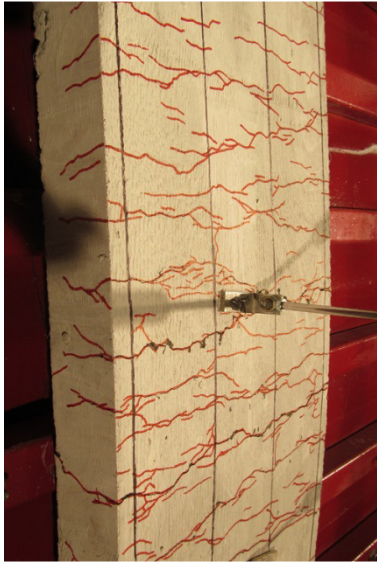
c) Blast 3



d) Blast 4



e) Blast 5



f) Hairline cracks at Blast 3



g) Mid-span cracking at Blast 5



h) Exposed rebar at Blast 5

Figure 4-17 SCC-1.2%ZP-#3NS-100, photographs at the end of Blast 1-5

4.6.7. SCC - 0.6%BP - #3NS - 100

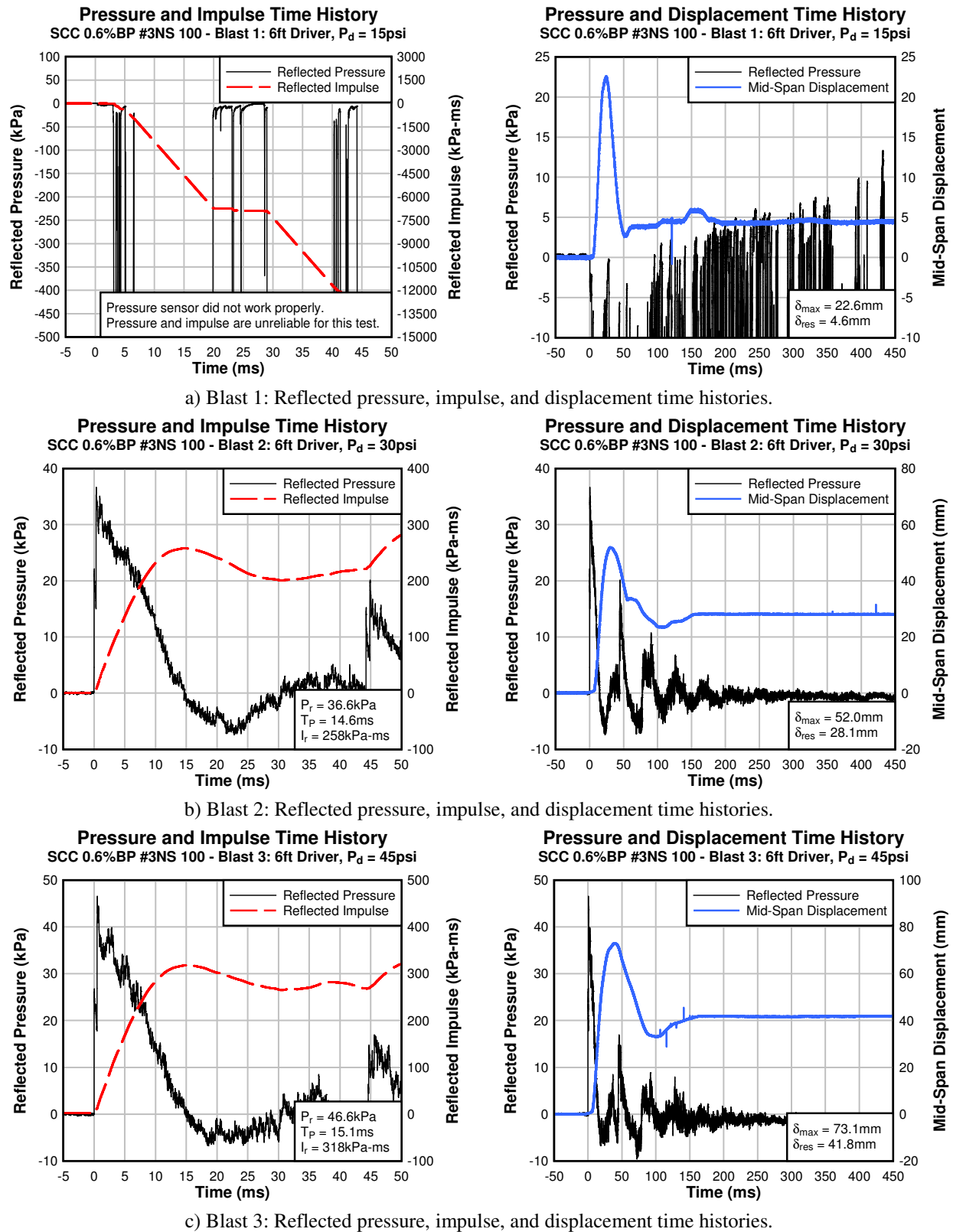
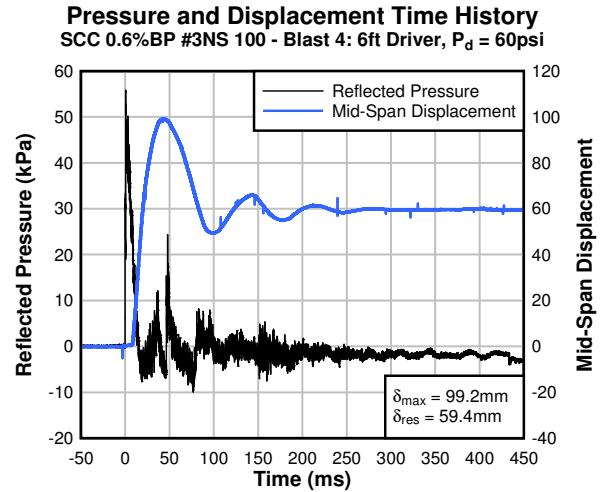
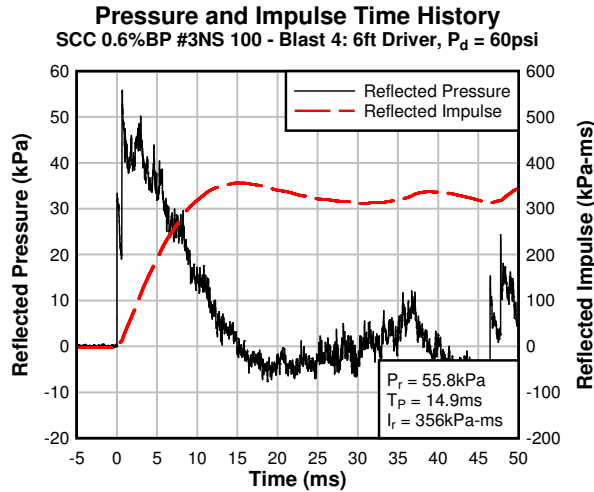
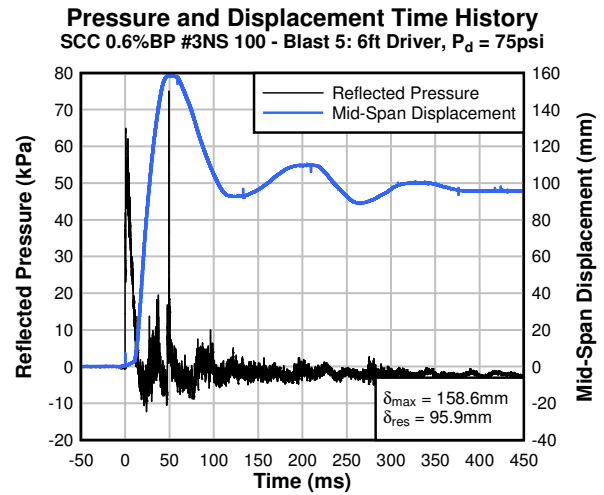
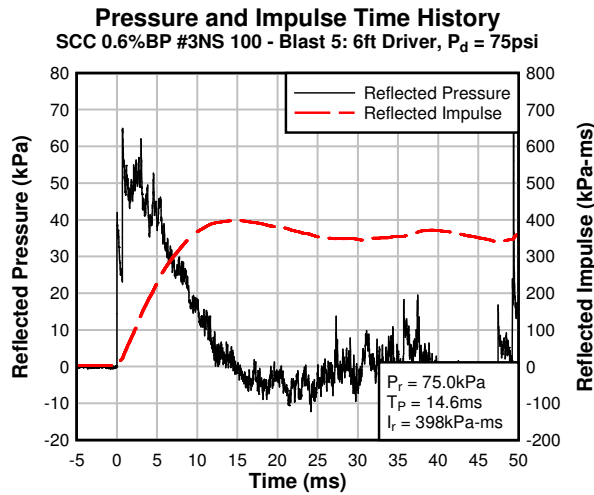


Figure 4-18 SCC-0.6%BP-#3NS-100, recorded reflected pressure, impulse and displacement for Blast 1-3



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



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

Figure 4-19 SCC-0.6%BP-#3NS-100, recorded reflected pressure, impulse and displacement for Blast 4-5

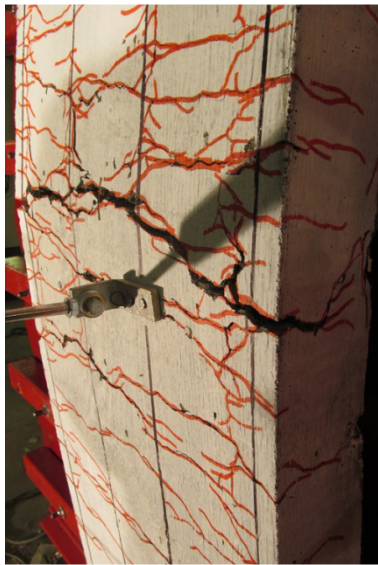
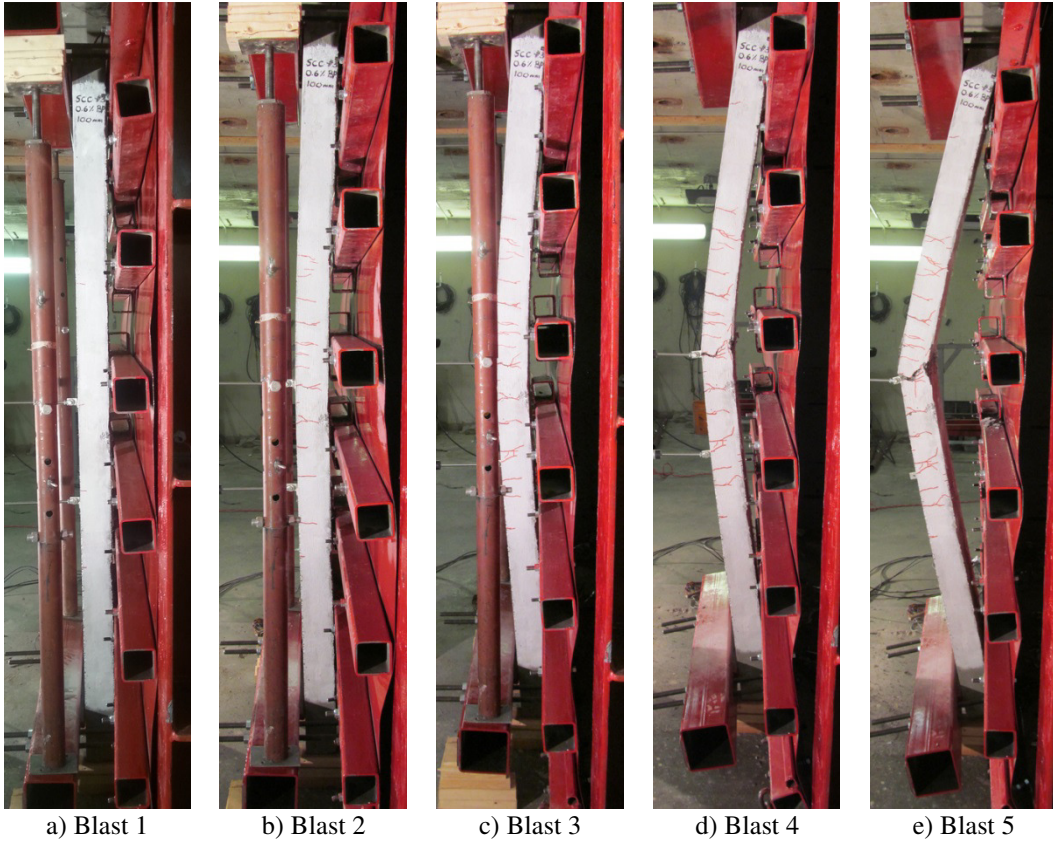


Figure 4-20 SCC-0.6%BP-#3NS-100, photographs at the end of Blast 1-5

4.6.8. SCC - 1.2%ZP - #3HS - 100

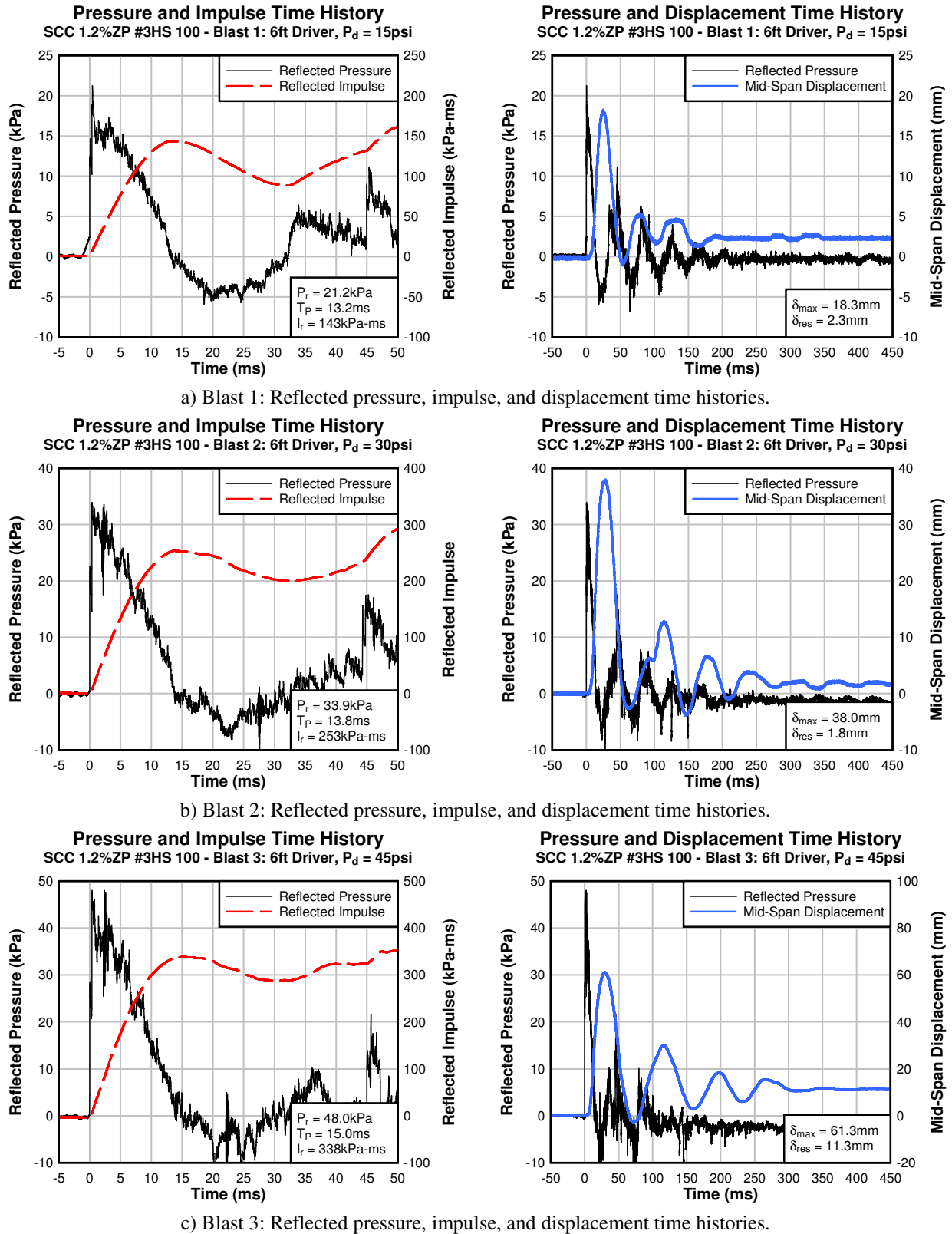
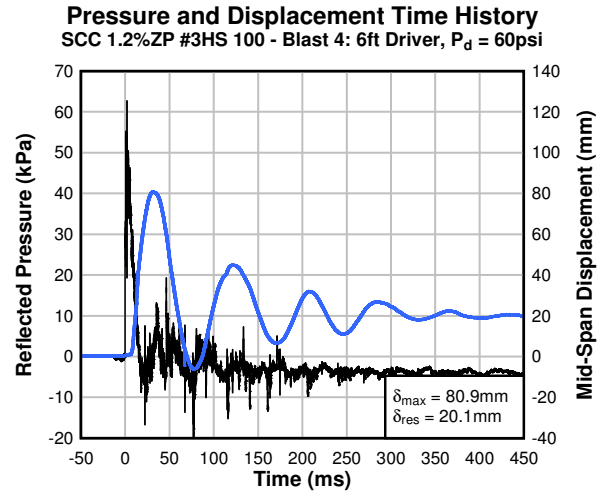
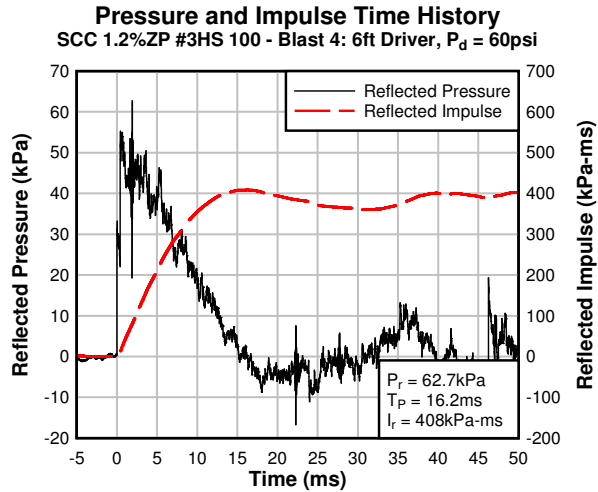
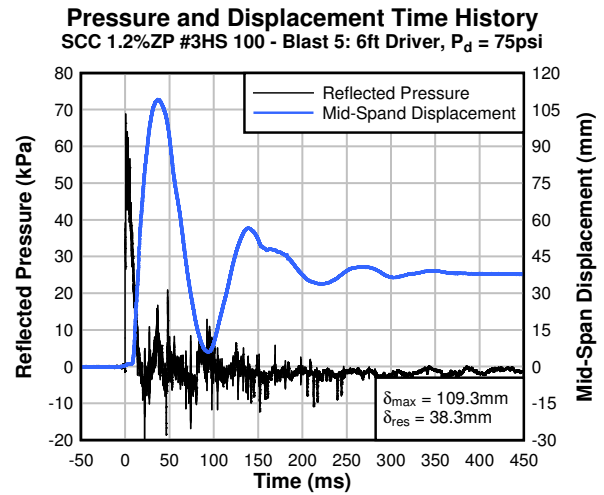
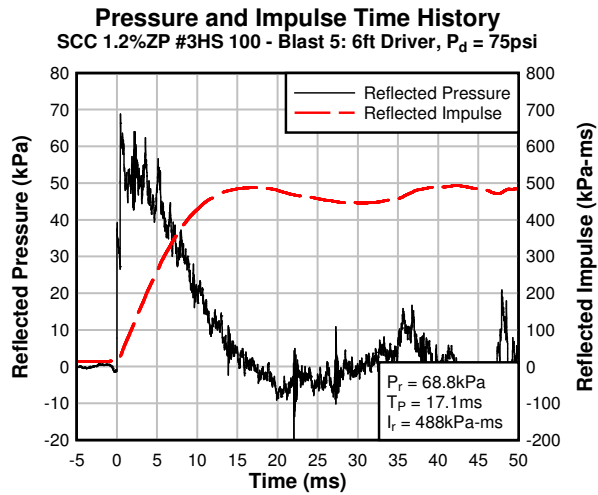


Figure 4-21 SCC-1.2%ZP-#3HS-100, recorded reflected pressure, impulse and displacement for Blast 1-3



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



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

Figure 4-22 SCC-1.2%ZP-#3HS-100, recorded reflected pressure, impulse and displacement for Blast 4-5



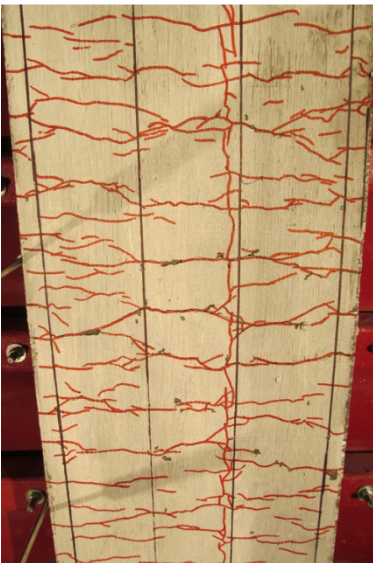
a) Blast 1

b) Blast 2

c) Blast 3

d) Blast 4

e) Blast 5



f) Mid-span cracking at Blast 4



g) Mid-span cracking at Blast 5



h) Mid-span cracking at Blast 5

Figure 4-23 SCC-1.2% ZP-#3HS-100, photographs at the end of Blast 1-5

4.6.9. CRC - 2%B - 6.3NS - 80

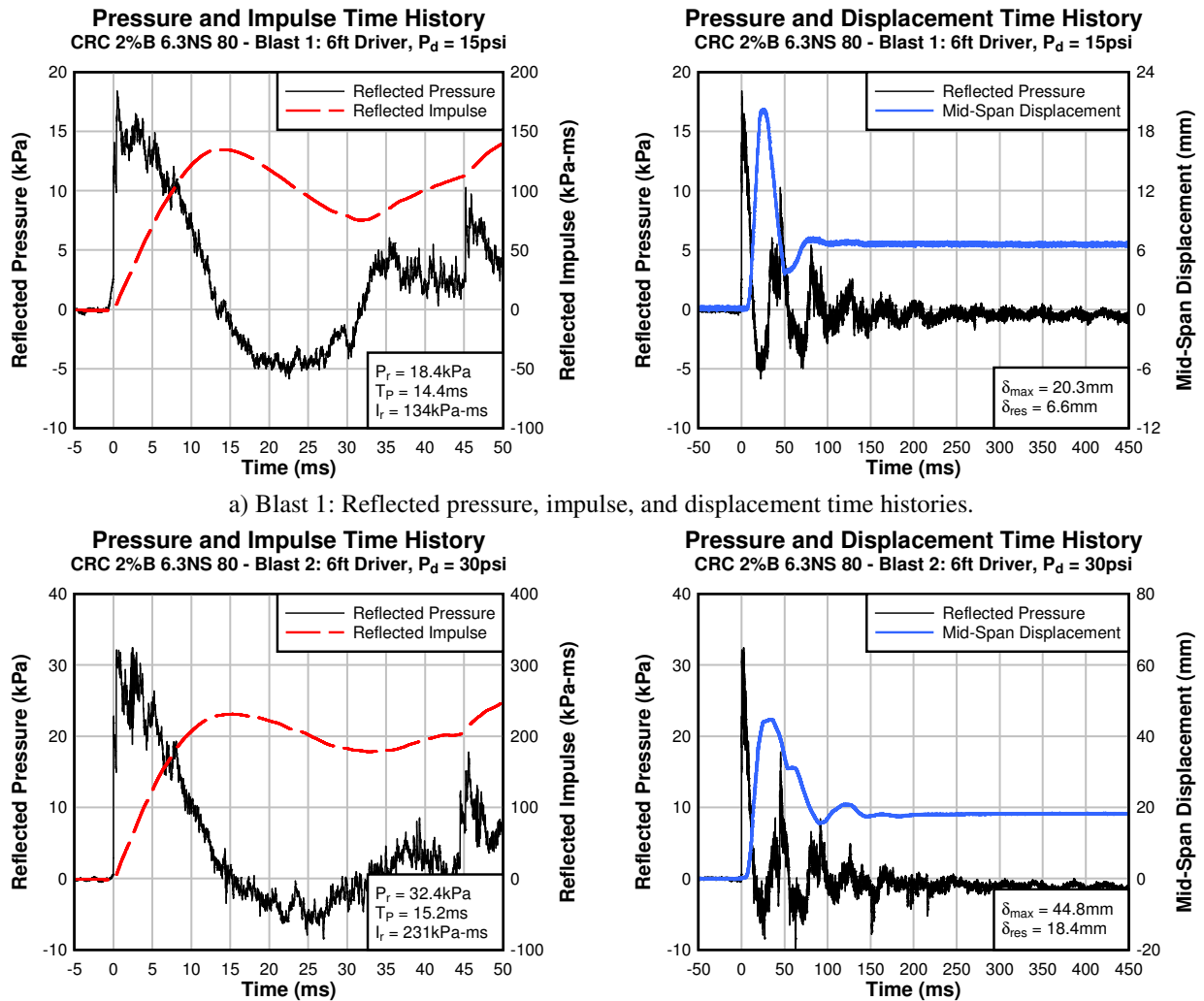
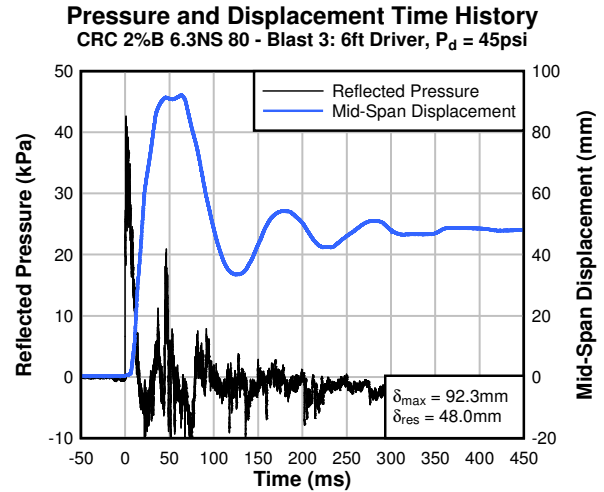
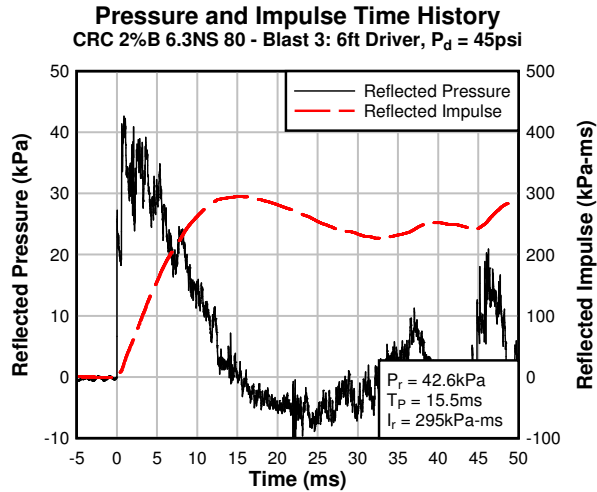
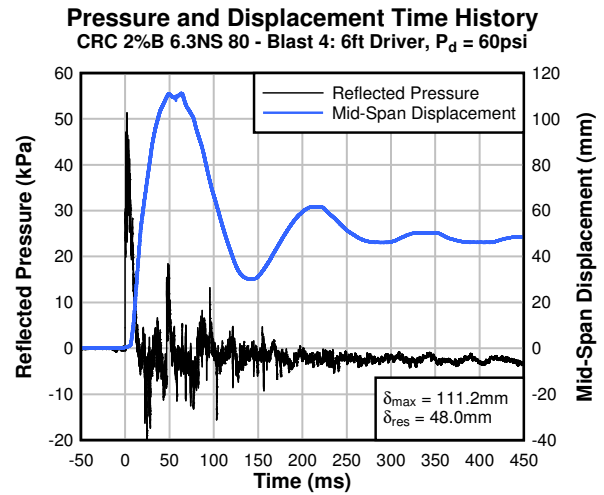
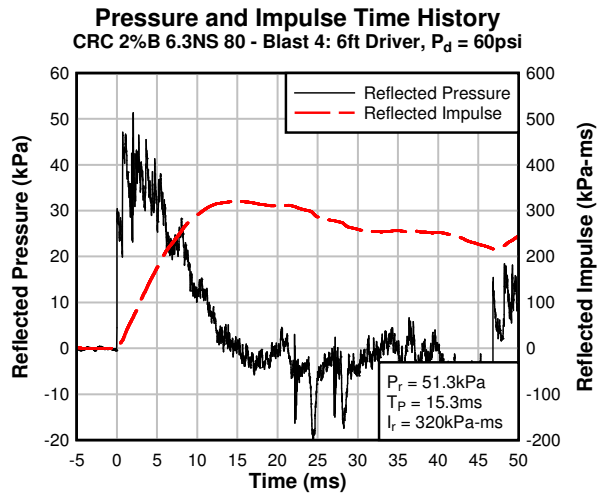


Figure 4-24 CRC-2%B-6.3NS-80, recorded reflected pressure, impulse and displacement for Blast 1-2

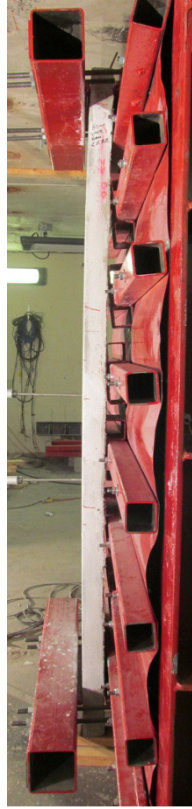


a) Blast 3: Reflected pressure, impulse, and displacement time histories.

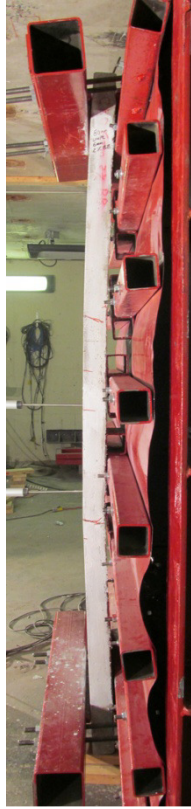


b) Blast 4: Reflected pressure, impulse, and displacement time histories.

Figure 4-25 CRC-2%B-6.3NS-80, recorded reflected pressure, impulse and displacement for Blast 3-4



a) Blast 1



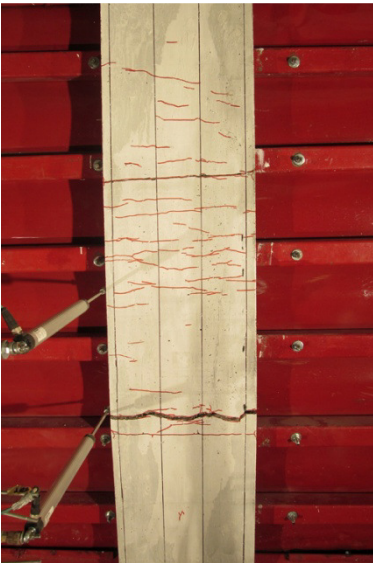
b) Blast 2



c) Blast 3



d) Blast 4



e) Mid-span cracking at Blast 3



f) Mid-span cracking at Blast 4



g) Tensile crack at Blast 4

Figure 4-26 CRC-2%B-6.3NS-80, photographs at the end of Blast 1-4

4.6.10. CRC - 2%B - #3NS - 80

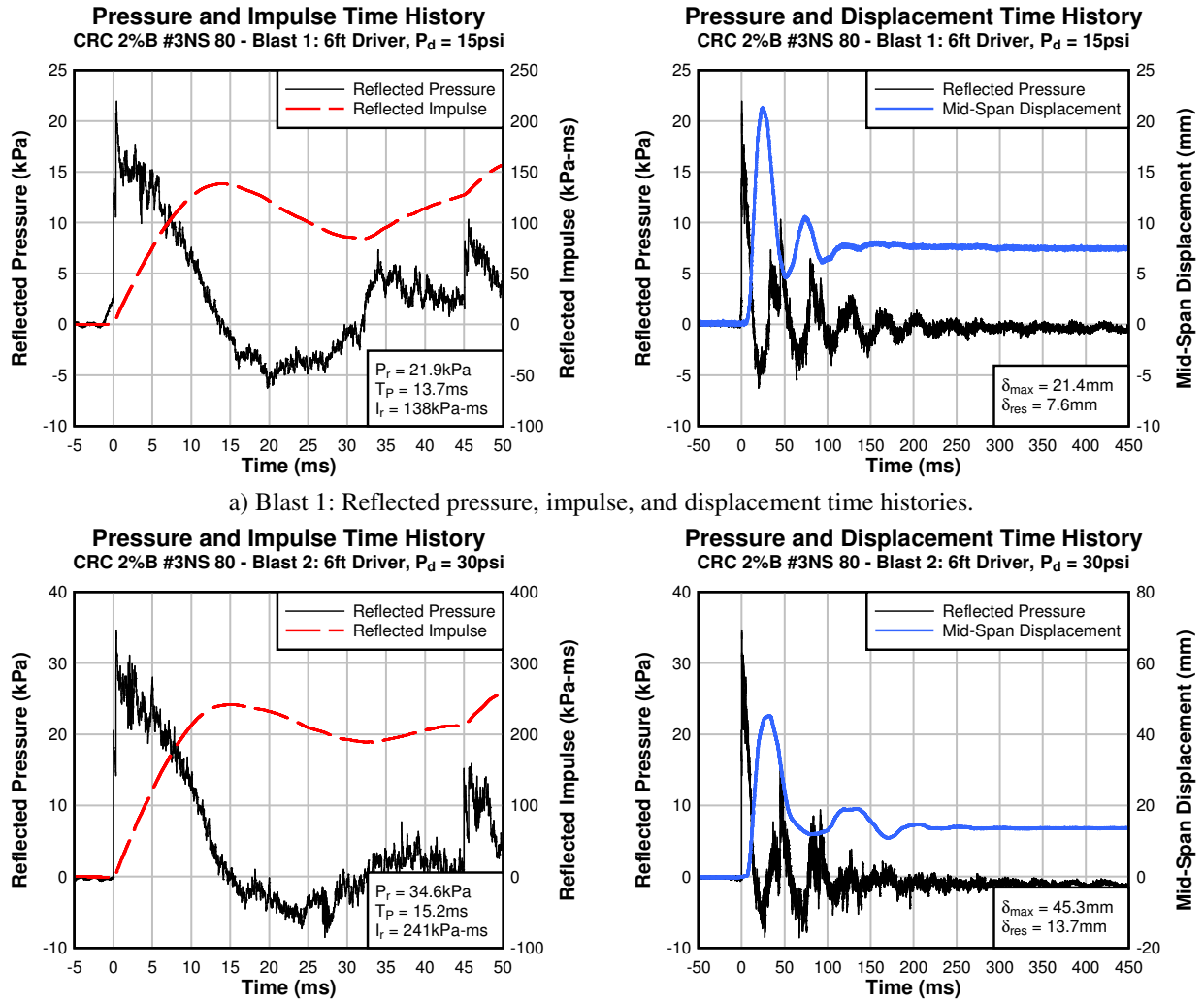
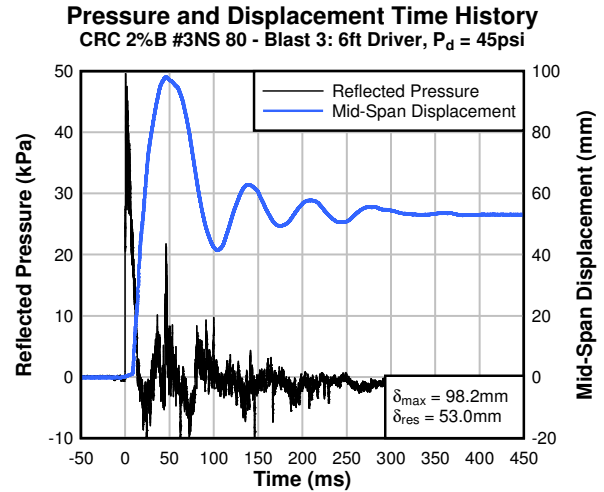
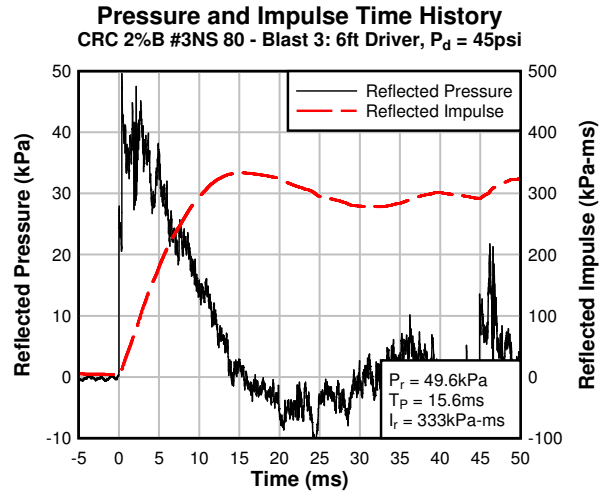
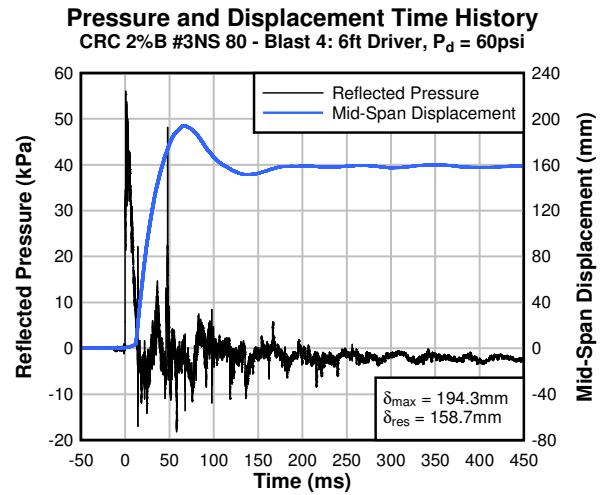
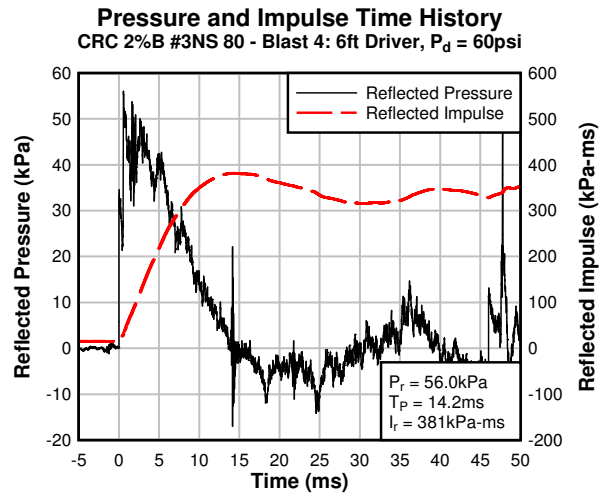


Figure 4-27 CRC-2%B-#3NS-80, recorded reflected pressure, impulse and displacement for Blast 1-2



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



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

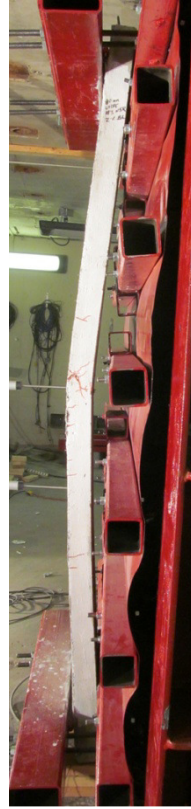
Figure 4-28 CRC-2%B-#3NS-80, recorded reflected pressure, impulse and displacement for Blast 3-4



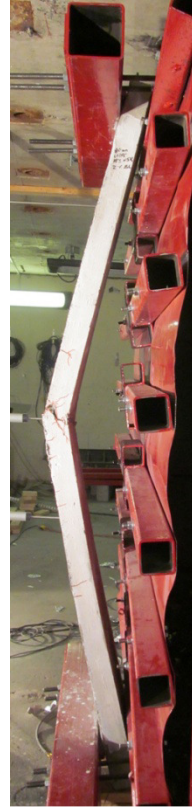
a) Blast 1



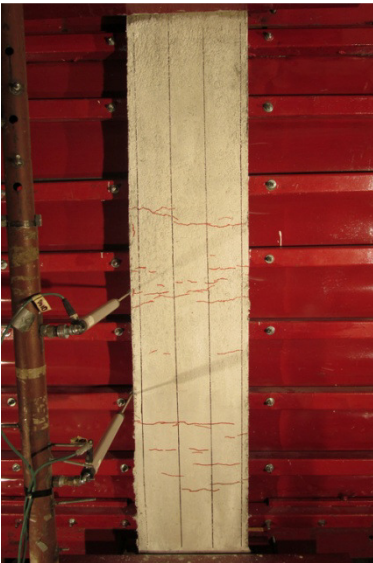
b) Blast 2



c) Blast 3



d) Blast 4



e) Hairline cracking at Blast 2



f) Mid-span cracking at Blast 3



g) Mid-span cracking at Blast 4

Figure 4-29 CRC-2%B-#3NS-80, photographs at the end of Blast 1-4

4.6.11. CRC - 2%B - #3NS - 100

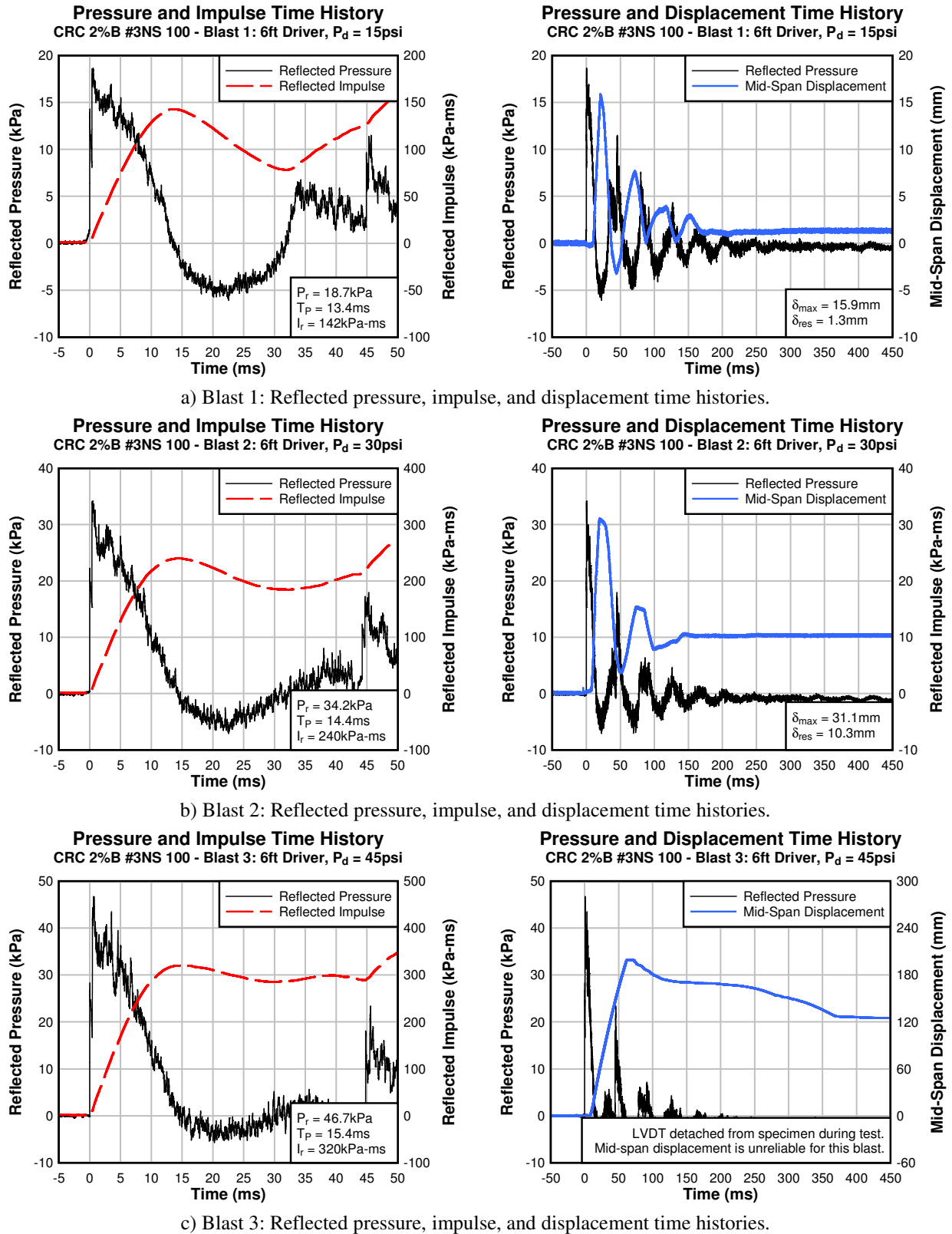
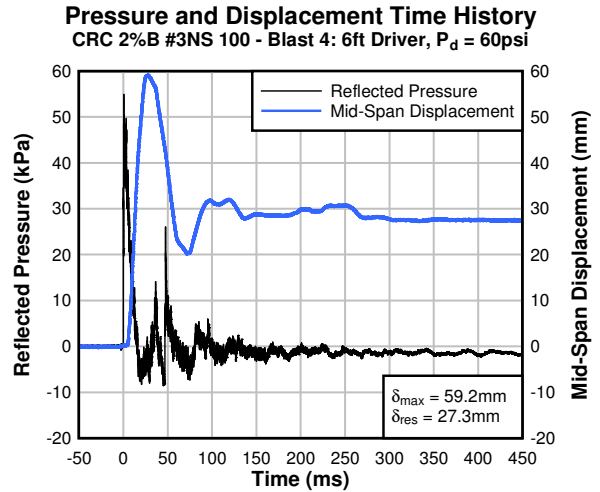
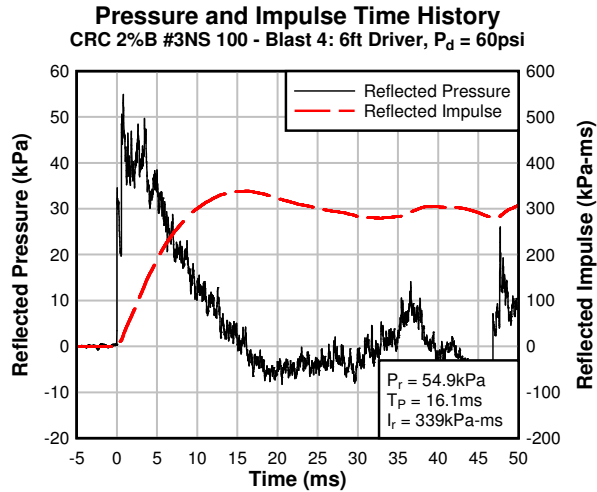
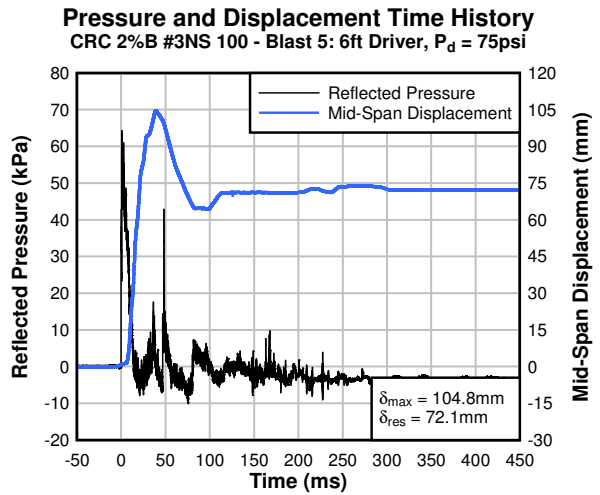
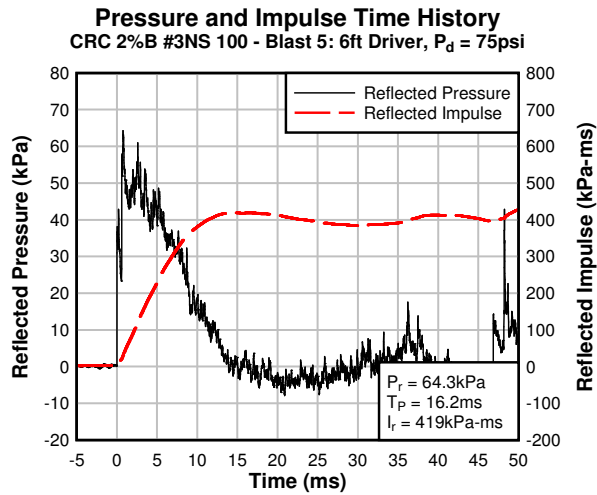


Figure 4-30 CRC-2%B-#3NS-100, recorded reflected pressure, impulse and displacement for Blast 1-3



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

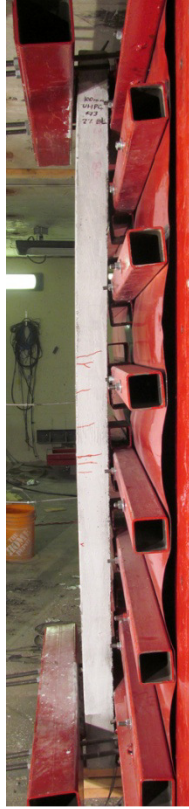


b) Blast 5: Reflected pressure, impulse, and displacement time histories.

Figure 4-31 CRC-2%B-#3NS-100, recorded reflected pressure, impulse and displacement for Blast 4-5



a) Blast 1



b) Blast 2



c) Blast 3



d) Blast 4



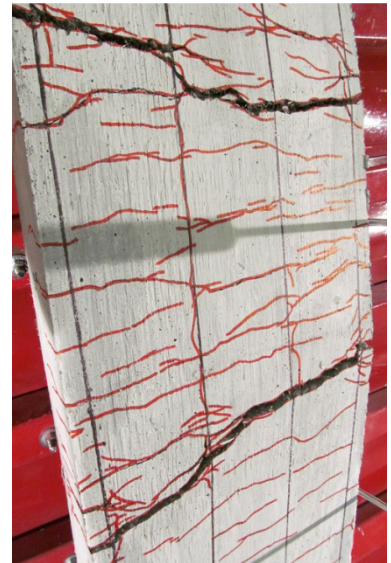
e) Blast 5



f) Mid-span cracking at Blast 3



g) Mid-span cracking at Blast 4



h) Mid-span cracking at Blast 5

Figure 4-32 CRC-2%B-#3NS-100, photographs at the end of Blast 1-5

4.6.12. CRC - 2%A - #3NS - 100

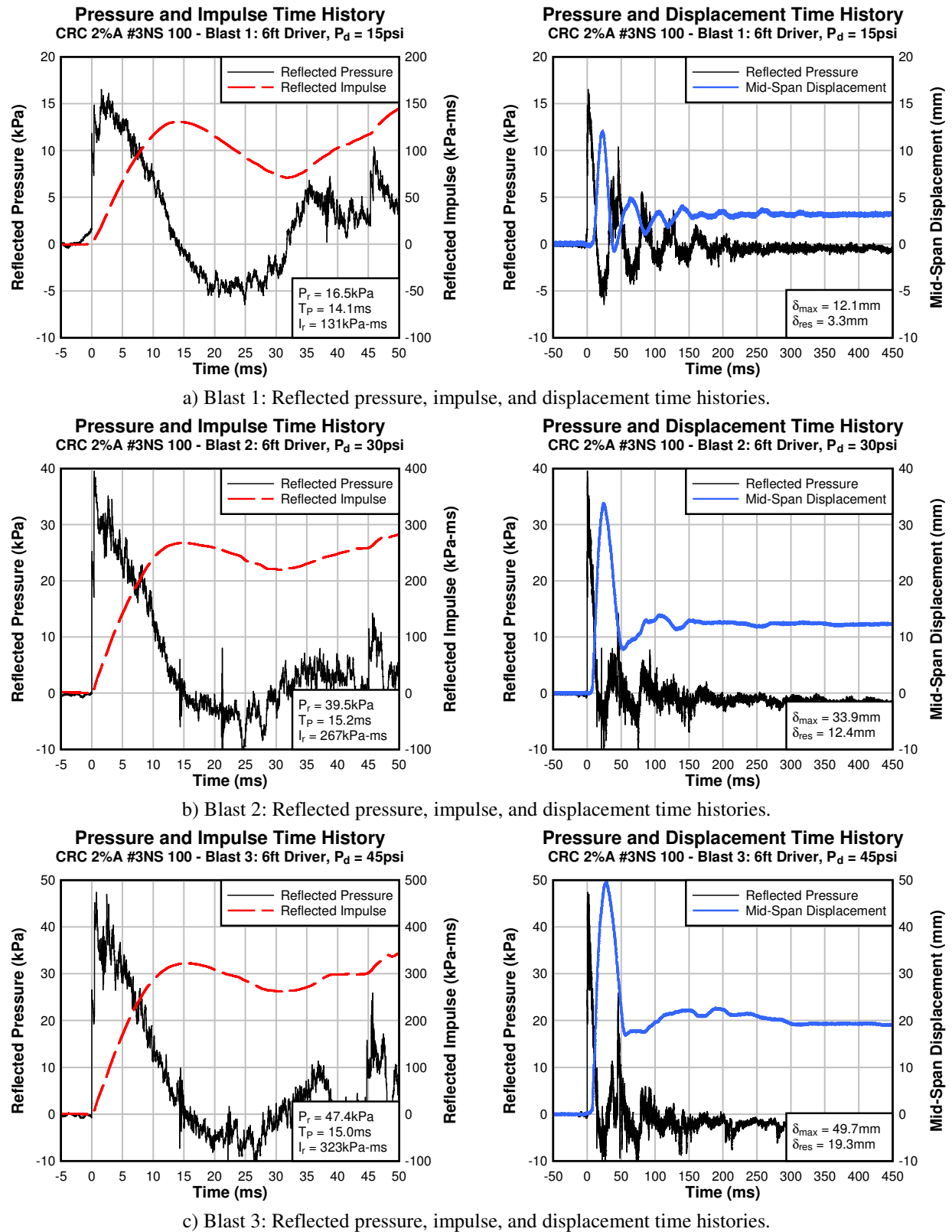
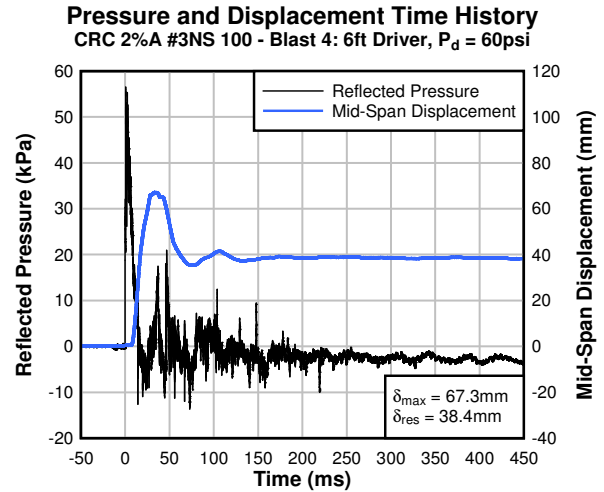
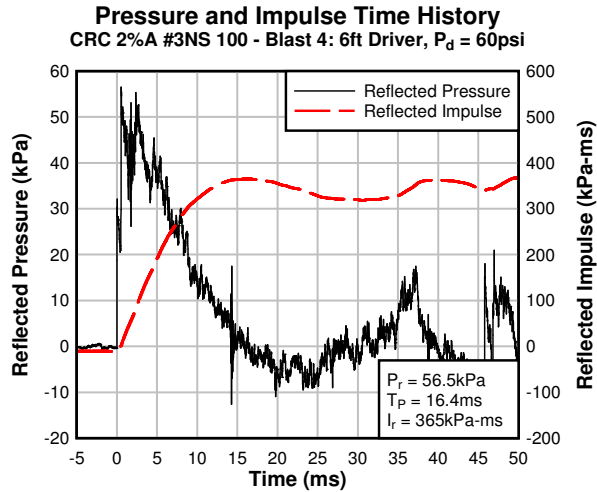
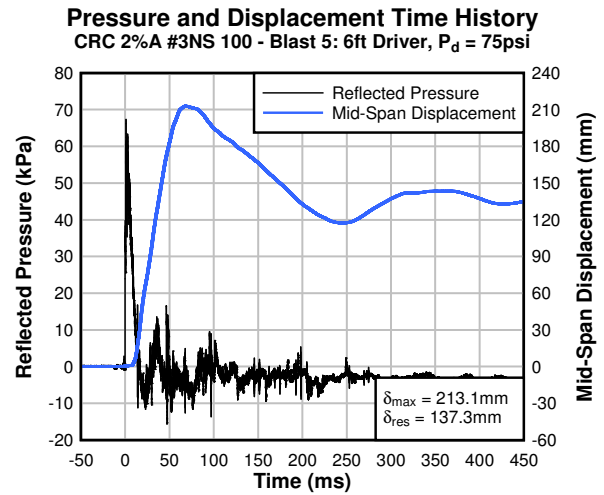
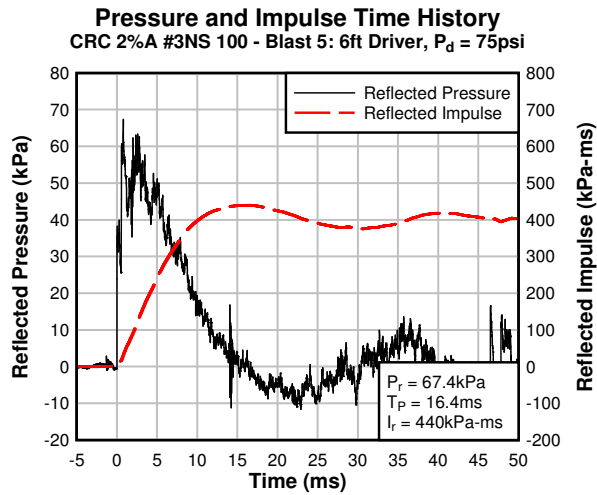


Figure 4-33 CRC-2%A-#3NS-100, recorded reflected pressure, impulse and displacement for Blast 1-3

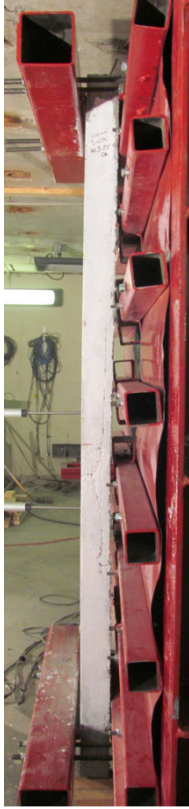


a) Blast 4: Reflected pressure, impulse, and displacement time histories.



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

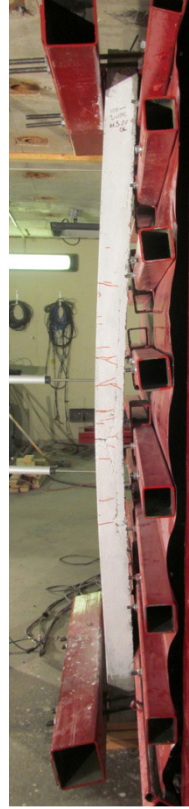
Figure 4-34 CRC-2%A-#3NS-100, recorded reflected pressure, impulse and displacement for Blast 4-5



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 cracking at Blast 5



h) Bar rupture at Blast 5

Figure 4-35 CRC-2%A-#3NS-100, photographs at the end of Blast 1-5

4.6.13. CRC - 2%A - #3HS - 100

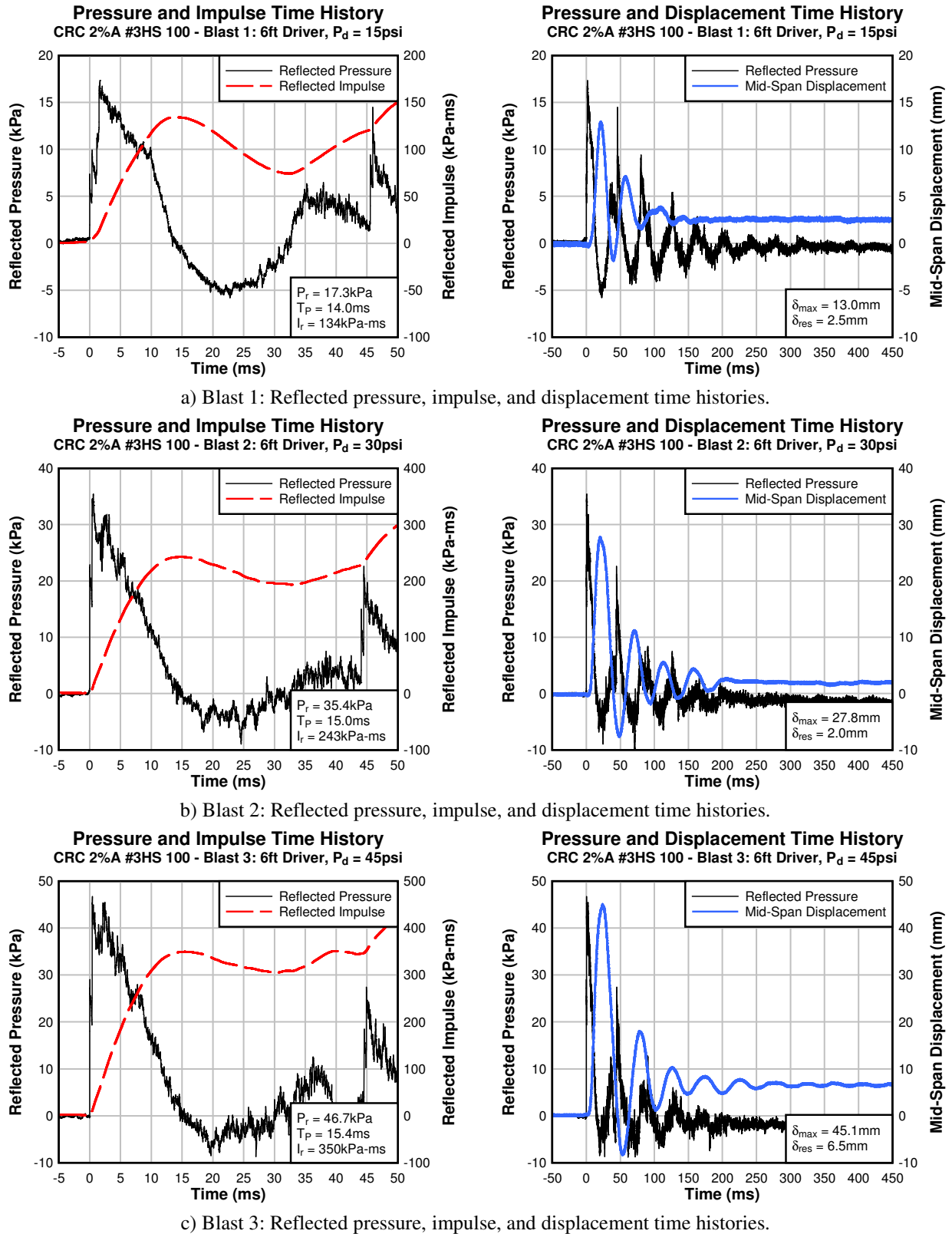
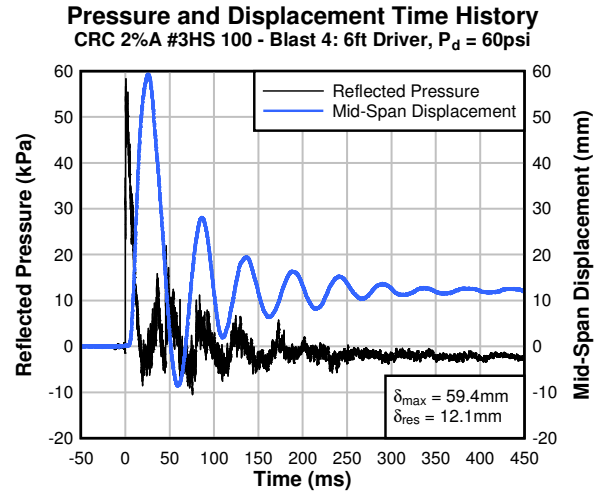
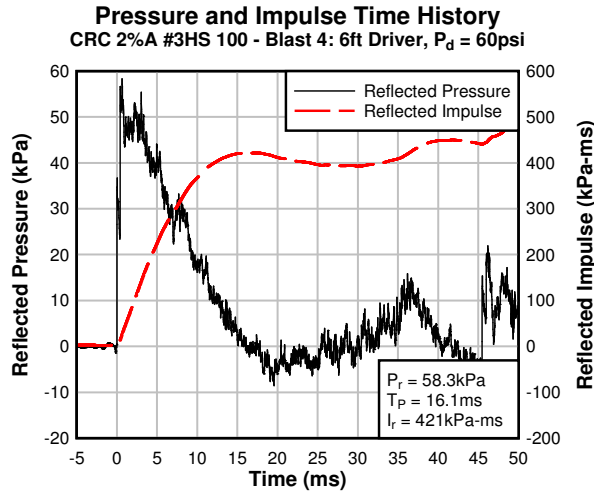
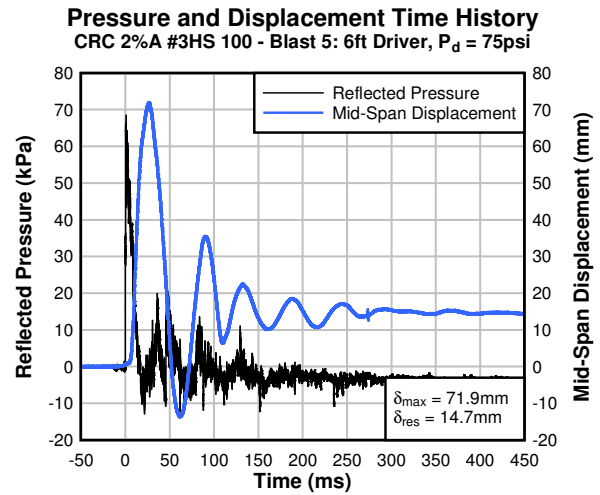
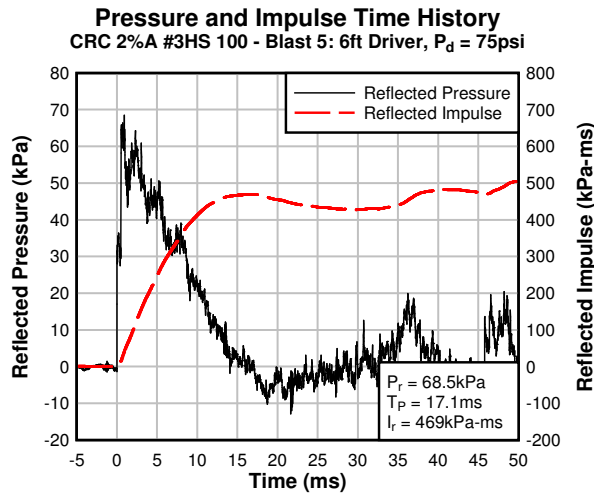


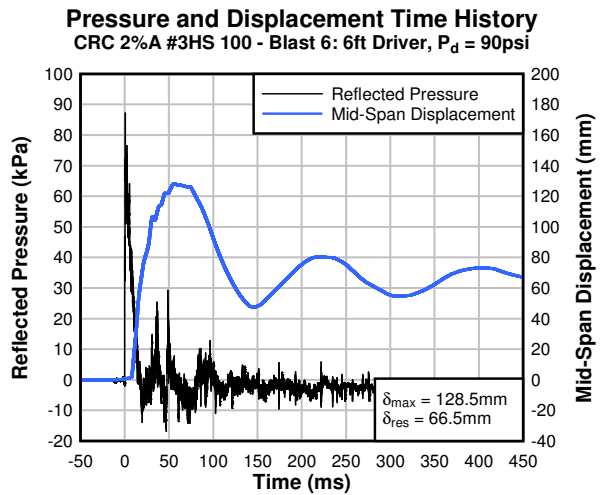
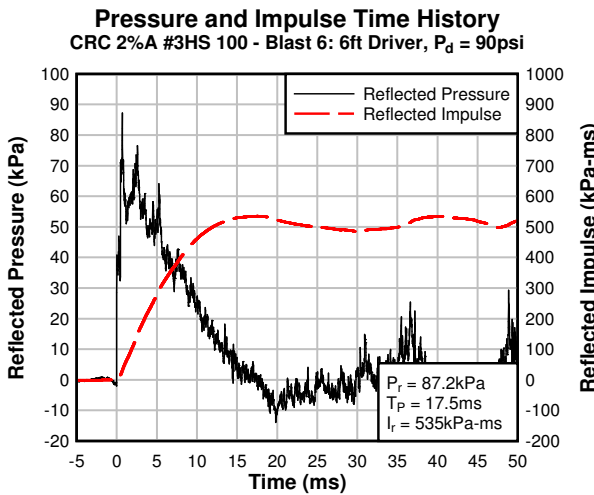
Figure 4-36 CRC-2%A-#3HS-100, recorded reflected pressure, impulse and displacement for Blast 1-3



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

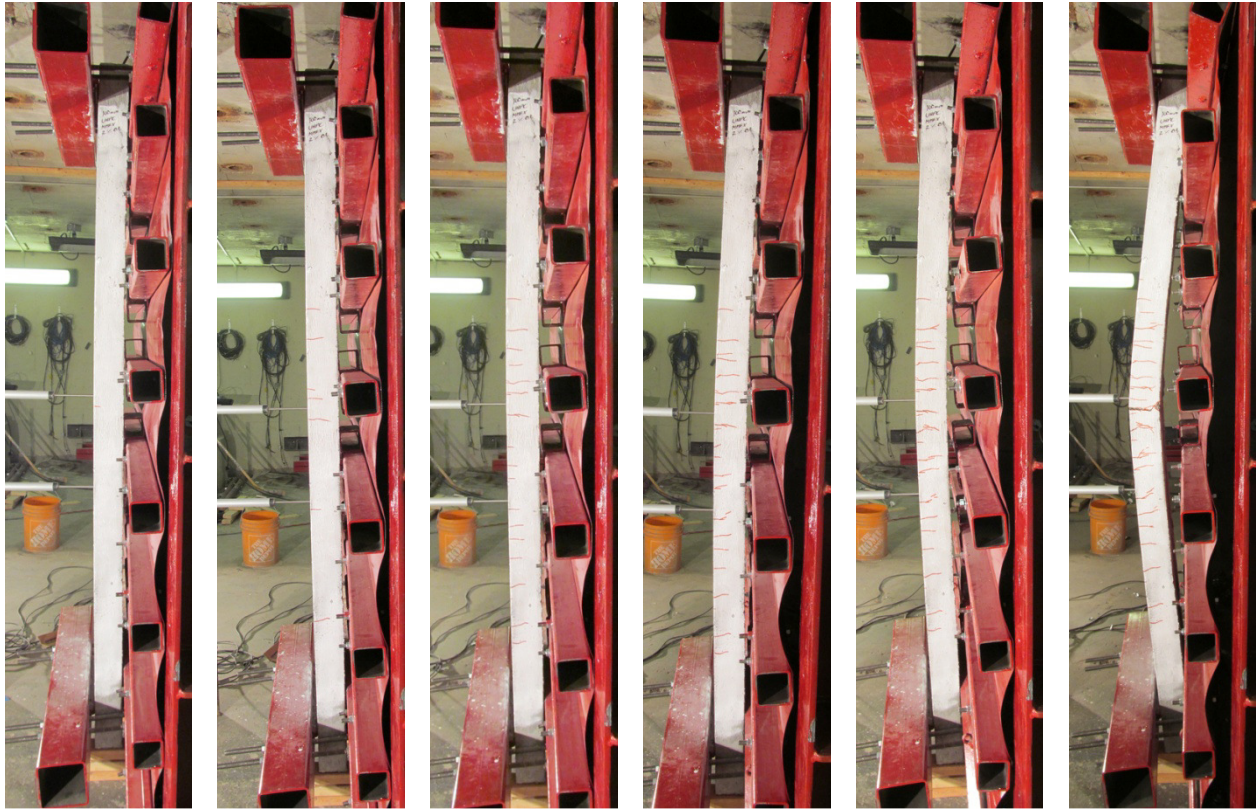


b) Blast 5: Reflected pressure, impulse, and displacement time histories.



c) Blast 6: Reflected pressure, impulse, and displacement time histories.

Figure 4-37 CRC-2%A #3HS-100, recorded reflected pressure, impulse and displacement for Blast 4-6



a) Blast 1

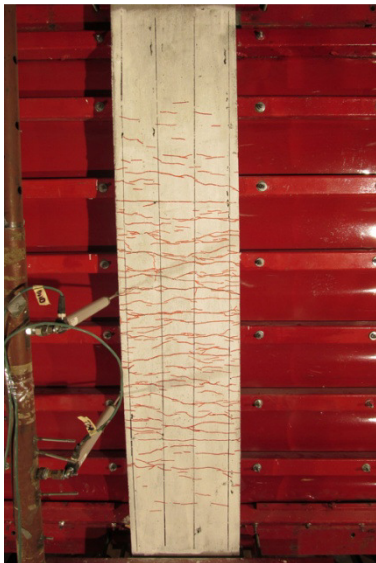
b) Blast 2

c) Blast 3

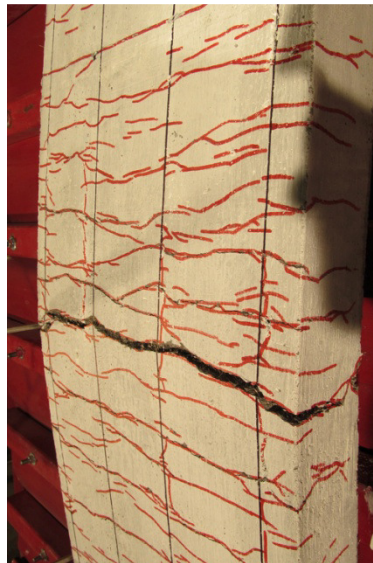
d) Blast 4

e) Blast 5

f) Blast 6



g) Specimen cracking at Blast 5



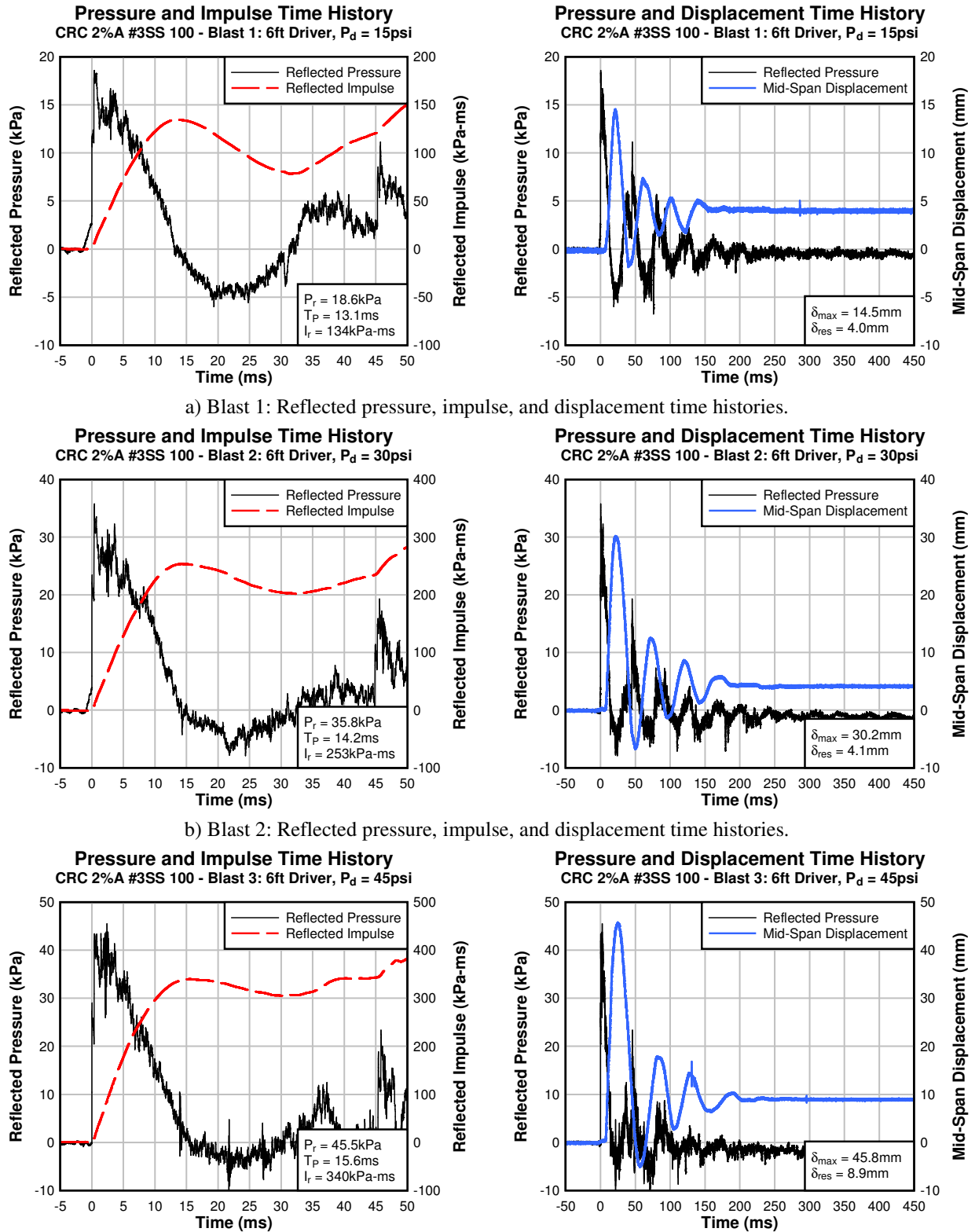
h) Mid-span cracking at Blast 6



i) Bar rupture at Blast 6

Figure 4-38 CRC-2%A-#3HS-100, photographs at the end of Blast 1-6

4.6.14. CRC - 2%A - #3SS - 100

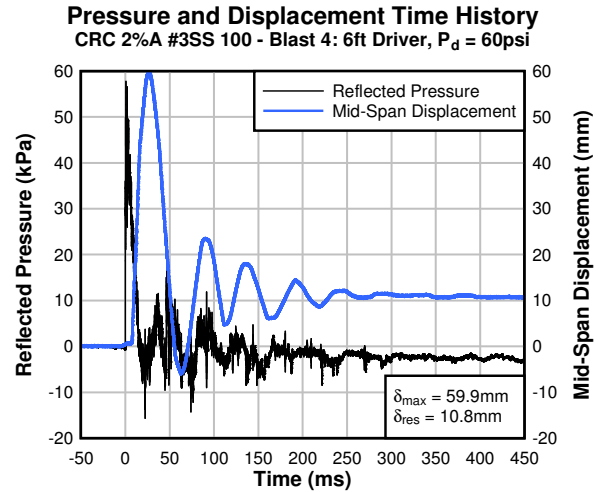
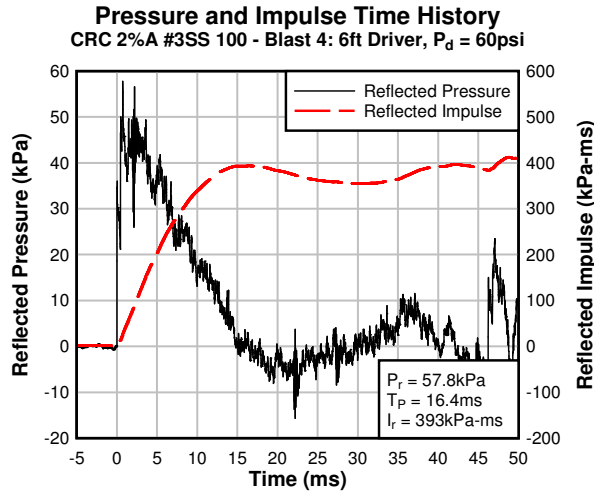


a) Blast 1: Reflected pressure, impulse, and displacement time histories.

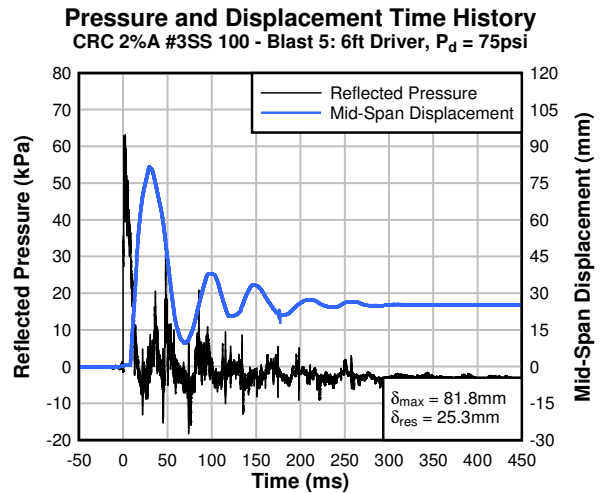
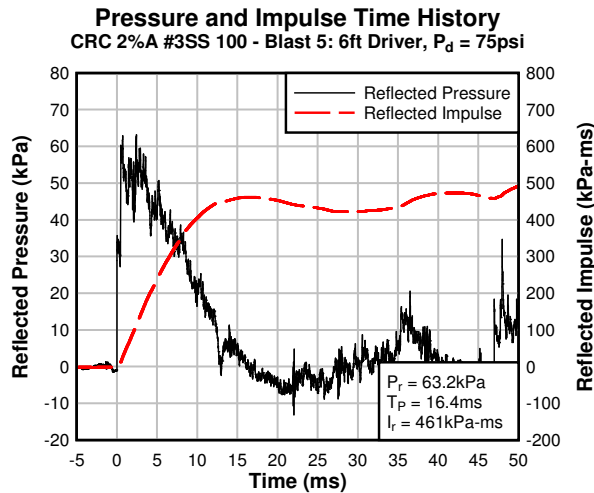
b) Blast 2: Reflected pressure, impulse, and displacement time histories.

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

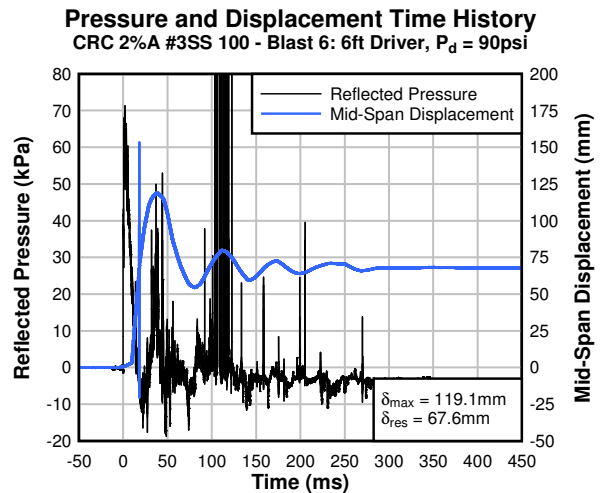
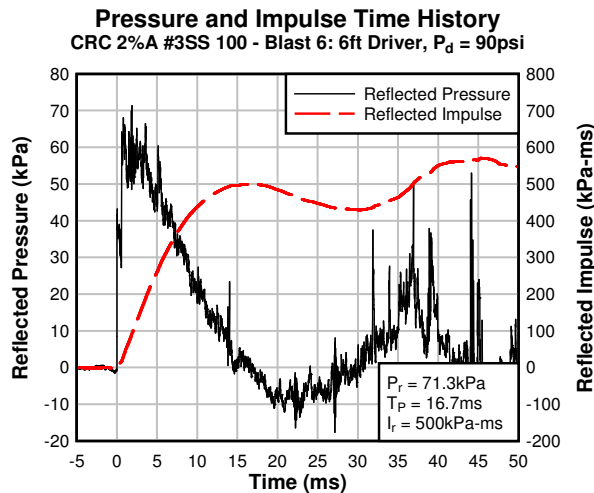
Figure 4-39 CRC-2%A-#3SS-100, recorded reflected pressure, impulse and displacement for Blast 1-3



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

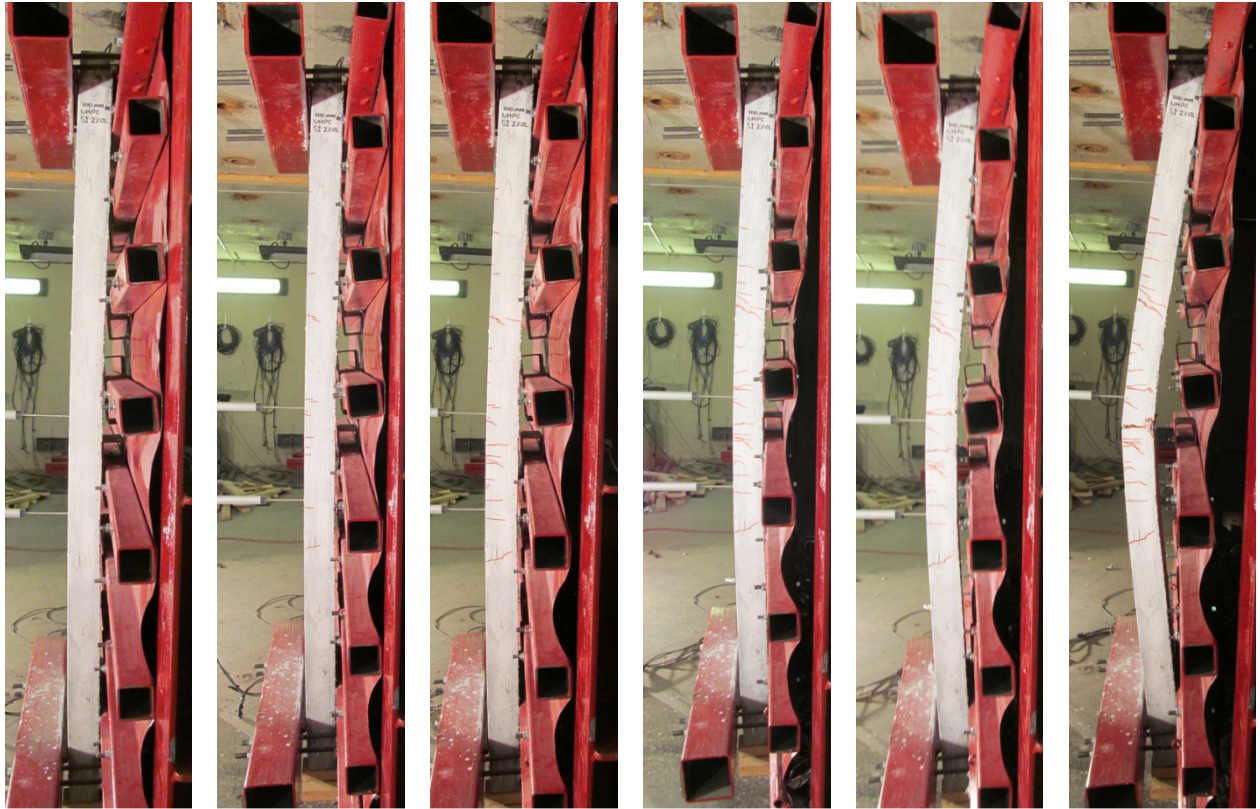


b) Blast 5: Reflected pressure, impulse, and displacement time histories.



c) Blast 6: Reflected pressure, impulse, and displacement time histories.

Figure 4-40 CRC-2%A-#3SS-100, recorded reflected pressure, impulse and displacement for Blast 4-6



a) Blast 1

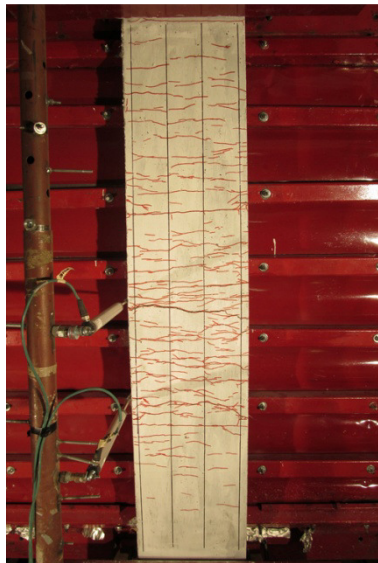
b) Blast 2

c) Blast 3

d) Blast 4

e) Blast 5

f) Blast 6



g) Specimen cracking at Blast 5



h) Mid-span cracking at Blast 6



i) Fibres visible at Blast 6

Figure 4-41 CRC-2%A-#3SS-100, photographs at the end of Blast 1-6

CHAPTER 5. DISCUSSION

5.1. CHAPTER OVERVIEW

The purpose of this chapter is to provide observations in regards to comparisons between the responses of all fourteen reinforced concrete slabs tested for the experimental program of this research. The performance criteria for these comparisons, as previously mentioned in the scope of work, relate to maximum and residual displacements, failure modes, failure blast load magnitudes, crack control and secondary fragmentation restrictions.

This chapter is divided in several sections, each presenting a different aspect as listed below:

The effects of the concrete type:

- in 80 mm slab specimens with non-deformed steel bars
 - SCC-0%-6.3NS-80 vs. SCC-1.2%ZP-6.3NS-80 vs. CRC-2%B-6.3NS-80
- in 100 mm slab specimens with deformed traditional steel bars
 - SCC-0%-#3NS-100 vs. SCC-1.2%ZP-#3NS-100 vs. CRC-2%A-#3NS-100
- in 100 mm slab specimens with deformed high strength steel bars
 - SCC-0%-#3HS-100 vs. SCC-1.2%ZP-#3HS-100 vs. CRC-2%A-#3HS-100

The effects of the steel reinforcement type:

- with high strength steel in regular concrete
 - SCC-0%-#3NS-100 vs. SCC-0%-#3HS-100
- with high strength steel in SFRC
 - SCC-1.2%ZP-#3NS-100 vs. SCC-1.2%ZP-#3HS-100
- with high strength steel in UHPFRC
 - CRC-2%B-#3NS-100 vs. CRC-2%B-#3HS-100
- with high ductility stainless steel in UHPFRC
 - CRC-2%B-#3NS-100 vs. CRC-2%B-#3SS-100
- comparing high strength steel and stainless steel specimens in UHPFRC
 - CRC-2%B-#3HS-100 vs. CRC-2%B-#3SS-100

The effects of the steel fibre volume ratio:

- in SFRC
 - SCC-0%-#3NS-100 vs. SCC-0.6%ZP-#3NS-100 vs. SCC-1.2%ZP-#3NS-100

The effects of the steel fibre type:

- in SFRC
 - SCC-0.6%ZP-#3NS-100 vs. SCC-0.6%BP-#3NS-100
- in UHPFRC

- CRC-2%A-#3NS-100 vs. CRC-2%B-#3NS-100

The effects of the concrete type on the slab depth:

- comparing 80 mm deep UHPFRC slabs against thicker 100 mm deep SCC slabs (CRC-2%B-#3NS-80 vs. SCC-0%-#3NS-100)

Damage tolerance and secondary fragmentation:

- comparing the general behaviour of SFRC and UHPFRC against plain SCC

5.2. GENERAL OBSERVATIONS

Based on data previously compiled and described through Chapter 4, the maximum and residual mid-span displacements for each blast tests are summarized in Figure 5-1 through Figure 5-6 along with notes on particular failure modes. In addition, Table 5-1 was included as a summary of the maximum crack width recorded for each test; note that the acronym HL stands for hairline cracks. A brief examination of these tables and charts indicated improvements in the behaviour of one-way slabs when using innovative materials over traditional materials. In most cases, these improvements were more perceptible during later blasts as the first blast tests were meant to be within the elastic range or yielding point of the specimens. Details on the effects of each material parameter were included in the subsequent sections.

Table 5-1 Summary of principal crack widths

Specimen	Principle Crack Width (mm)					
	Blast 1	Blast 2	Blast 3	Blast 4	Blast 5	Blast 6
SCC-0%-6.3NS-80	1.75	5.5	15-40	-	-	-
SCC-0%-#3NS-100	0.40	1.25	2.00	4-20	15-30	-
SCC-0%-#3HS-100	HL	HL	0.30	1.00	4	N/A
SCC-1.2%ZP-6.3NS-80	0.40	6	13-25	25-33	-	-
SCC-0.6%ZP-#3NS-100	HL	1.25	2.00	9-13	25-35	-
SCC-1.2%ZP-#3NS-100	HL	1.00	2.5	12.5	25-45	-
SCC-0.6%BP-#3NS-100	HL	0.50	1.75	12	25	-
SCC-1.2%ZP-#3HS-100	HL	HL	0.30	1.50	4-10	-
CRC-2%B-6.3NS-80	HL	2.5	11	18	-	-
CRC-2%B-#3NS-80	HL	0.40	9	22-28	-	-
CRC-2%B-#3NS-100	HL	1.00	3-7	5-9	12-15	-
CRC-2%A-#3NS-100	HL	0.75	2.5	6-10	33	-
CRC-2%A-#3HS-100	HL	HL	0.40	0.75	2.00	7-10
CRC-2%A-#3SS-100	HL	HL	0.40	1.00	4	13

Blast 1: Maximum and Residual Displacements

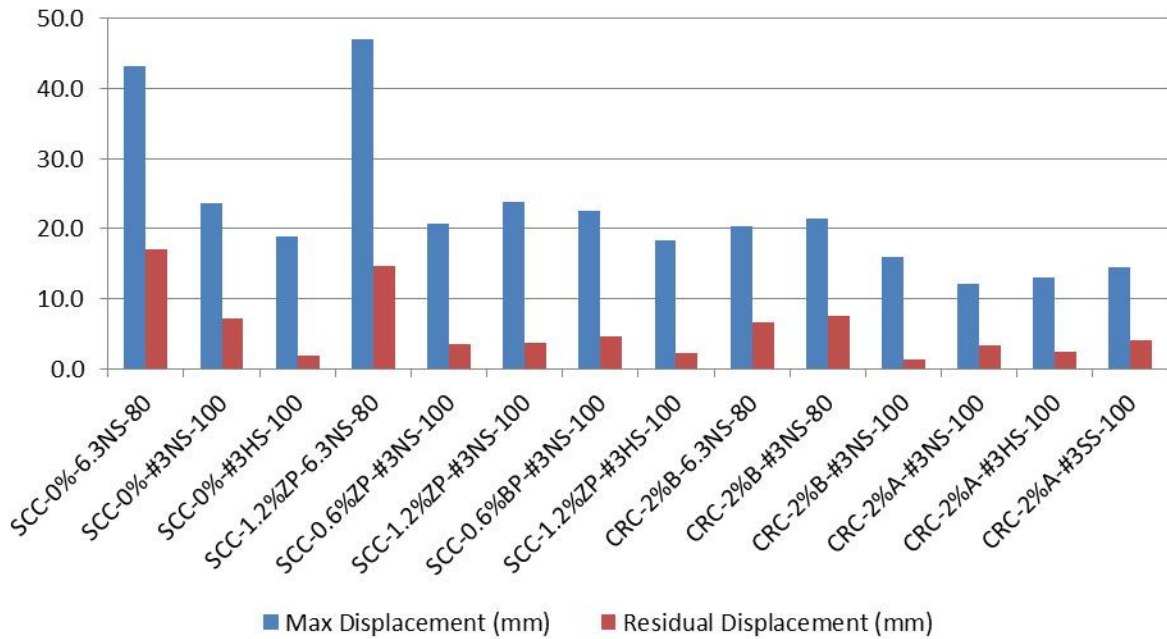


Figure 5-1 Maximum and residual displacements for blast 1

Blast 2: Maximum and Residual Displacements

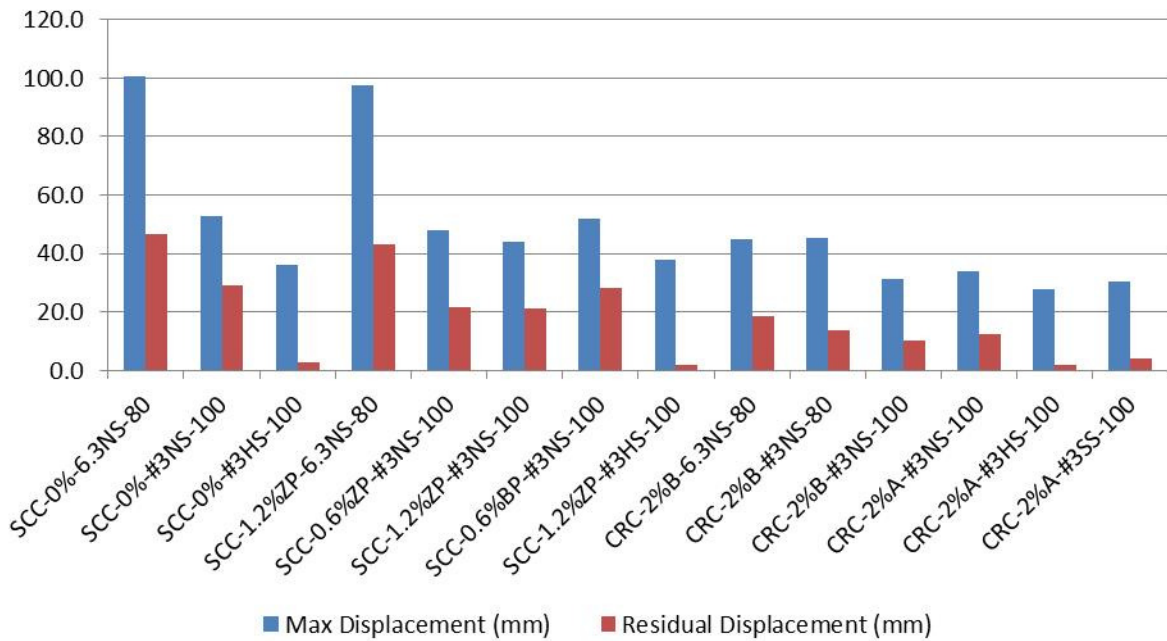


Figure 5-2 Maximum and residual displacements for blast 2

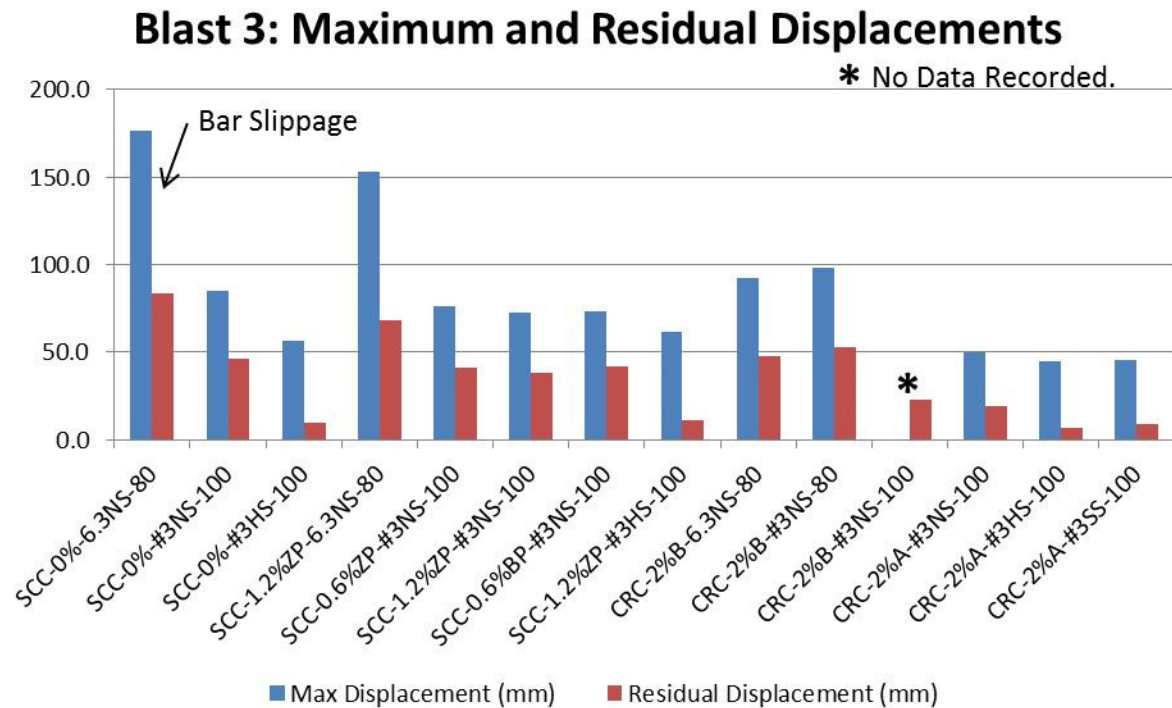


Figure 5-3 Maximum and residual displacements for blast 3

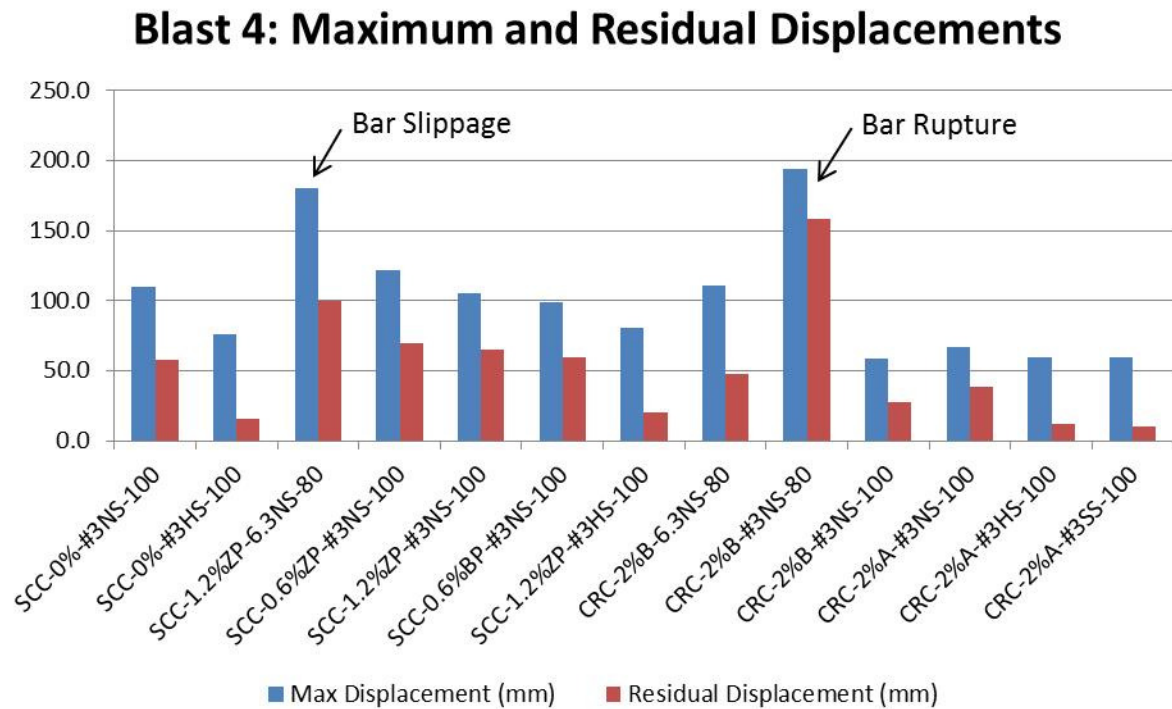


Figure 5-4 Maximum and residual displacements for blast 4

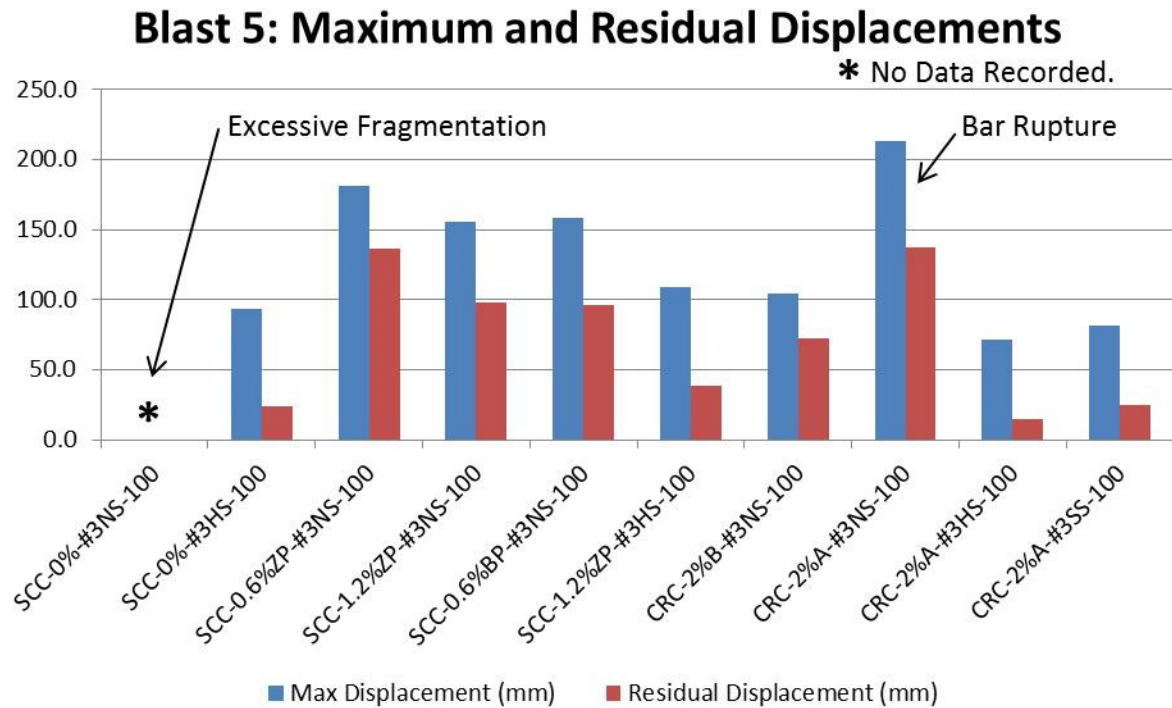


Figure 5-5 Maximum and residual displacements for blast 5

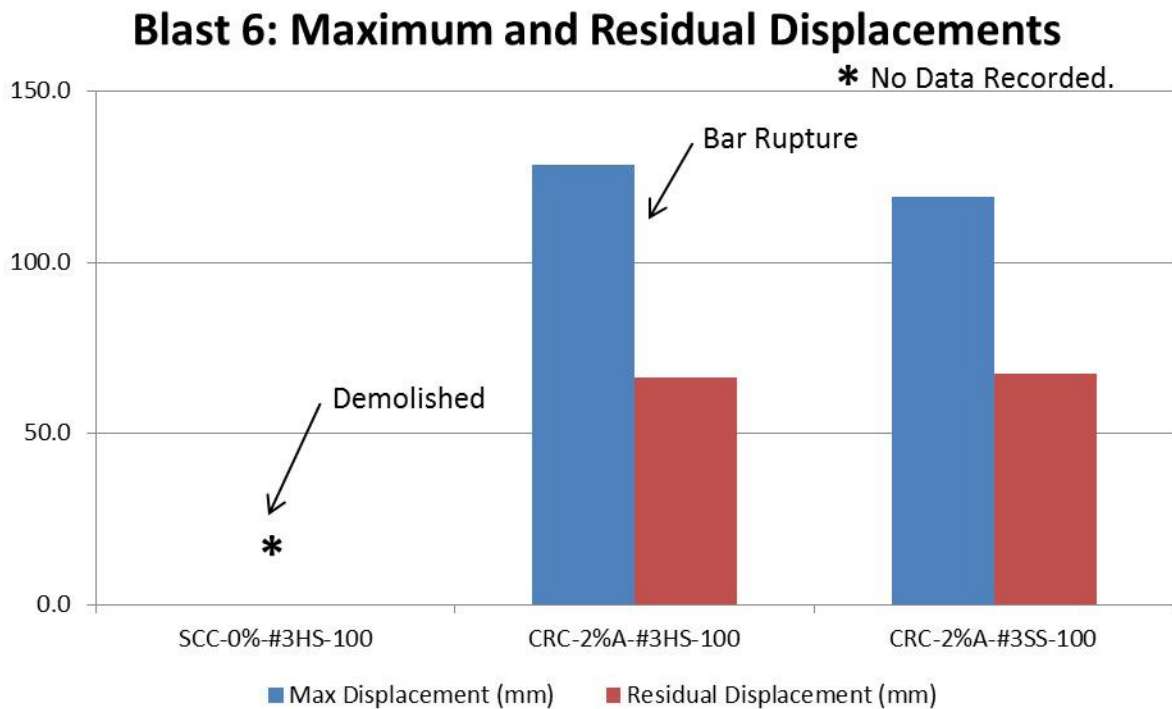


Figure 5-6 Maximum and residual displacements for blast 6

5.3. EFFECTS OF CONCRETE TYPE

The reinforced concrete slabs that were tested in this research showed significant improvements from the use of SFRC and UHPFRC against the plain self-consolidated concrete mix. The following comparisons provide details on the benefits that were observed during the experimental program.

5.3.1. EFFECTS OF CONCRETE TYPE: SCC vs. SFRC vs. UHPFRC – 80MM SPECIMENS

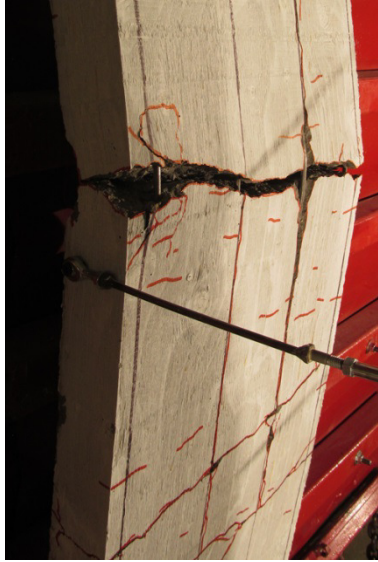
The control specimen SCC-0%-6.3NS-80, the SFRC specimen SCC-1.2%ZP-6.3NS-80 and the UHPFRC specimen CRC-2%B-6.3NS-80 were all designed with a depth of 80 mm and non-deformed steel bars with a diameter of 6.3 mm. Relevant photographs are included in Figure 5-7. Comparative displacement bar charts and time histories for blasts 2 and 3 are provided in Figure 5-8 and Figure 5-9.

The UHPFRC mix provided in specimen CRC-2%B-6.3NS-80 greatly reduced the magnitude of the mid-span displacements when compared to the two other aforementioned slabs. When compared with the control specimen at blast 2, it showed a reduction of 55.5% of the maximum displacement and 60.5% of the residual displacement. At blast 3, the maximum and residual mid-span displacements decreased by 47.7% and 42.7% respectively.

A similar trend was observed between the UHPFRC specimen and the SFRC specimen. At blast 2, the displacements were reduced by 54.1% for the maximum and 57.4% for the residual. Finally, at blast 3, the maximum and residual mid-span displacements diminished by 39.6% and 29.6%.

As for the SFRC specimen, when compared with the 80 mm control specimen, it showed a slight improvement in terms of displacements. At blast 2, the maximum and residual mid-span displacements were reduced by 3.0% and 7.3% respectively. At blast 3, the reduction reached 13.4% for the maximum displacement and 18.6% for the residual displacement.

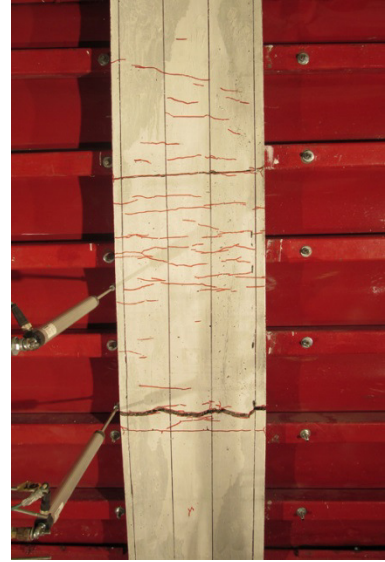
In addition to an improved response of the system, it should be noted that both the UHPFRC and SFRC specimens were capable of resisting a greater blast load prior to failure than the control; while failure occurred during blast 3 for the control specimen, both fibre reinforced concrete slabs failed during blast 4. Damage was also less prominent for the UHPFRC specimen as seen in Figure 5-7(c). However, normal strength reinforcement was prone to rupture in UHPFRC due to the strength of the concrete mix as portrayed in Figure 5-7(e).



a) SCC-0%-6.3NS-80
Bar slippage at Blast 3



b) SCC-1.2%ZP-6.3NS-80
Damage at Blast 3



c) CRC-2%B-6.3NS-80
Damage at Blast 3

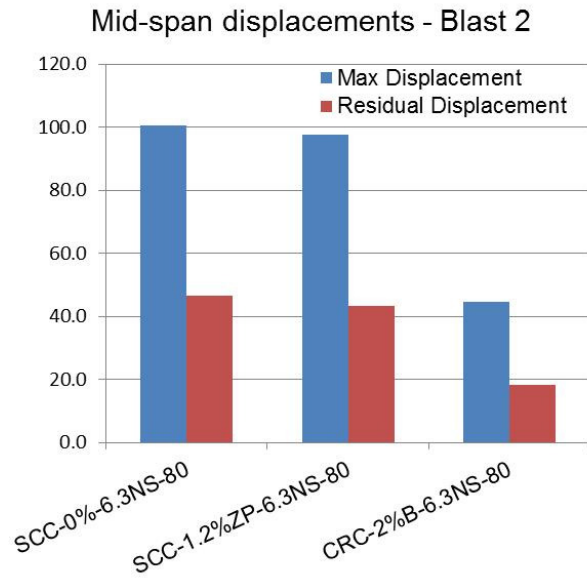


d) SCC-1.2%ZP-6.3NS-80
Bar slippage at Blast 4

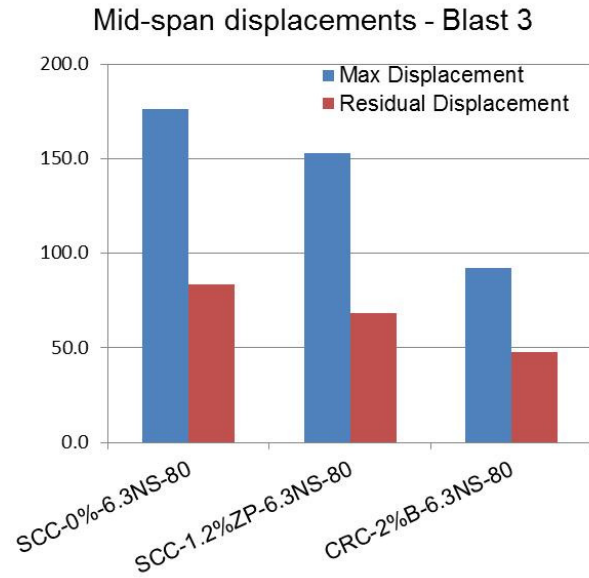


e) CRC-2%B-6.3NS-80
Bar rupture at Blast 4

Figure 5-7 Photographs; effects of concrete type in 80 mm slabs

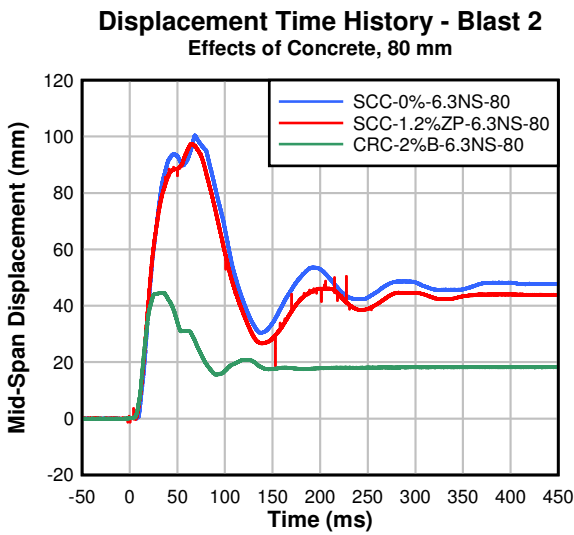


a) Blast 2

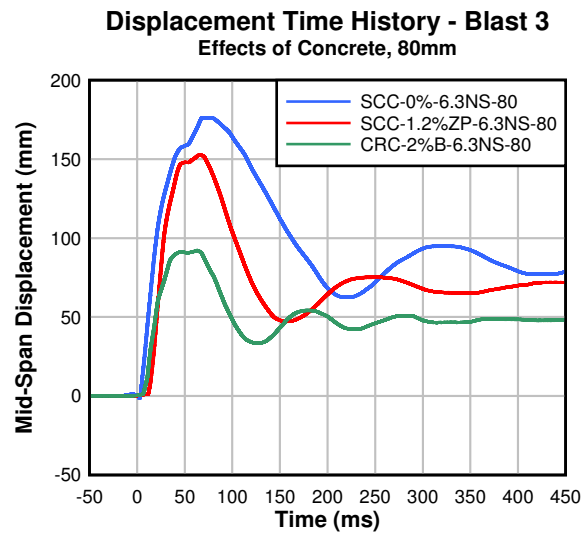


b) Blast 3

Figure 5-8 Maximum and residual displacements; effects of concrete type in 80 mm slabs



a) Blast 2



b) Blast 3

Figure 5-9 Displacement time histories; effects of concrete type in 80 mm slabs

5.3.2. EFFECTS OF CONCRETE TYPE: SCC vs. SFRC vs. UHPFRC – 100MM SPECIMENS

The control specimen SCC-0%-#3NS-100, the SFRC specimen SCC-1.2%ZP-#3NS-100 and the UHPFRC specimen CRC-2%A-#3NS-100 were all designed with a depth of 100 mm and US size #3 deformed steel bars. Relevant photographs are included in Figure 5-10. Comparative displacement bar charts and time histories for blasts 3 and 4 are provided in Figure 5-11 and Figure 5-12.

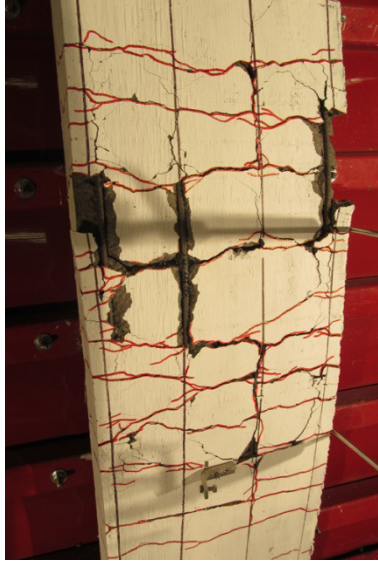
Similarly to the previous comparison with 80 mm reinforced concrete slabs, the UHPFRC mix used for specimen CRC-2%A-#3NS-100 significantly reduced the magnitude of the mid-span displacements. When compared with the control specimen at blast 3, the maximum and residual displacements decreased by 41.5% and 58.3% respectively. At blast 4, the displacements were reduced by 38.9% for the maximum and 33.9% for the residual.

When compared with the SFRC specimen, the UHPFRC specimen shows a similar trend. At blast 3, the maximum and residual mid-span displacements were diminished by 31.8% and 49.3% respectively. At blast 4, the reduction reaches 36.0% for the maximum displacement and 41.4% for the residual displacement.

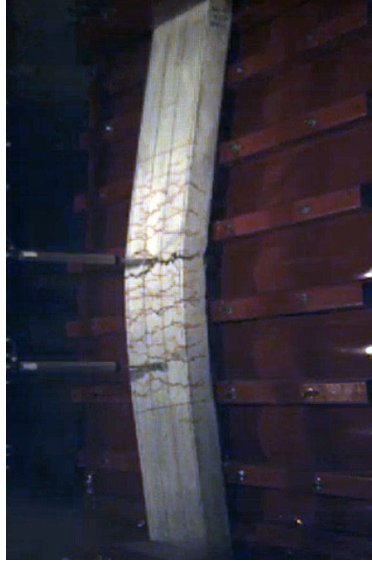
Finally, when compared to the control, the SFRC specimen showed a slight improvement for displacements. At blast 3, the maximum and residual mid-span displacements were diminished by 14.1% and 17.7% respectively. For blast 4, the maximum displacement was reduced by 4.6% however the residual displacement was greater than the control's residual by 12.7%.

The minimal improvements to the mid-span displacements with the SFRC specimen show that the addition of steel fibres does not necessarily decrease the displacement response of the simply supported slab system. However, when observing cracking and fragmentation on Figure 5-10, photographs B and E display an improvement in damage control when compared with the control specimen's photographs A and D.

The use of UHPFRC further improves crack control as shown on Figure 5-10(f) and Figure 5-10(f). Yet, the specimen still suffers from the brittle bar rupture failure that was mentioned in the previous sub-section with the set of 80 mm specimens.



a) SCC-0%-#3NS-100
Exposed rebar at Blast 4



b) SCC-1.2%ZP-#3NS-100
Tensile cracking at Blast 4



c) CRC-2%A-#3NS-100
Tensile cracking at Blast 4



d) SCC-0%-#3NS-100
Excessive fragmentation at Blast 5

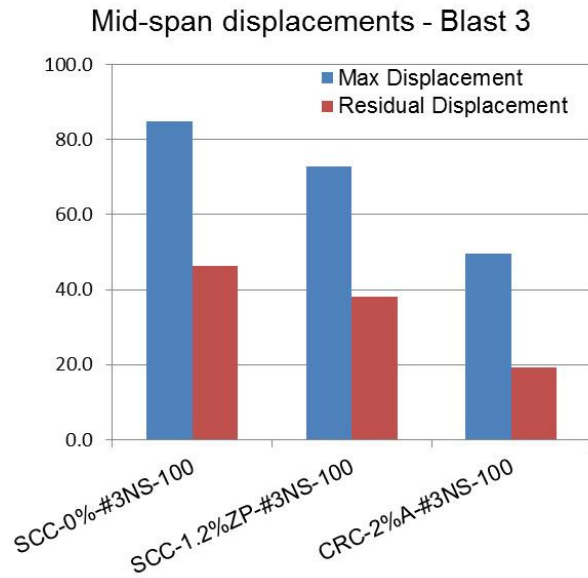


e) SCC-1.2%ZP-#3NS-100
Improved crack control at Blast 5

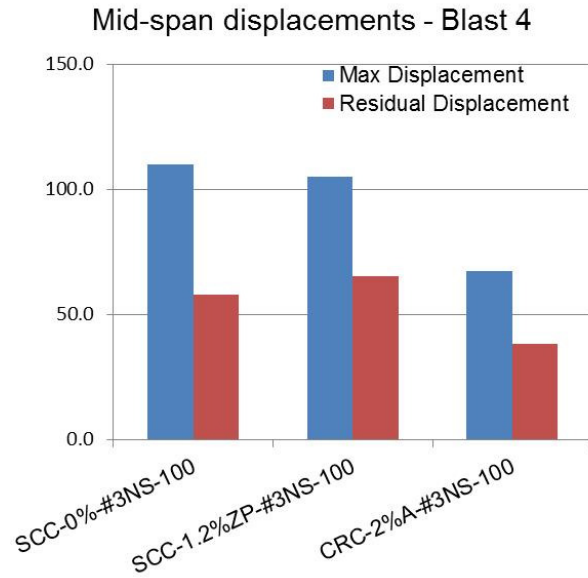


f) CRC-2%A-#3NS-100
Bar rupture at Blast 5

Figure 5-10 Photographs; effects of concrete type in 100 mm slabs

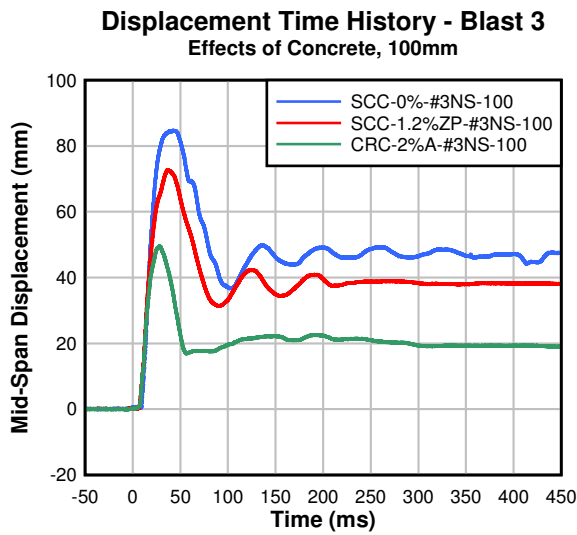


a) Blast 3

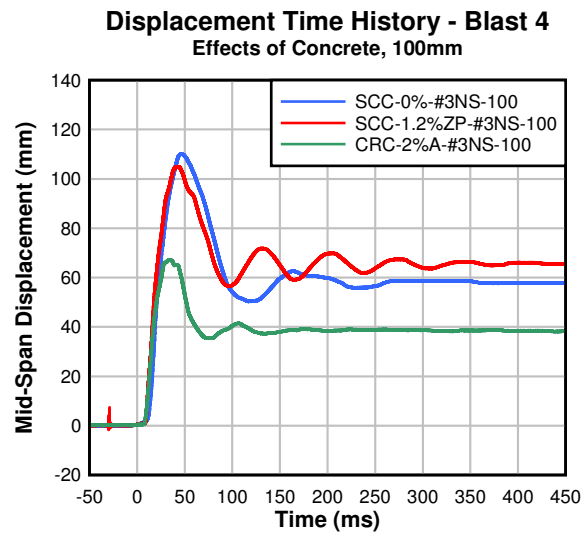


b) Blast 4

Figure 5-11 Maximum and residual displacements; effects of concrete type in 100 mm slabs



a) Blast 3



b) Blast 4

Figure 5-12 Displacement time histories; effects of concrete type in 100 mm slabs

5.3.3. EFFECTS OF CONCRETE TYPE: SCC vs. SFRC vs. UHPFRC – HS SPECIMENS

When combined with high strength steel reinforcement such as MMFX₂, it can be questioned whether or not the type of concrete has a major influence on the response of a reinforced concrete slab. Hence, the control specimen SCC-0%-#3HS-100, the SFRC specimen SCC-1.2%ZP-#3HS-100 and the UHPFRC specimen CRC-2%A-#3HS-100 were designed with the aforementioned high strength steel reinforcement bars for the purpose of this assessment. Relevant photographs are included in Figure 5-13. Comparative displacement bar charts and time histories for blasts 4 and 5 are provided in Figure 5-14 and Figure 5-15.

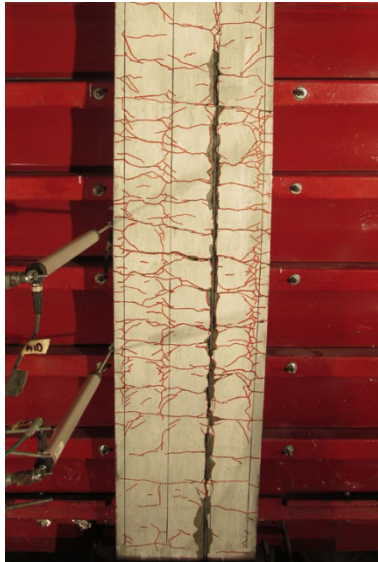
The UHPFRC specimen CRC-2%A-#3HS-100 had a greater performance than the other two specimens, reducing significantly the magnitude of the mid-span displacements and providing a better control of cracking. When compared with the plain concrete specimen SCC-0%-#3HS-100 at blast 4, the maximum and residual displacements both decreased by 21.9%. At blast 5, the mid-span displacements were reduced by 23.4% for the maximum and 39.5% for the residual.

When compared with the SFRC specimen SCC-1.2%ZP-#3HS-100, the UHPFRC specimen showed a similar trend. At blast 4, the maximum and residual mid-span displacements diminished by 26.6% and 39.8% respectively. For blast 5, the mid-span displacements were reduced by 34.2% for the maximum and 61.6% for the residual.

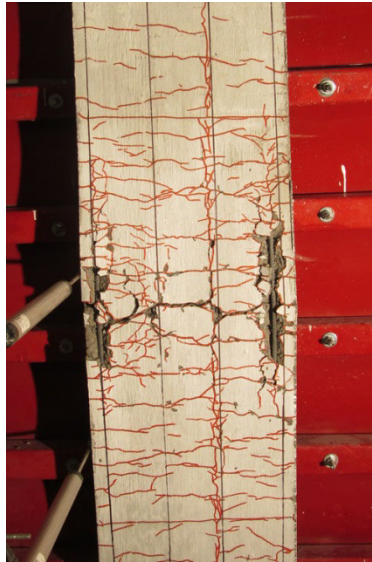
To complete the loop, when comparing the SFRC specimen with the control specimen an increase in the magnitude of the response could be observed. For instance, at blast 4, the maximum and residual mid-span displacements augmented by 6.3% and 29.7% respectively. For blast 5, the mid-span displacements were greater by 16.4% for the maximum and 57.6% for the residual.

It should be noted that the steel fibres in SFRC were meant to provide better cracking control and were not expected to decrease the displacements but rather render the concrete mix more ductile. While rebar was exposed in Figure 5-13(b), it was not as important as the plain concrete's exposure in Figure 5-13(a). When implementing the UHPFRC mix, tensile cracking is barely visible until failure; at blast 5, in Figure 5-13(c), the maximum crack width was 2.00 mm.

In terms of failure magnitude, all three specimens were capable of resisting the pressure and impulse of blast 5 without major displacements. As for their failure mode, SCC-0%-#3HS-100 resulted in a total demolition of the specimen at blast 6 as no fibres could retain the concrete chunks together after cracking; this can be seen from the high speed video screenshot from Figure 5-13(d). Specimen CRC-2%A-#3HS-100, which controlled fragmentation, failed in tension due to the tensile reinforcement bars rupturing which is also a rather brittle and sudden failure mode; a photograph of the end result was included in Figure 5-13(e). This could be due to the high strength required to break UHPFRC.



a) SCC-0%-#3HS-100
Exposed rebar at Blast 5



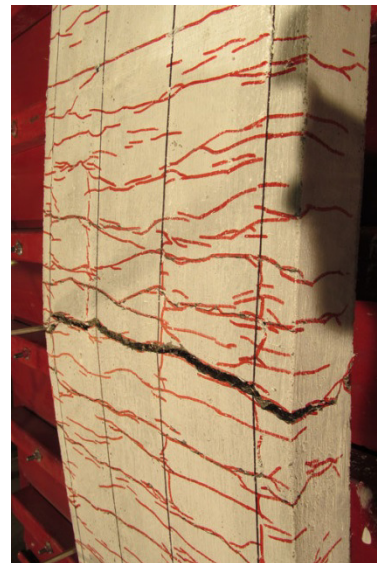
b) SCC-1.2%ZP-#3HS-100
Exposed rebar at Blast 5



c) CRC-2%A-#3HS-100
Tensile cracking at Blast 5

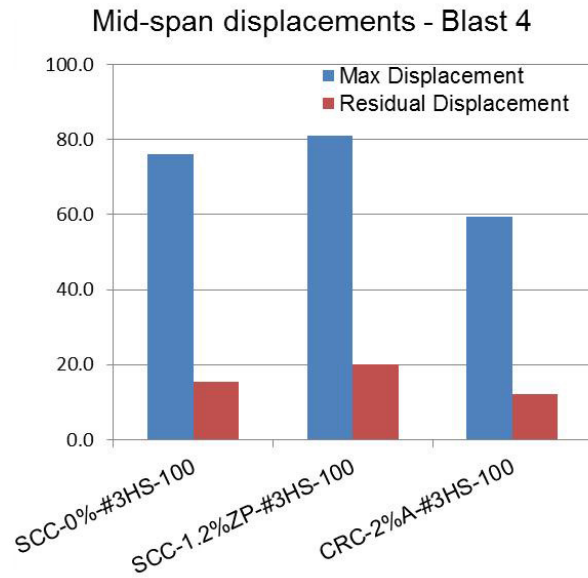


d) SCC-0%-#3HS-100
Excessive fragmentation at Blast 6

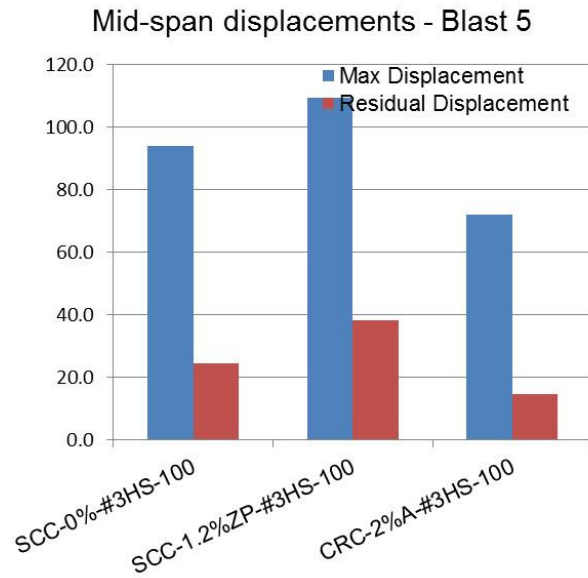


e) CRC-2%A-#3HS-100
Tensile cracking and bar rupture at Blast 6

Figure 5-13 Photographs; effects of concrete type with high strength steel specimens

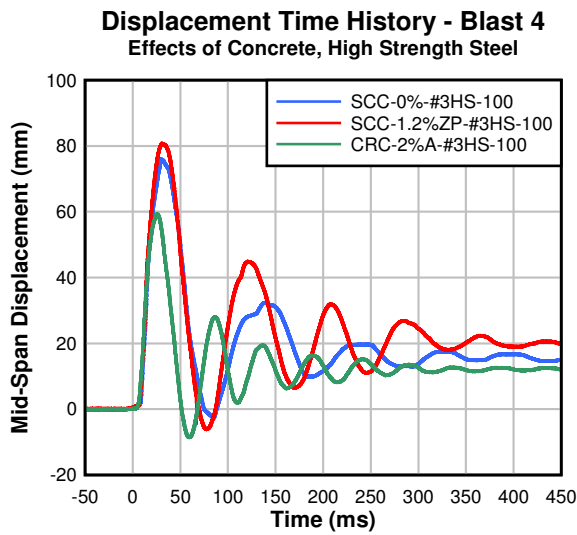


a) Blast 4

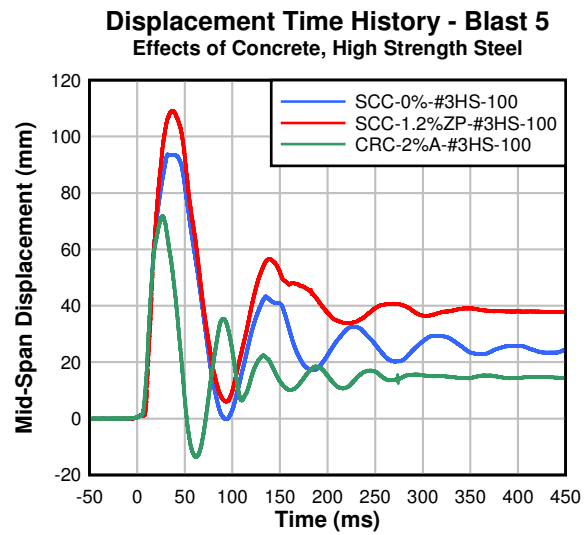


b) Blast 5

Figure 5-14 Maximum and residual displacements; effects of concrete type with high strength steel



a) Blast 4



b) Blast 5

Figure 5-15 Displacement time histories; effects of concrete type with high strength steel

5.4. EFFECTS OF STEEL REINFORCEMENT TYPE

Based on the data obtained from the experimental program, reinforced concrete slabs seemed to benefit from the use of high strength steel reinforcement, especially in terms of controlling displacements during similar blast loads and improving the overall blast resistance. The following sub-sections demonstrate this observation within each type of concrete mix.

5.4.1. EFFECTS OF STEEL REINFORCEMENT – PLAIN CONCRETE SPECIMENS

The control specimen SCC-0%-#3NS-100 and SCC-0%-#3HS-100 were both designed with a depth of 100 mm and normal strength concrete without steel fibres. Relevant photographs of the slabs after testing are included in Figure 5-16. Comparative displacement bar charts and time histories for blasts 3 and 4 are provided in Figure 5-17 and Figure 5-18.

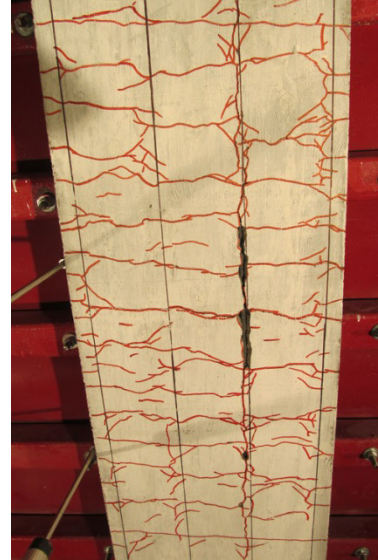
In regards to mid-span displacements, the provision of high strength steel had a major impact on both the maximum and residual measurements. When compared with the control specimen for blast 3, the high strength specimen had a reduction of 31.3% for the maximum displacement and 90.8% for residual displacement. For blast 4, the displacement decreased by 33.3% for the maximum and 79.5% for residual. The responses could not be compared for blast 5 due to a lack of data for the control specimen.

In addition to reducing the magnitude of displacements, the high strength steel specimen was able to resist a higher load than the control specimen prior to failure; it achieved failure during blast 6 unlike the control specimen which failed during blast 5. However, both specimens exhibited high amounts of secondary fragmentation during their last test which was not a desirable outcome. This last behaviour can be observed on the high speed video screenshots from Figure 5-16(c) and Figure 5-16(d).

One can suspect that this type of failure involves the lack of transverse reinforcement and steel fibres from both specimens. But it should be noted that prior to reaching the failure load, the high strength steel did reduce significantly tensile cracking for similar blasts as seen between Figure 5-16(a) and Figure 5-16(b).



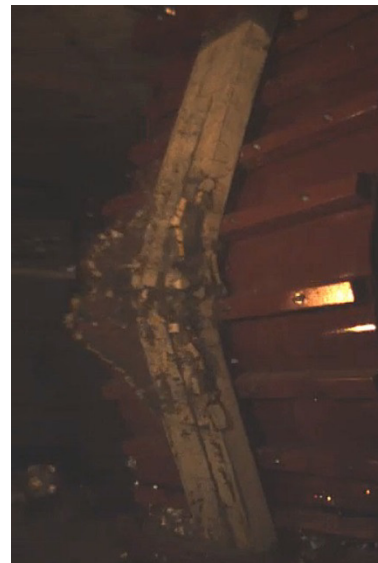
a) SCC-0%-#3NS-100
Damage at Blast 4



b) SCC-0%-#3HS-100
Damage at Blast 4

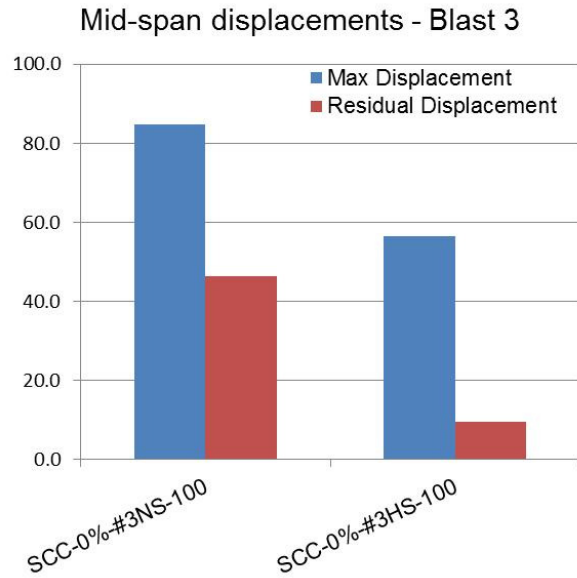


c) SCC-0%-#3NS-100
Excessive fragmentation during Blast 5

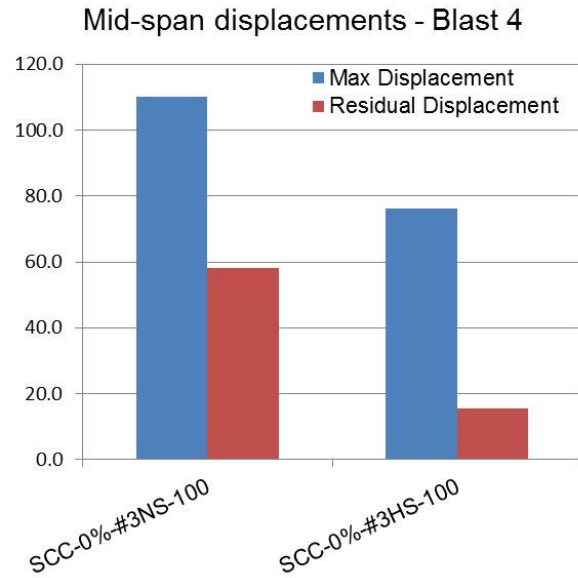


d) SCC-0%-#3HS-100
Excessive fragmentation during Blast 6

Figure 5-16 Photographs; effects of steel reinforcement with plain concrete specimens

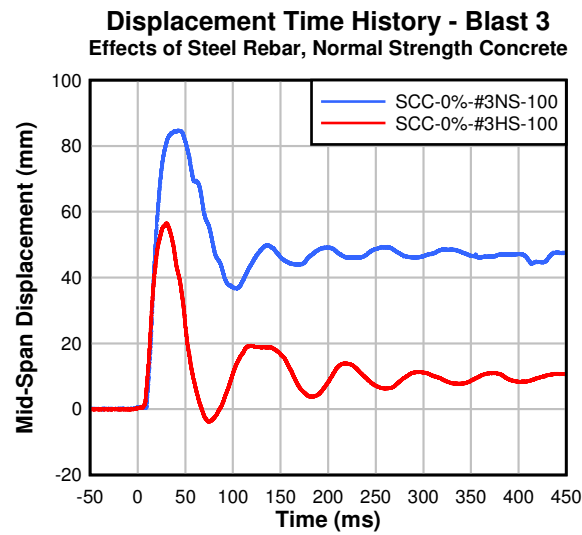


a) Blast 3

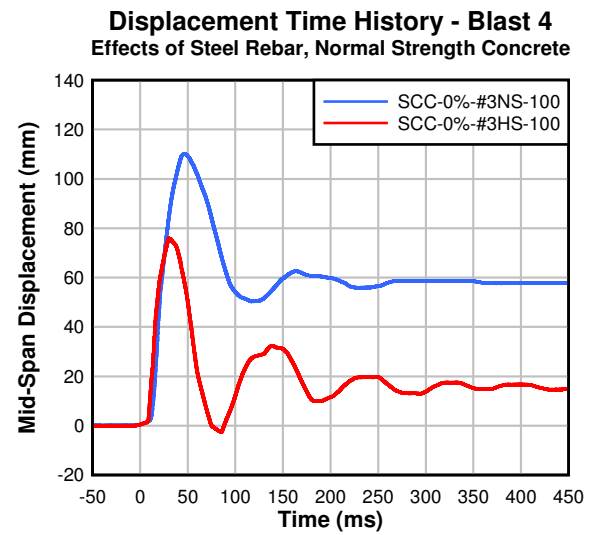


b) Blast 4

Figure 5-17 Maximum and residual displacements; effects of steel reinforcement with plain concrete



a) Blast 3



b) Blast 4

Figure 5-18 Displacement time histories; effects of steel reinforcement with plain concrete

5.4.2. EFFECTS OF STEEL REINFORCEMENT – SFRC SPECIMENS

Specimens SCC-1.2%ZP-#3NS-100 and SCC-1.2%ZP-#3HS-100 were both designed with a depth of 100 mm and similar steel fibre reinforced concrete mixes. Relevant photographs are included in Figure 5-19. Comparative displacement bar charts and time histories for blasts 3 and 4 are provided in Figure 5-20 and Figure 5-21; these also include data from the main control specimen SCC-0%-#3NS-100.

In regards to mid-span displacements, the provision of high strength steel in SFRC significantly improved the response of the one-way slab. When compared to the main control specimen at blast 3, the specimen with combined fibres and high strength steel (SCC-1.2%ZP-#3HS-100) showed a reduction of 27.8% of the maximum displacement and 75.6% of the residual displacement. For blast 4, the displacements decreased by 26.6% for the maximum and 65.4% for the residual measurements.

A similar trend could be observed when this high strength steel SFRC specimen was compared to SCC-1.2%ZP-#3NS-100, for which the only design difference is the steel type. For blast 3, the high strength specimen showed a reduction of 15.9% of the maximum displacement and 70.3% of the residual displacement. For blast 4, the displacements diminished by 23.0% for the maximum and 69.3% for the residual measurements.

The high strength steel SFRC specimen was not tested with blast 6 but was considered to have successfully resisted its last attempt unlike the other two specimens which had excessive displacements at blast 5. The response of SCC-1.2%ZP-#3HS-100 also led to lesser crack widths and lower amounts of fragmentation as it can be seen on the photographs of Figure 5-19. Indeed, at blast 4, which was prior to the failure any of the currently compared specimens, the maximum tensile crack width of the high strength steel specimen was 1.50 mm against 12.5 mm for the regular steel SFRC specimen.



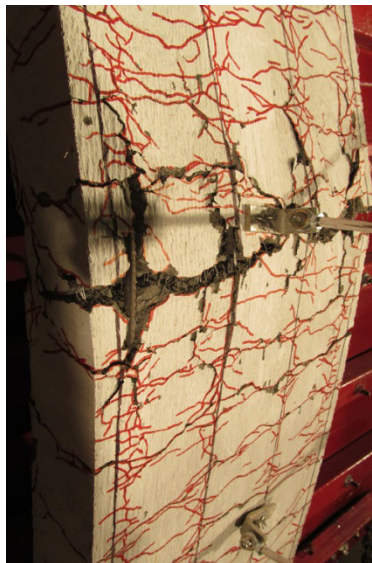
a) SCC-0%-#3NS-100
Damage after Blast 4



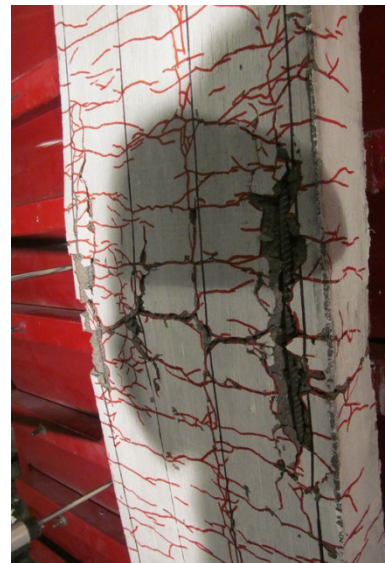
b) SCC-1.2%ZP-#3HS-100
Minimal cracking after Blast 4



c) SCC-0%-#3NS-100
Bar exposure after Blast 5

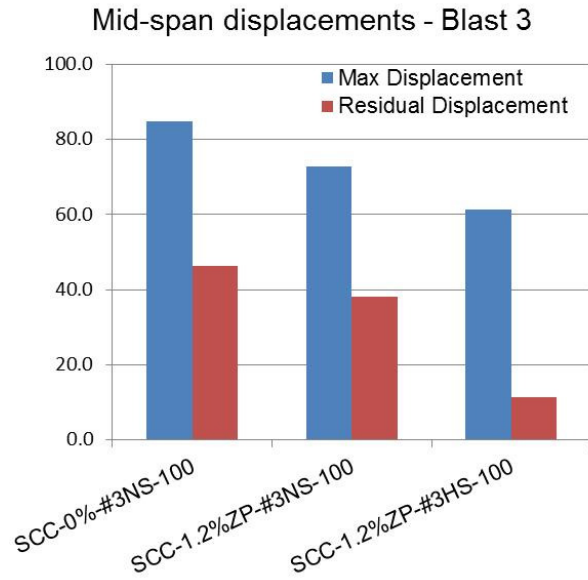


d) SCC-1.2%ZP-#3NS-100
Bar exposure after Blast 5

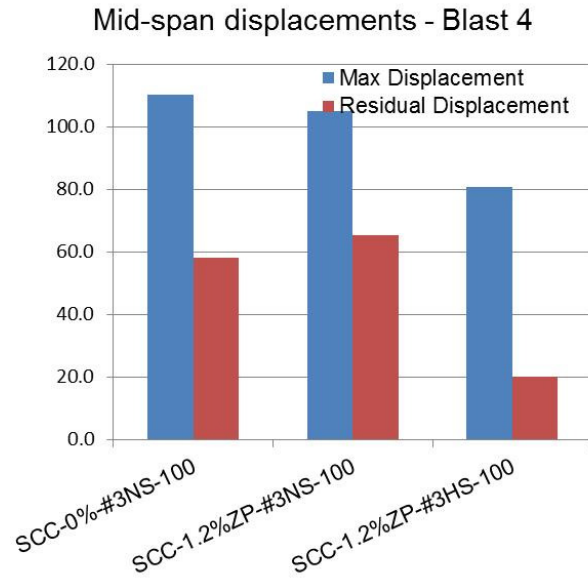


e) SCC-1.2%ZP-#3HS-100
Bar exposure after Blast 5

Figure 5-19 Photographs; effects of steel reinforcement with SFRC specimens

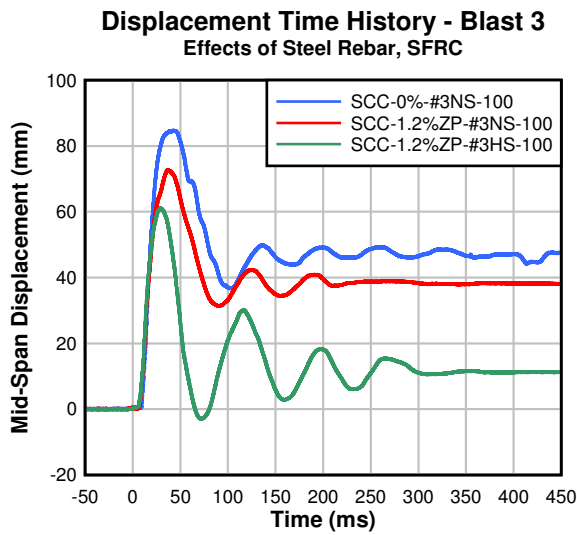


a) Blast 3

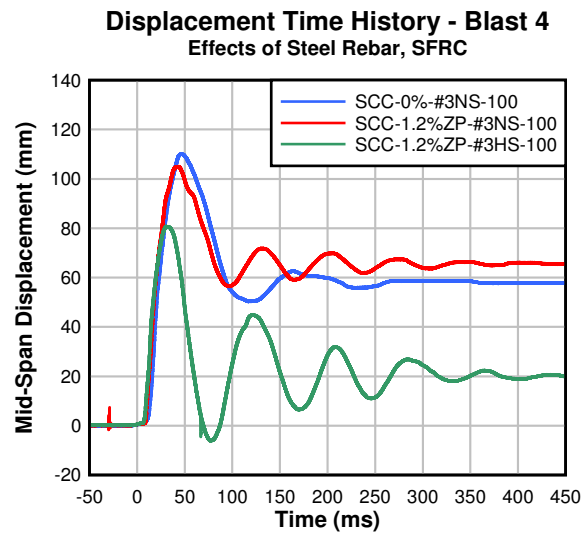


b) Blast 4

Figure 5-20 Maximum and residual displacements; effects of steel reinforcement with SFRC



a) Blast 3



b) Blast 4

Figure 5-21 Displacement time histories; effects of steel reinforcement with SFRC

5.4.3. EFFECTS OF STEEL REINFORCEMENT – UHPFRC SPECIMENS

Specimens CRC-2%A-#3NS-100 and CRC-2%A-#3HS-100 were designed with a depth of 100 mm and similar ultra-high performance fibre reinforced concrete mixes. Relevant photographs are included in Figure 5-22. Comparative displacement bar charts and time histories for blasts 4 and 5 are provided in Figure 5-23 and Figure 5-24; these also include data from the main control specimen SCC-0%-#3NS-100.

In regards to the mid-span displacements, the provision of high strength steel in UHPFRC specimens followed a similar trend than the previous sets of comparison with plain concrete and SFRC, improving the response of the one-way reinforced concrete slab. When compared with the control specimen SCC-0%-#3NS-100 at blast 4, the specimen with combined UHPFRC and high strength steel showed a reduction of the maximum and residual displacements by 46.1% and 79.2% respectively. During blast 5, no data was recorded for the control specimen and it had significant secondary fragmentation, unlike CRC-2%A-#3HS-100 which did not fail until the 6th blast.

The performance trend of high strength steel could also be observed when comparing both UHPFRC specimens. Against CRC-2%A-#3NS-100, the high strength steel specimen CRC-2%A-#3HS-100 showed improvements in the behaviour of the slab when subjected to equivalent dynamic blast loads. At blast 4, the maximum and residual mid-span displacements were reduced by 11.7% and 68.5% respectively. At blast 5, the response was lowered by 66.3% for the maximum displacement and 89.3% for the residual. It should also be noted that the high strength steel specimen was considered to have successfully resisted blast 5 and did not fail until tension bars rupture during blast 6.

In terms of crack control, the high strength steel specimen showed lesser crack widths than the other UHPFRC specimen due to its less pronounced responses. This behaviour can be seen on photographs (a) through (c) of Figure 5-22, where Figure 5-22(a) is the control specimen prior to failure. The same trend was also observed for the final blast test on photographs (d) through (f) of Figure 5-22.

It should be noted that the provision of high strength steel did not change the final outcome of the reinforced concrete slab as both UHPFRC specimens experienced tensile bar rupture. A more ductile bar would be required to ensure a less brittle failure mode.



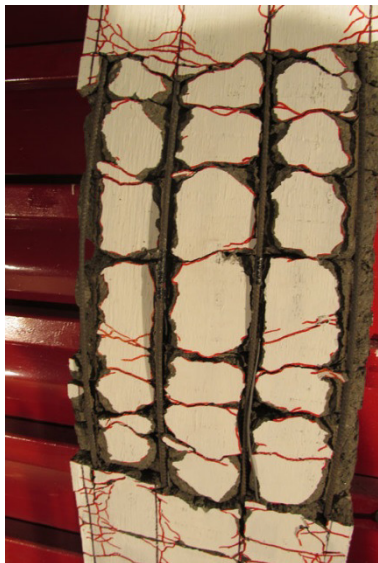
a) SCC-0%-#3NS-100
Damage after Blast 4



b) CRC-2%A-#3NS-100
Tensile cracking after Blast 4



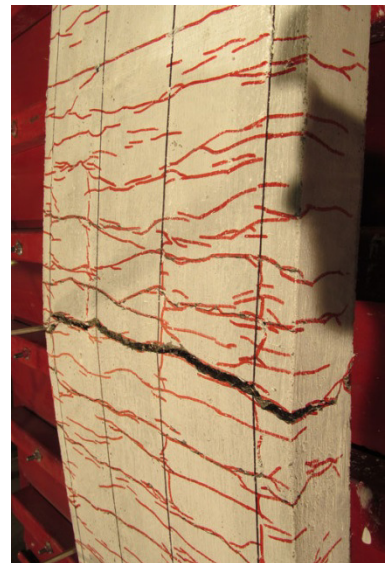
c) CRC-2%A-#3HS-100
Tensile cracking after Blast 5



d) SCC-0%-#3NS-100
Bar exposure after Blast 5

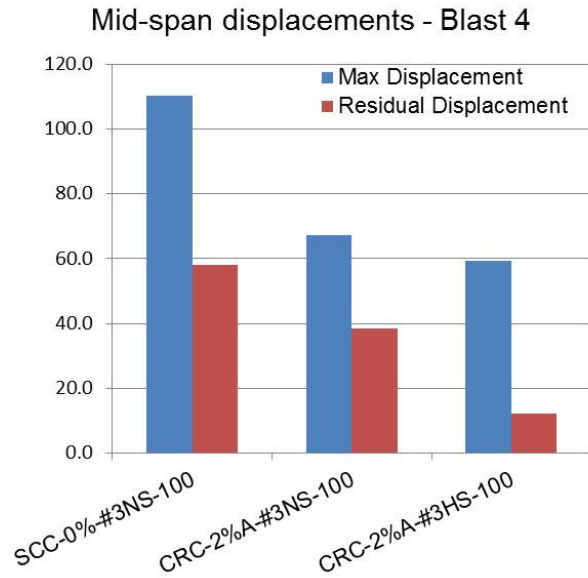


e) CRC-2%A-#3NS-100
Bar rupture failure after Blast 5

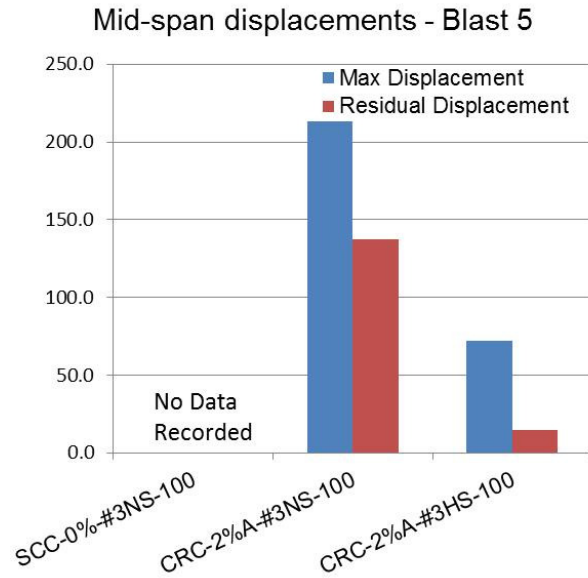


f) CRC-2%A-#3HS-100
Bar rupture failure after Blast 6

Figure 5-22 Photographs; effects of steel reinforcement with UHPFRC specimens

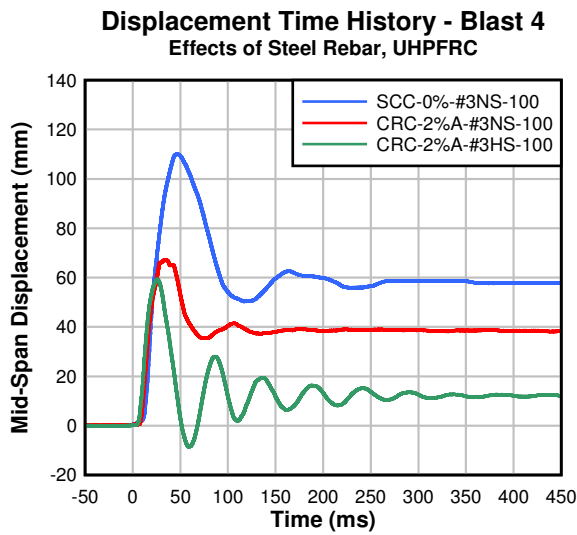


a) Blast 4

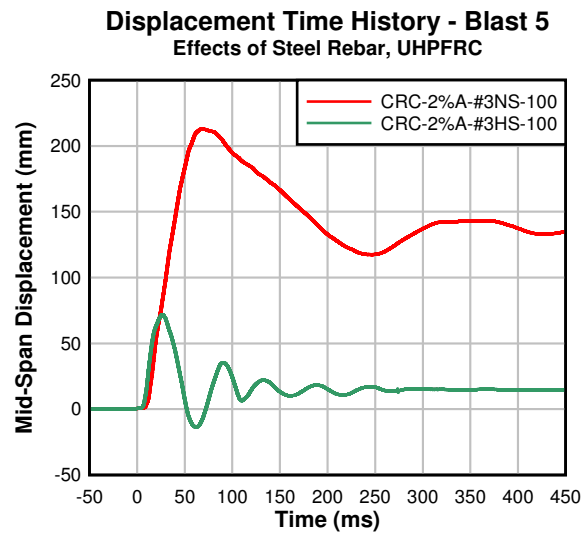


b) Blast 5

Figure 5-23 Maximum and residual displacements; effects of steel reinforcement with UHPFRC



a) Blast 4



b) Blast 5

Figure 5-24 Displacement time histories; effects of steel reinforcement with UHPFRC

5.4.4. EFFECTS OF STEEL REINFORCEMENT – STAINLESS STEEL SPECIMEN

Specimens CRC-2%A-#3NS-100 and CRC-2%A-#3SS-100 were designed with a depth of 100 mm and similar ultra-high performance fibre reinforced concrete mixes. Relevant photographs are included in Figure 4-25. Comparative displacement bar charts and time histories for blasts 4 and 5 are provided in Figure 4-26 and Figure 4-27; these also include data from the main control specimen SCC-0%-#3NS-100.

The provision of high ductility stainless steel reinforcement in UHPFRC specimens greatly improved the response of the one-way slabs. When compared with the control specimen SCC-0%-#3NS-100 at blast 4, the specimen with combined UHPFRC and stainless steel reduced the maximum and residual displacements by 45.6% and 81.4% respectively. During blast 5, no data was recorded for the control specimen and it had significant secondary fragmentation unlike CRC-2%A-#3SS-100, which survived until Blast 6 pressures.

An increase in performance of the stainless steel specimen over the other similar UHPFRC specimen, CRC-2%A-#3NS-100, could also be observed through examination of the dynamic behaviour of the specimens. Indeed, at blast 4, the stainless steel specimen was capable of reducing the maximum and residual displacements by 11.0% and 71.9% respectively. At blast 5, the displacements diminished by 61.6% for the maximum and 81.6% for the residual. It should also be noted that during blast 5, the tensile bars of CRC-2%A-#3NS-100 ruptured, leading to the failure of the specimen and greater displacements.

Hence, the provision of the high ductility stainless steel in UHPFRC specimens does significantly improve the response of one-way slabs. In this study the use of stainless steel reduced displacements at equivalent blast loads while also making the slab capable of resisting higher dynamic loads without being subject to a brittle failure mode. In terms of crack control, the stainless steel specimen showed lesser crack widths through the entire testing process when compared with the other two specimens. This can be noted from Figure 4-25 in which photographs (c) and (f) represent the concerned specimen.



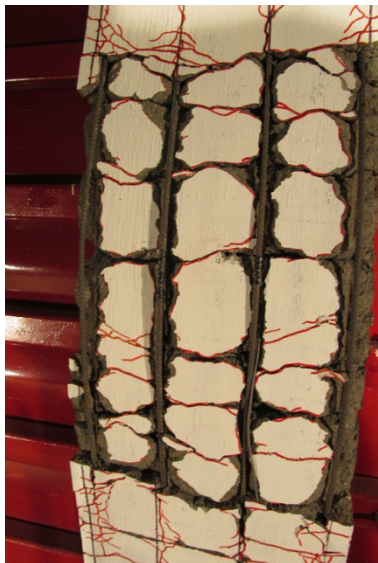
a) SCC-0%-#3NS-100
Damage after Blast 4



b) CRC-2%A-#3NS-100
Tensile cracking after Blast 4



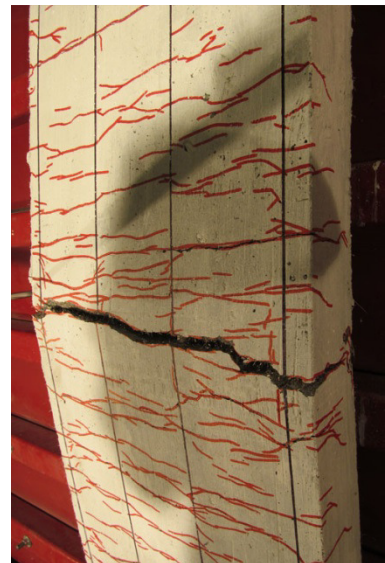
c) CRC-2%A-#3SS-100
Tensile cracking after Blast 5



d) SCC-0%-#3NS-100
Bar exposure after Blast 5

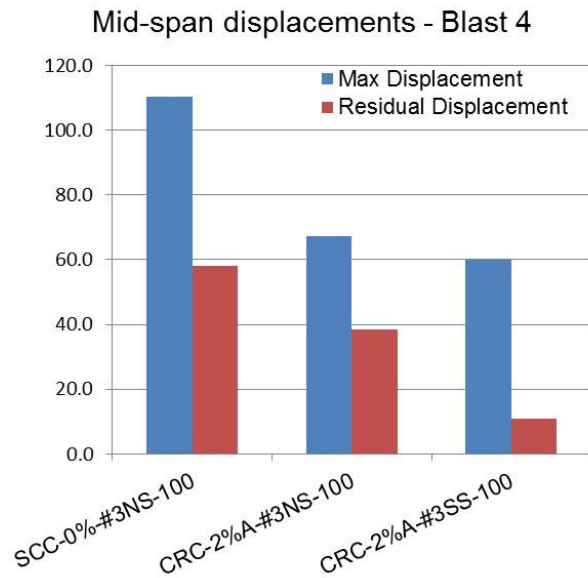


e) CRC-2%A-#3NS-100
Bar rupture after Blast 5

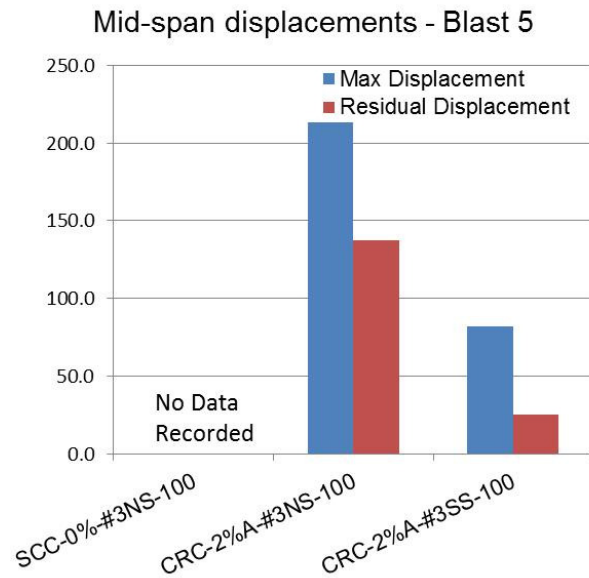


f) CRC-2%A-#3SS-100
Tensile cracking after Blast 6

Figure 5-25 Photographs; effects of stainless steel reinforcement

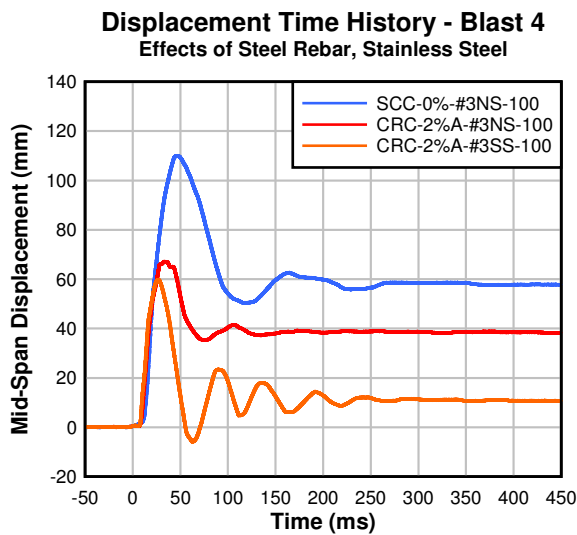


a) Blast 4

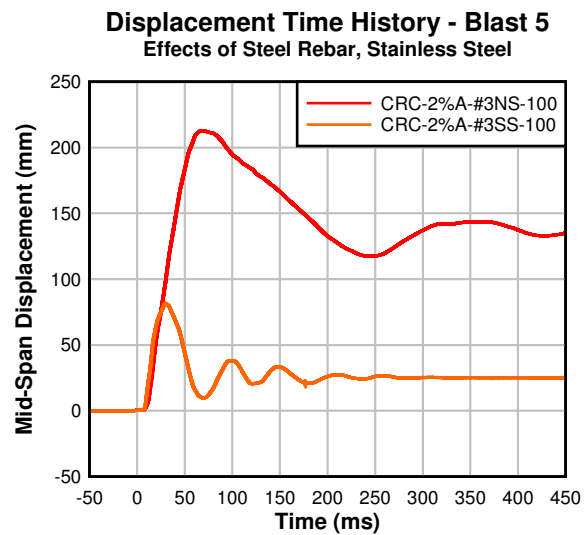


b) Blast 5

Figure 5-26 Maximum and residual displacements; effects of stainless steel reinforcement



a) Blast 4



b) Blast 5

Figure 5-27 Displacement time histories; effects of stainless steel reinforcement

5.4.5. EFFECTS OF STEEL REINFORCEMENT – HIGH STRENGTH STEEL vs. STAINLESS STEEL

Specimens CRC-2%A-#3HS-100 and CRC-2%A-#3SS-100 were designed with a depth of 100 mm and similar ultra-high performance fibre reinforced concrete mixes. The purpose of this sub-section is to observe differences between the behaviour of UHPFRC slabs with two different types of high performance steel reinforcement. Relevant photographs are included in Figure 5-28. Comparative displacement bar charts and time histories for blasts 5 and 6 are provided in Figure 5-29 and Figure 5-30.

As previously discussed, both these specimens showed significant improvements over traditional materials on different aspects such as the ability to resist higher blast loads prior to failure, to reduce the amplitude of the dynamic response, and to control cracking and fragmentation with greater success. In terms of lateral mid-span displacements, the high strength steel specimen CRC-2%A-#3HS-100 had a slightly better response during blast 5 than its stainless steel counterpart. The maximum and residual displacements were lower by 12.1% and 41.9% respectively. This can be explained by the higher yield strength of the MMFX₂ steel reinforcement bars.

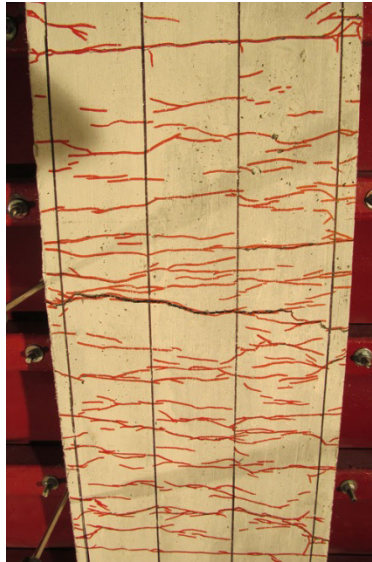
This trend did not follow during blast 6 due to the rupture of the high strength steel tensile bars as shown in Figure 5-28(c); it did not possess the high ductility of the stainless steel specimen (Figure 5-28(f)). The mid-span displacements increased by 7.9% for the maximum and diminished by 1.6% for the residual.

The high strength steel specimen generally had a minimal advantage over the response of the stainless steel specimen over the course of the testing process until failure occurred. Due to the lesser elongation limits of the MMFX₂ reinforcement bars, CRC-2%A-#3HS-100 had a lower ductility, hence an earlier rupture of the tensile bars in UHPFRC conditions than the comparable CRC-2%A-#3SS-100 specimen.

It should be however noted that both specimen configurations were effective at reducing cracking at similar levels, with the high strength steel specimen having slightly better results. At blast 5, the maximum tensile crack widths were 2 mm for the high strength steel specimen against 4 mm for stainless steel specimen. At blast 6, the maximums reached 10 mm against 13 mm.



a) CRC-2%A-#3HS-100
Tensile cracking after Blast 5



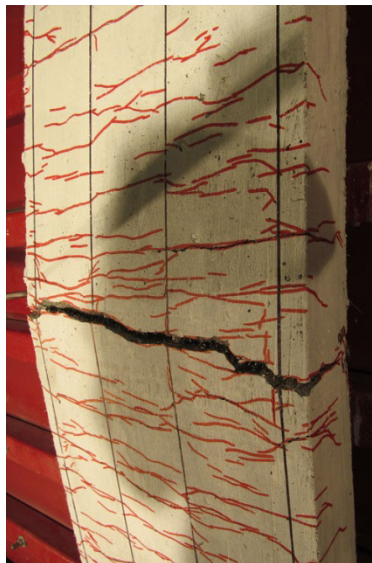
b) CRC-2%A-#3SS-100
Tensile cracking after Blast 5



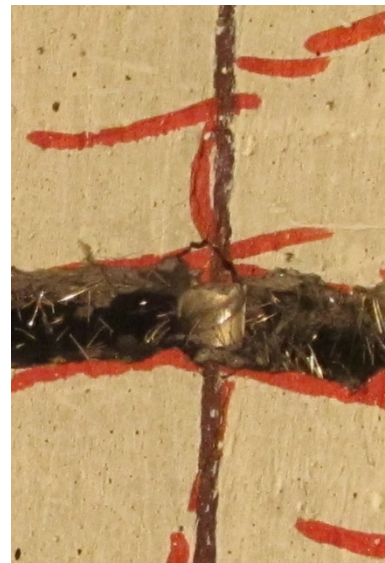
c) CRC-2%A-#3HS-100
Ruptured high strength steel bar



d) CRC-2%A-#3HS-100
Bar rupture failure after Blast 6

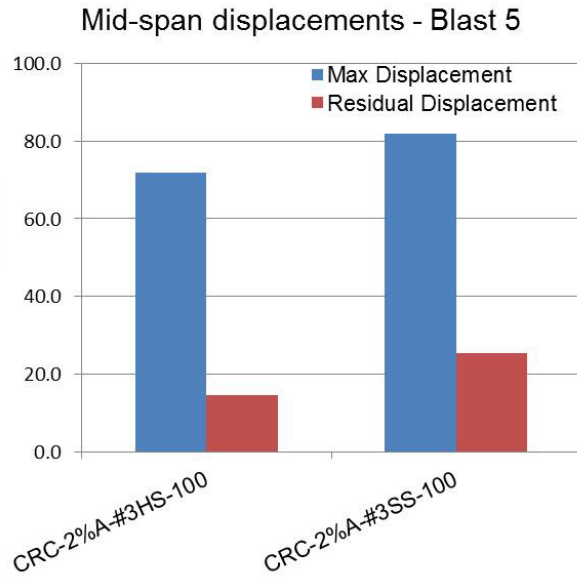


e) CRC-2%A-#3SS-100
Tensile cracking after Blast 6

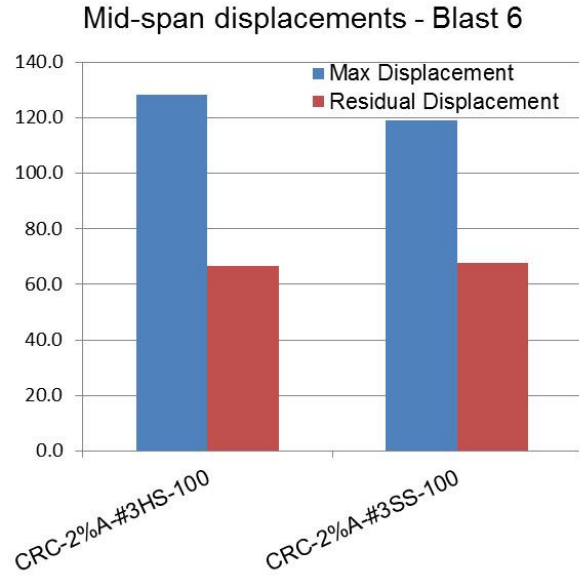


f) CRC-2%A-#3SS-100
Intact stainless steel bar

Figure 5-28 Photographs; high strength steel vs. stainless steel reinforcement

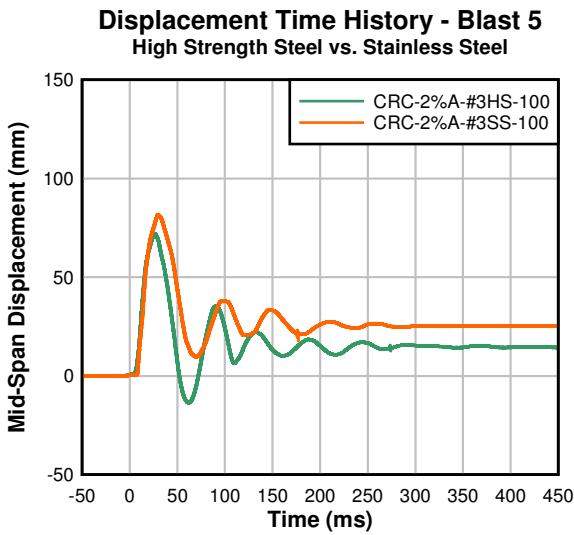


a) Blast 5

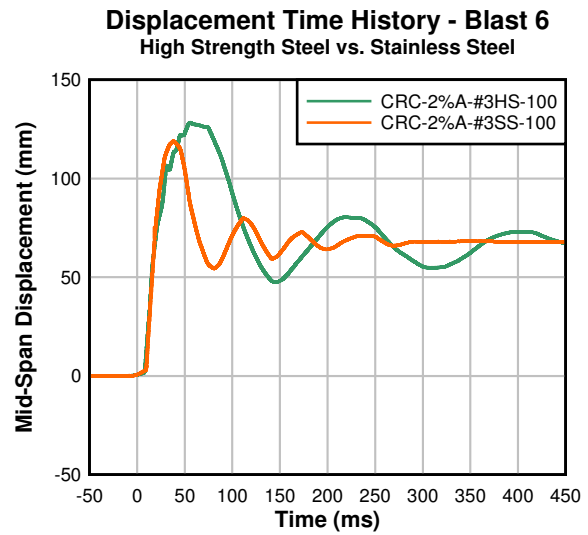


b) Blast 6

Figure 5-29 Maximum and residual displacements; high strength steel vs. stainless steel reinforcement



a) Blast 5



b) Blast 6

Figure 5-30 Displacement time histories; high strength steel vs. stainless steel reinforcement

5.5. EFFECTS OF STEEL FIBRES

The inclusion of steel fibres in concrete mixes has shown to be beneficial in regards to the behaviour of reinforced concrete slabs. It improved damage control, reducing cracking and secondary fragmentation. Specimens containing steel fibres also saw a boost in ductility. The following sub-sections provide observations in regards to the effects of steel fibres when changes are applied to the fibre content ratio or to the fibre type.

5.5.1. EFFECTS OF STEEL FIBRES – STEEL FIBRE CONTENT RATIO

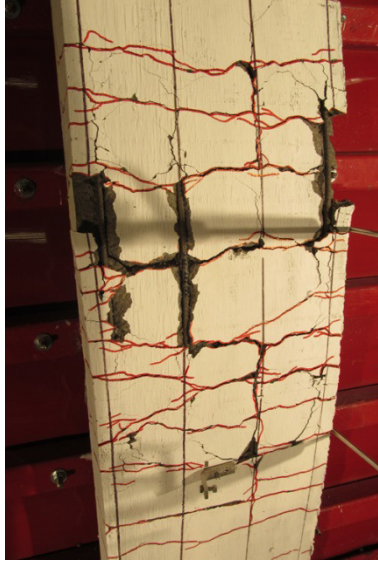
Specimens SCC-0.6%ZP-#3NS-100 and SCC-1.2%ZP-#3NS-100 were designed in order to observe the effects of varying the steel fibre volume ratio in SFRC slabs. Both specimens were specified as 100 mm slabs with regular steel reinforcement bars, normal strength concrete, and ZP-type fibres. These will also be compared to the control specimen SCC-0%-#3NS-100 in order to have three ratios: 0%, 0.6% and 1.2%. Relevant photographs are included in Figure 5-31. Comparative displacement bar charts and time histories for blasts 3 and 4 are provided in Figure 5-32 and Figure 5-33.

When compared to the control specimen, the high fibre content specimen SCC-1.2%ZP-#3NS-100 performs slightly better in terms of response. At blast 3, the maximum and residual displacements diminished by 14.4% and 17.7% respectively. At blast 4, the maximum displacement was reduced by 4.6% and the residual displacement was slightly higher by 12.7%.

Similarly, specimen SCC-1.2%ZP-#3NS-100 performed better than SCC-0.6%ZP-#3NS-100 which had half the steel fibre content. At blast 3, the maximum and residual displacements lowered by 4.3% and 7.5% respectively. At blast 4, the displacements were reduced by 13.9% for the maximum and 6.2% for the residual.

As mentioned above, the 1.2% ZP fibre content specimen generally performed better than the 0% and 0.6% specimens when displacements are compared. However, the benefits of higher fibre contents are better perceived when observing cracking and secondary fragmentation. Photographs from Figure 5-31 clearly show that varied fibre contents can improve the behaviour of a slab through a reduction in tensile cracking and concrete cover loss during the final blast tests.

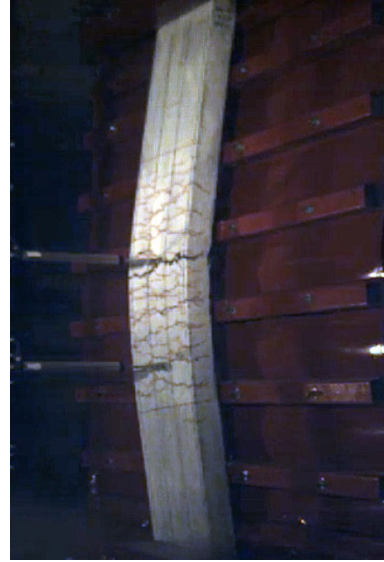
Further studies should be conducted to determine the optimal fibre content with the steel fibre that was analyzed in this section. Additional tests should also be considered for the fibre content of other types of steel fibres in regular concrete and UHPFRC.



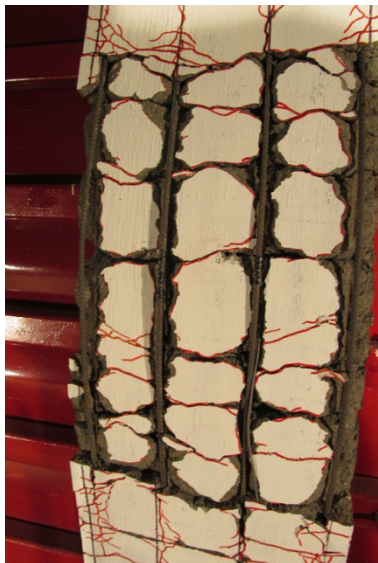
a) SCC-0%-#3NS-100
Damage after Blast 4



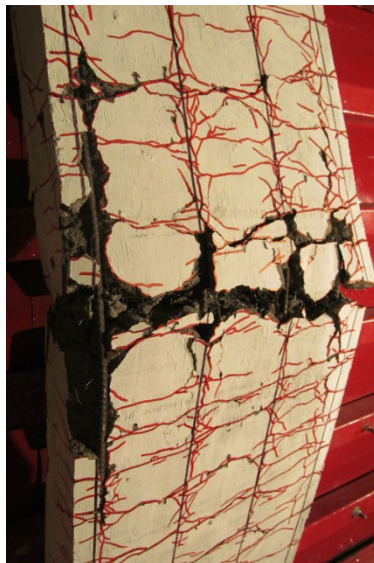
b) SCC-0.6%ZP-#3NS-100
Damage after Blast 4



c) SCC-0.1.2%ZP-#3NS-100
Tensile cracking at Blast 4



d) SCC-0%-#3NS-100
Exposed rebar after Blast 5

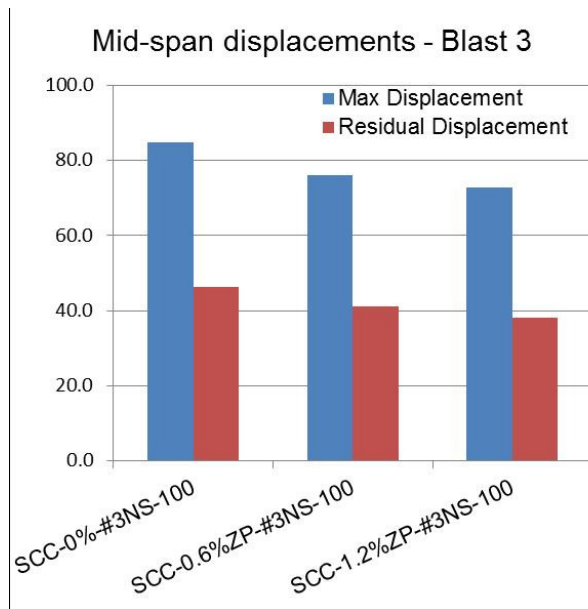


e) SCC-0.6%ZP-#3NS-100
Exposed rebar after Blast 5

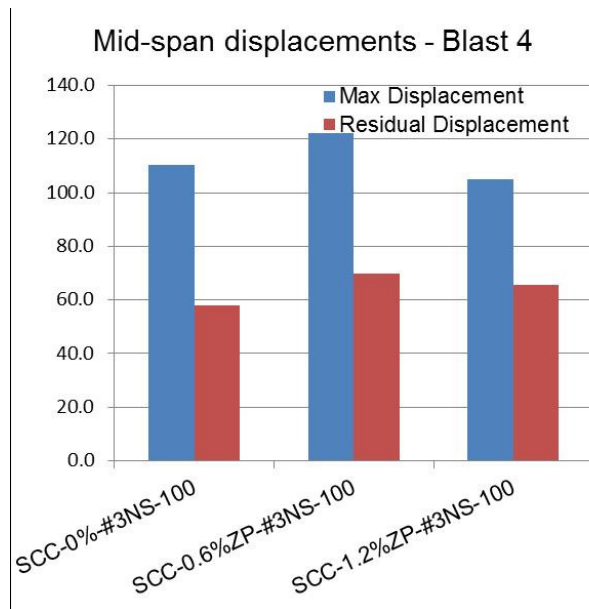


f) SCC-1.2%ZP-#3NS-100
Damage after Blast 5

Figure 5-31 Photographs; effects of steel fibre content ratio

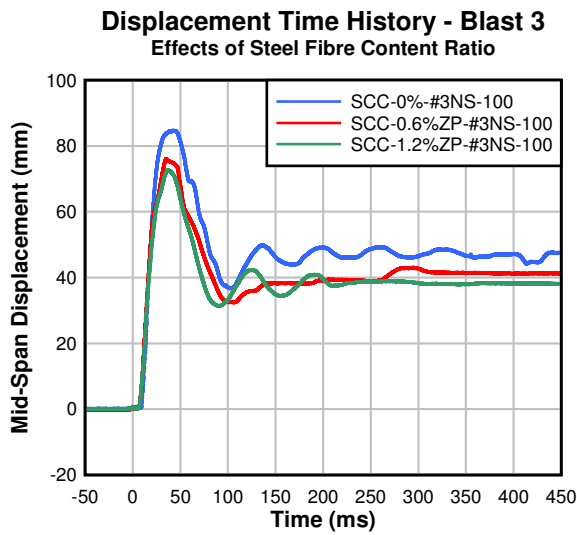


a) Blast 3

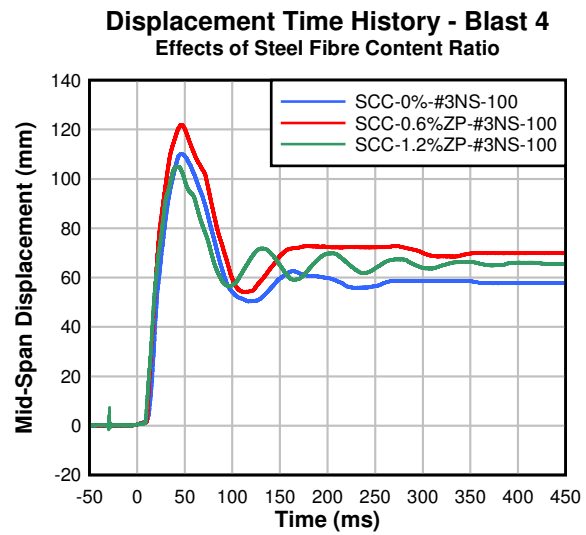


b) Blast 4

Figure 5-32 Maximum and residual displacements; effects of steel fibre content ratio



a) Blast 3



b) Blast 4

Figure 5-33 Displacement time histories; effects of steel fibre content ratio

5.5.2. EFFECTS OF STEEL FIBRES – STEEL FIBRE TYPE (SFRC)

Specimens SCC-0.6%ZP-#3NS-100 and SCC-0.6%BP-#3NS-100 were designed to compare the effects of different steel fibre types on the performance of SFRC slabs. Both specimens were specified as 100 mm slabs with regular steel reinforcement bars, normal strength concrete, and 0.6% fibre content. Relevant photographs are included in Figure 5-34. Comparative displacement bar charts and time histories for blasts 3 and 4 are provided in Figure 5-35 and Figure 5-36. The control specimen SCC-0%-#3NS-100 has been included for references purposes as well as SFRC specimen SCC-1.2%ZP-#3NS-100.

During blast 3, the performance of the SCC-0.6%BP-#3NS-100 specimen was comparable to its counterpart SCC-0.6%ZP-#3NS-100. The displacements decreased by 4.1% for the maximum and increased by 1.5% for the residual. For blast 4, the BP specimen could be considered to have a better performance with maximum and residual displacements being reduced by 18.8% and 14.9% respectively. This could be related to the damage from the previous blast as comparable between Figure 5-34(a) and Figure 5-34(b).

Since the BP steel fibres have a greater aspect ratio than the ZP fibres, it was also considered to compare SCC-0.6%BP-#3NS-100 with the higher fibre content specimen SCC-1.2%ZP-#3NS-100. At blast 3, the maximum and residual displacements were only higher by 0.3% and 9.7% respectively. At blast 4, the response is lower for the BP specimen with a diminution of 5.6% for the maximum displacement and 9.3% for the residual.

Between both types of fibre, the BP steel fibres with a greater aspect ratio and higher yield strength seem to perform better at higher blast loads than the ZP steel fibres for a similar fibre content ratio. They also seem to offer a better response than the ZP specimen with higher volumetric ratio. Cracking and secondary fragmentation is also improved when observing photographs (d) through (f) in Figure 5-34 as the extent of the tensile bar exposure and crack widths are reduced.

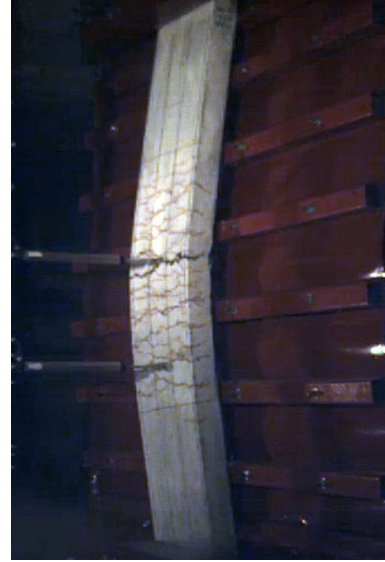
Further studies should be done to optimize the fibre content of the BP steel fibres as their properties seem beneficial on the behaviour of the slabs.



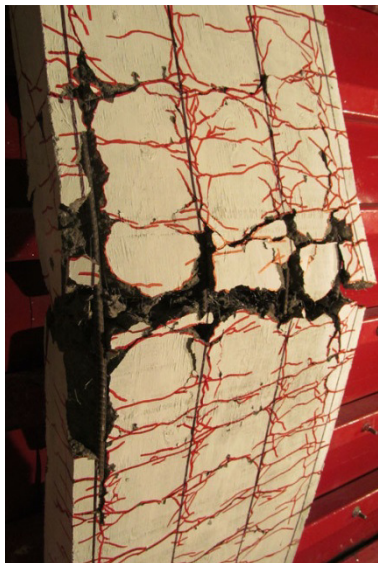
a) SCC-0.6%ZP-#3NS-100
Damage after Blast 4



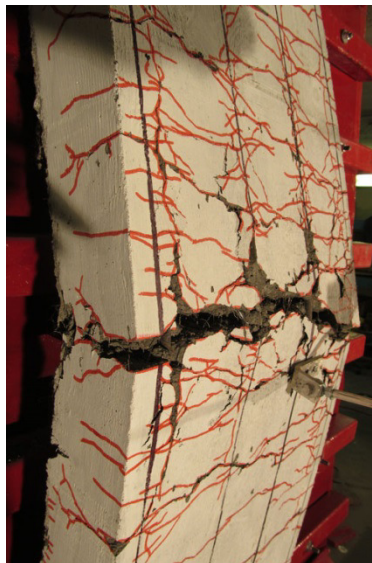
b) SCC-0.6%BP-#3NS-100
Damage after Blast 4



c) SCC-1.2%ZP-#3NS-100
Damage after Blast 4



d) SCC-0.6%ZP-#3NS-100
Exposed rebar after Blast 5

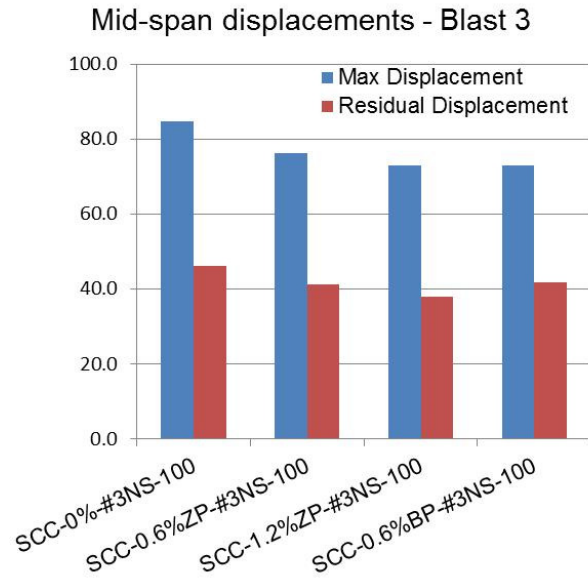


e) SCC-0.6%BP-#3NS-100
Tensile cracking after Blast 5

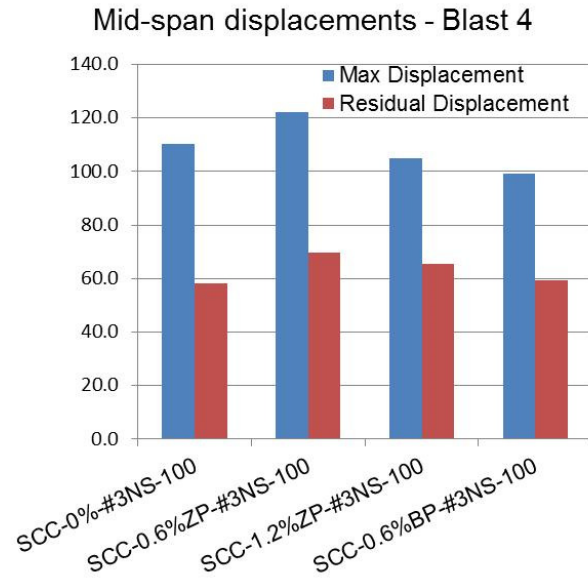


f) SCC-1.2%ZP-#3NS-100
Tensile cracking after Blast 5

Figure 5-34 Photographs; effects of steel fibre type in SFRC

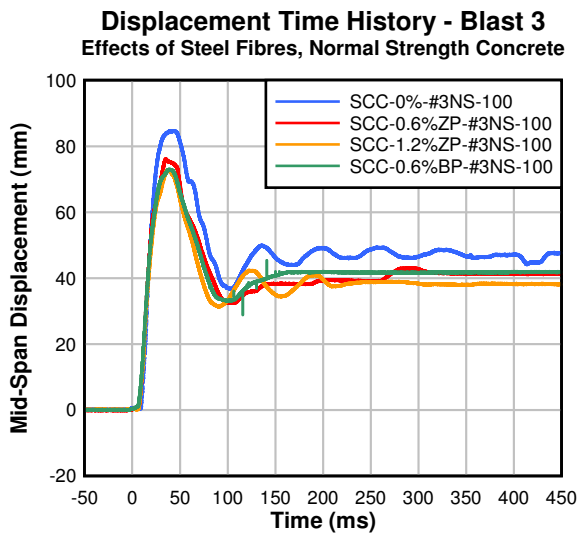


a) Blast 3

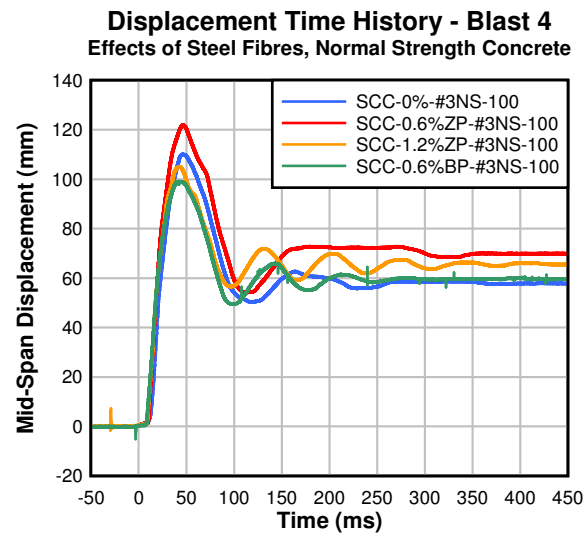


b) Blast 4

Figure 5-35 Maximum and residual displacements; effects of steel fibres in SFRC



a) Blast 3



b) Blast 4

Figure 5-36 Displacement time histories; effects of steel fibres in SFRC

5.5.3. EFFECTS OF STEEL FIBRES – STEEL FIBRE TYPE (UHPFRC)

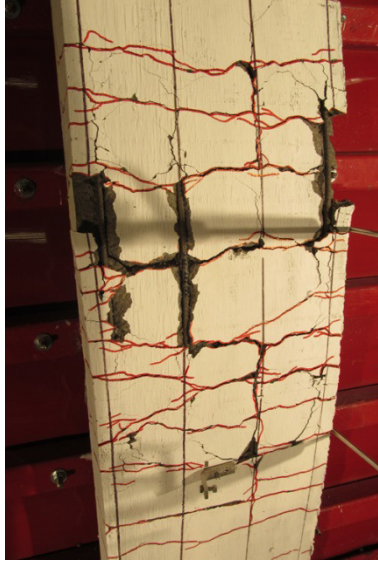
Specimens CRC-2%A-#3NS-100 and CRC-2%B-#3NS-100 were designed in order to observe the effects of different steel micro-fibres on the performance of UHPFRC slabs. Both specimens were constructed as 100 mm slabs with regular steel reinforcement bars, ultra-high performance fibre reinforced concrete, and 2% fibre content. Relevant photographs are included in Figure 5-37. Comparative displacement bar charts and time histories for blasts 3 and 4 are provided in Figure 5-38 and Figure 5-39. The control specimen SCC-0%-#3NS-100 was included for reference purposes.

In terms of response, CRC-2%B-#3NS-100 performed slightly worse than the other UHPFRC specimen during blast 3 with the residual displacement increasing by 19.2%. The maximum displacement could not be compared due to an LVDT detaching from the B specimen. For blast 4, the opposite was observed; the maximum and residual displacements reduced by 12.0% and 28.9% respectively.

It was also noticed that specimen CRC-2%B-#3NS-100 had smaller tensile crack widths than CRC-2%A-#3NS-100 during the final blast tests as summarized in Table 5-1. This would indicate that the B-type fibres have better crack control properties than the A-type fibres for a similar fibre content ratio.

However, it should be noted that both specimens did not have major tensile cracks at mid-span; the hinge formed itself lower than the mid-span for CRC-2%A-#3NS-100 while two hinges were formed below and above the mid-span for CRC-2%B-#3NS-100. This behaviour can be seen on Figure 5-37(e) and Figure 5-37(f) and may lead to incorrect conclusions in the correlation between the performance of each specimen and the response during the testing phase.

Additional tests would be required to obtain a more accurate understanding of the effect of micro-fibre type on mid-span response for comparison purposes. The fibre content ratio should also be varied to obtain an optimal parameter for each type of steel micro-fibre.



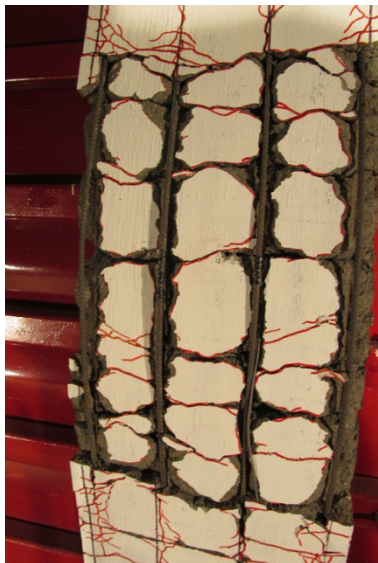
a) SCC-0%-#3NS-100
Damage after Blast 4



b) CRC-2%A-#3NS-100
Tensile cracking after Blast 4



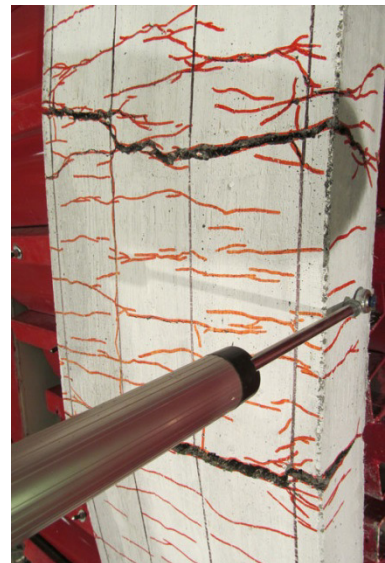
c) CRC-2%B-#3NS-100
Tensile cracking after Blast 4



d) SCC-0%-#3NS-100
Exposed rebar after Blast 5

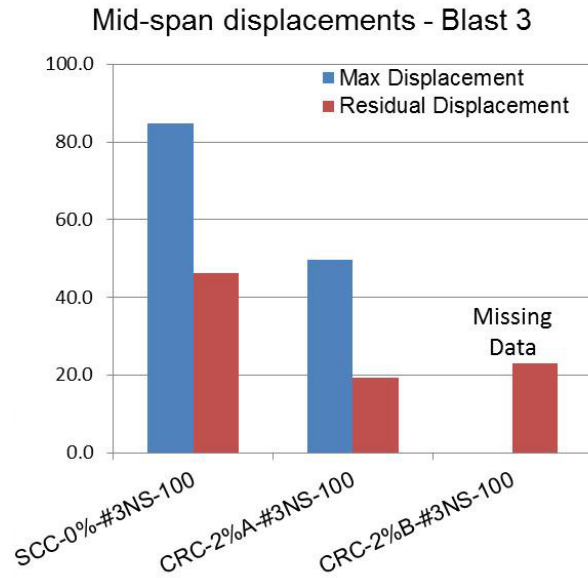


e) CRC-2%A-#3NS-100
Bar rupture at Blast 5

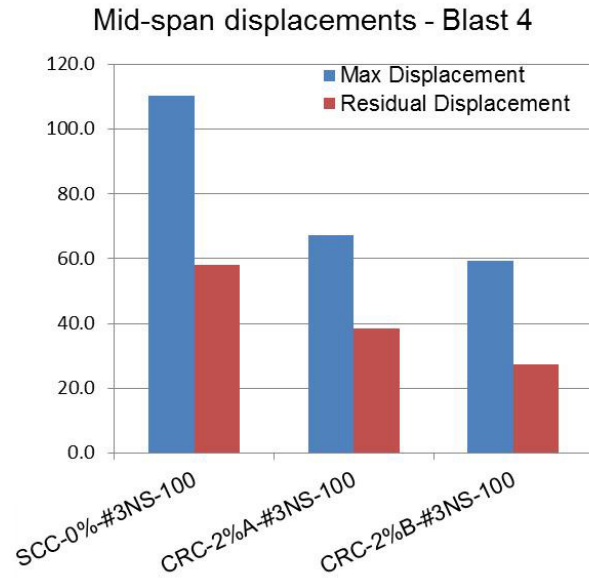


f) CRC-2%B-#3NS-100
Tensile cracking after Blast 5

Figure 5-37 Photographs; effects of steel fibre type in UHPFRC

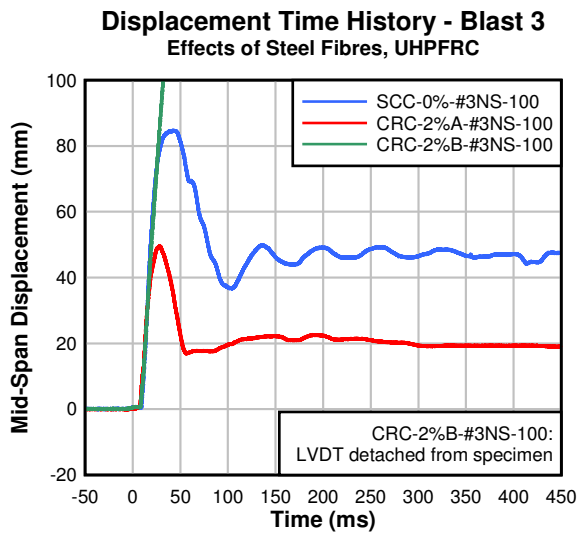


a) Blast 3

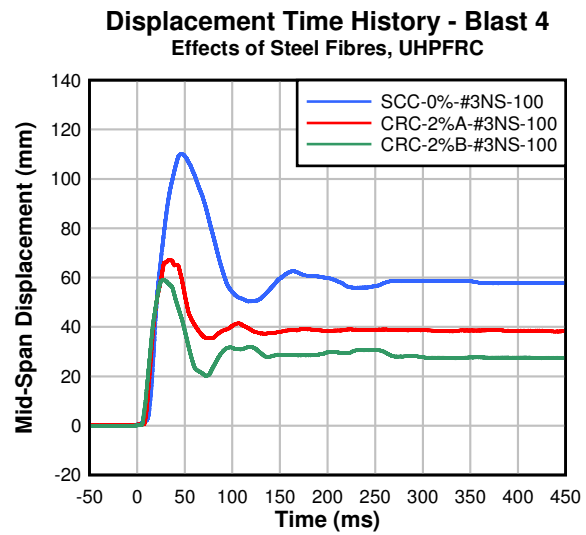


b) Blast 4

Figure 5-38 Maximum and residual displacements; effects of steel fibres in UHPFRC



a) Blast 3



b) Blast 4

Figure 5-39 Displacement time histories; effects of steel fibres in UHPFRC

5.6. EFFECTS OF CONCRETE SLAB DEPTH

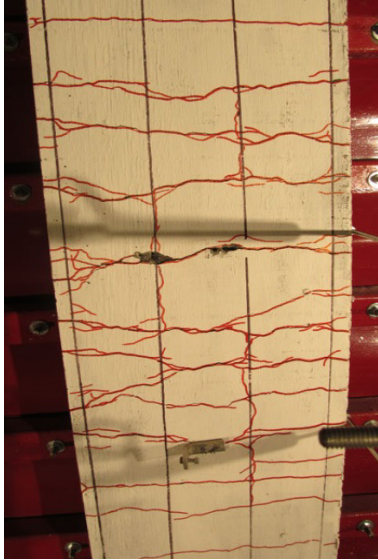
5.6.1. EFFECT OF CONCRETE SLAB DEPTH – CONTROL vs. UHPFRC

Specimen CRC-2%B-#3NS-80 is an 80 mm deep UHPFRC specimen designed in order to observe the benefits of high performance concrete when reducing the depth of a concrete slab. It will be compared to the control specimen SCC-0%-#3NS-100 which has the same type of steel reinforcement, normal strength concrete instead of UHPFRC, and a depth of 100 mm. Relevant photographs are included in Figure 5-40. Comparative displacement bar charts and time histories for blasts 3 and 4 are provided in Figure 5-41 and Figure 5-42.

A reduction in size of the concrete slab specimens in CRC-2%B-#3NS-80 leads to a lower mass for the system as well as a smaller internal lever arm for flexural resistance. In the case of two specimens with different depths but similar concrete mixes and steel reinforcement bars, this would mean a greater response on the part of the smaller specimen for a particular blast load. The purpose of CRC-2%B-#3NS-80 is to test the effects of replacing the concrete type in such situation for a high performance material.

While CRC-2%B-#3NS-80 was capable of performing similarly or even better than the 100 mm control specimen during the first two blast tests as seen on the previously included Figure 5-1 and Figure 5-2, was not stiff enough to ensure a lower response during the rest of the testing phase. For blast 3, the maximum and residual displacements increased by 15.7% and 14.5% respectively. During blast 4, the specimen's tensile rebar ruptured leading to maximum and residual displacements increases in the ranges of 76.3% and 173.1% respectively.

It should be noted that while the response was not improved, the UHPFRC specimen's failure mode was still an improvement over the control specimen as it reduced the amount of secondary fragmentation through the use of compact reinforced composite and steel fibres, as seen between Figure 5-42(c) and Figure 5-42(d). Tensile cracking was also focused at mid-span instead of spreading over the plastic hinge region of the specimen.



a) SCC-0%-#3NS-100
Damage after Blast 3



b) CRC-2%B-#3NS-80
Tensile cracking after Blast 3

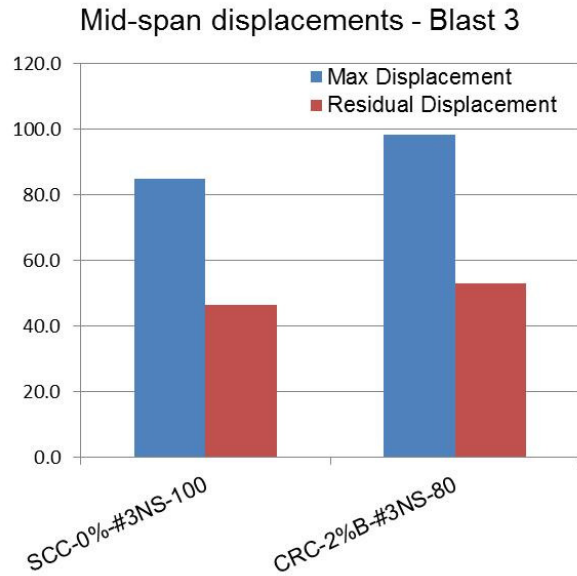


c) SCC-0%-#3NS-100
Excessive fragmentation at Blast 5

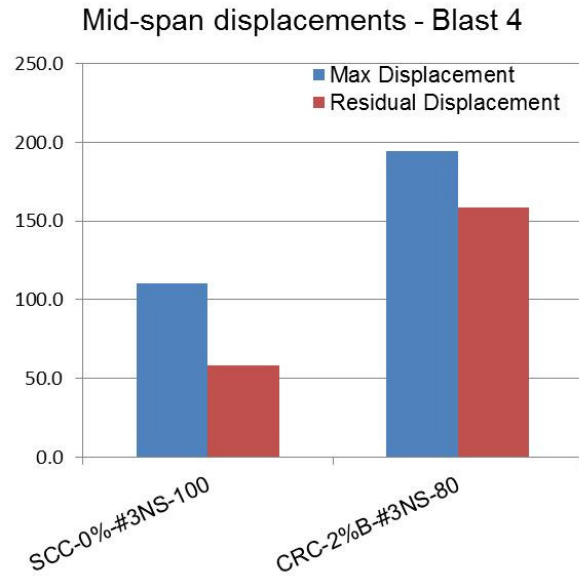


d) CRC-2%B-#3NS-80
Bar rupture at Blast 4

Figure 5-40 Photographs; effects of slab depth and UHPFRC

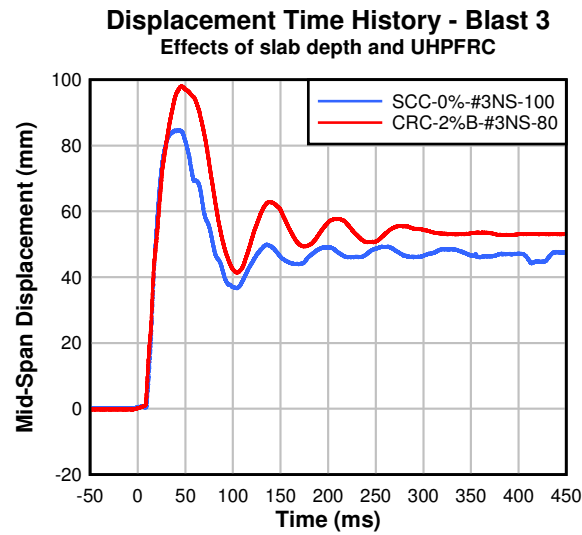


a) Blast 3

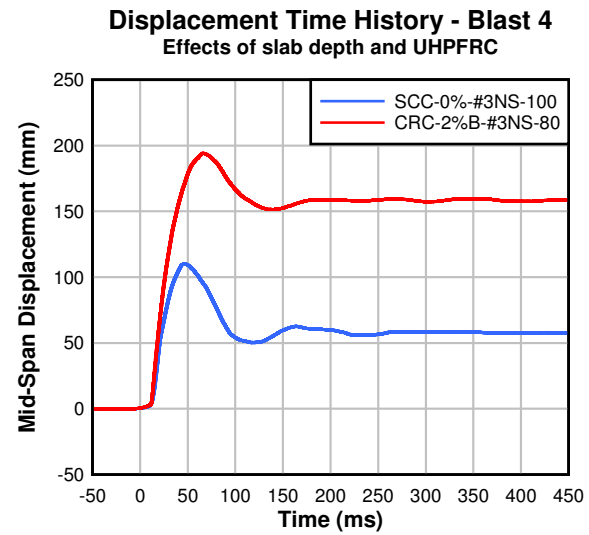


b) Blast 4

Figure 5-41 Maximum and residual displacements; effects of slab depth and UHPFRC



a) Blast 3



b) Blast 4

Figure 5-42 Displacement time histories; effects of slab depth and UHPFRC

5.7. DAMAGE TOLERANCE AND FRAGMENTATION

While the magnitude of the response of each specimen against blast loads is one of the principal criteria in regards to the behaviour and performance of these one-way slabs, the capacity of reducing damage and secondary fragmentation is also a rather important characteristic to be observed.

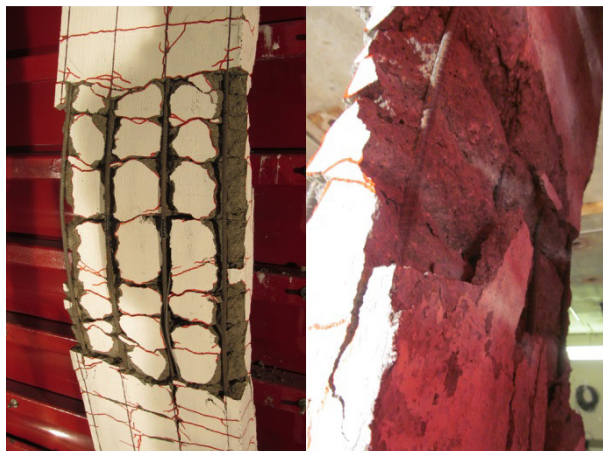
When compared to the control specimens, the SFRC and UHPFRC slabs showed improvements in the ability of controlling cracking, spalling and debris. The photographs of the control specimen SCC-0%-#3NS-100 provided in Figure 5-43(a) show that there was excessive crushing on the compression side as well as spalling resulting in the loss of concrete cover, the exposure of reinforcement bars, and an overall significant damage to the integrity of the slab. On the other hand, the SFRC specimen SCC-1.2%ZP-#3NS-100 included in Figure 5-43(b) show minimal damage on the compression side and reduced damage on the tension side after the same number of blast tests.

Further improvements could be seen with UHPFRC specimen CRC-2%A-#3NS-100 in Figure 5-43(c) for which only a single large tensile crack could be observed among hairline cracks after blast 5. It should be however noted that this specimen's failure was due to the rupture of the tensile reinforcement bars. In order to mitigate this situation, UHPFRC specimen CRC-2%A-#3SS-100 was designed with high ductility stainless steel reinforcement bars. The result, shown in Figure 5-43(d), was a specimen capable of resisting higher blast loads with enough ductility to ensure that the tensile bars don't rupture prior to the failure of the slab. Again, it should be noted that damage was minimal with a single major tensile crack at mid-span.

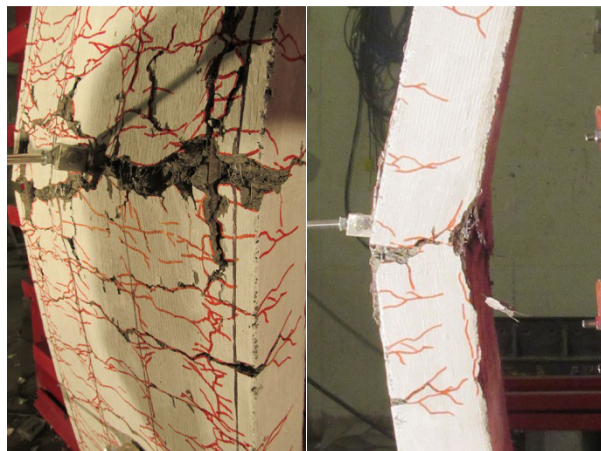
In regards to flying debris, the control specimens had significant secondary fragmentation as seen on the high-speed video of the final blast test; frames were included in Figure 5-44 for the control specimen with conventional steel, and Figure 5-45 for the control specimen with high strength steel. The size of the fragments varied from minuscule sand-like particles to larger debris which could fit in one's hand. They flew several meters after the blast but no setup was prepared to measure the potential impact of these fragments on the surrounding environment.

Steel fibres were used in order to reduce the amount of secondary fragmentation occurring due to the blast loads. In the case of SFRC specimens, the resulting flying rubble seen in Figure 5-46(a) and (c) was much smaller due to steel fibres restraining the release of concrete fragments. Further improvements were observed with the introduction of UHPFRC and high strength steel where the only noticeable secondary fragmentation was similar to powder as portrayed in Figure 5-46(b) and (d).

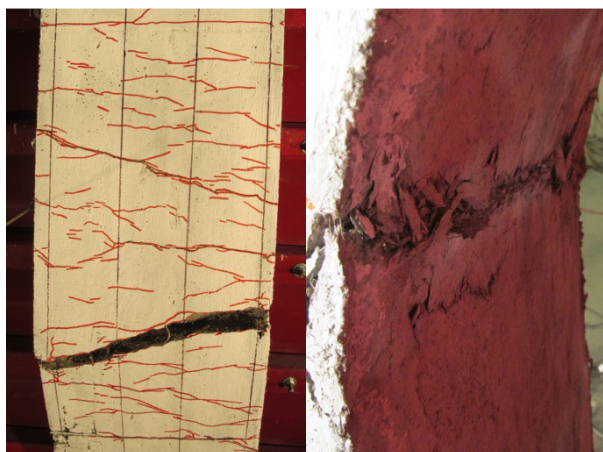
It should be noted that the slabs were subjected to multiple repeated blast tests prior to applying the failure blast load and that accumulated damage may have affected the final behaviour of the specimens. Further studies would be recommended to compare the behaviour of slabs under a single blast loads and repeated testing.



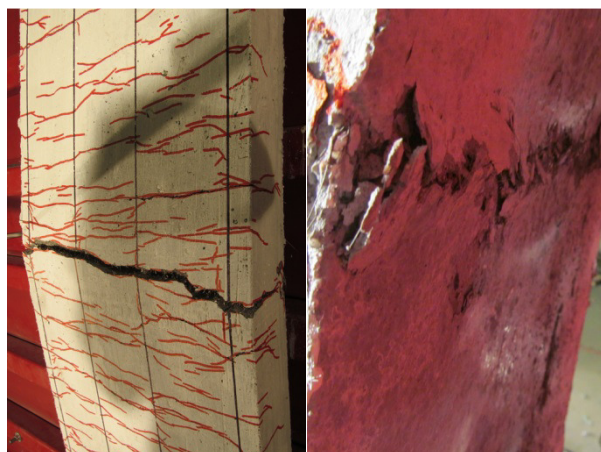
a) SCC-0%-#3NS-100 after Blast 5



b) SCC-1.2%ZP-#3NS-100 after Blast 5



c) CRC-2%A-#3NS-100 after Blast 5



c) CRC-2%A-#3SS-100 after Blast 6

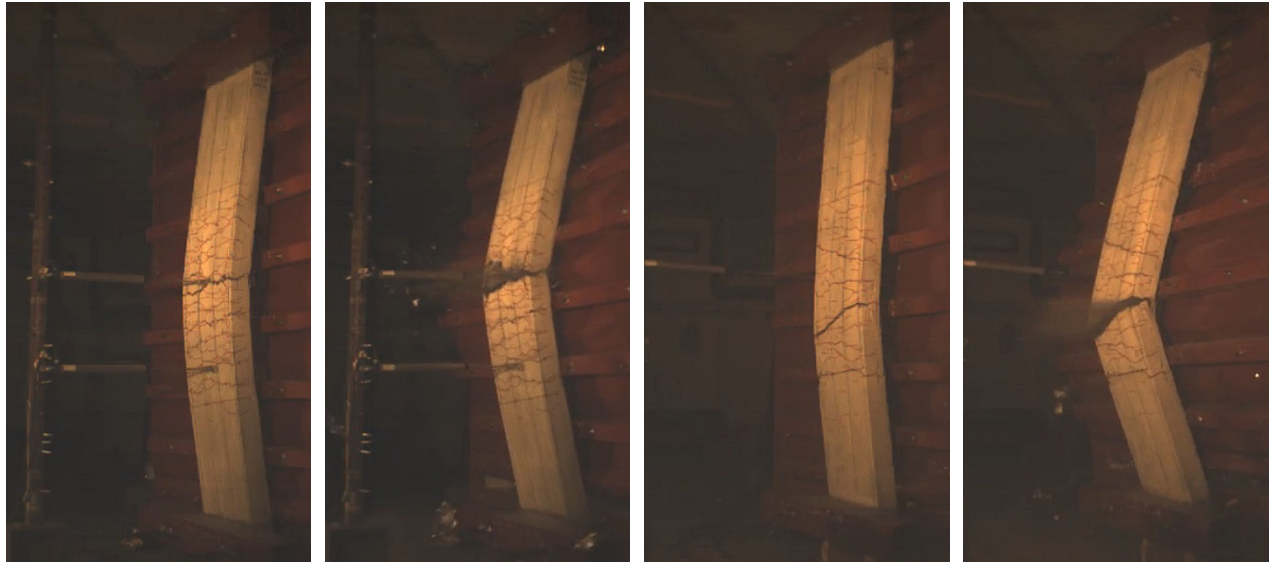
Figure 5-43 Comparison of damage tolerance in SCC, SFRC and UHPFRC slabs



Figure 5-44 SCC-0%-#3NS-100 secondary fragmentation



Figure 5-45 SCC-0%-#3HS-100 secondary fragmentation



a) SCC-1.2%ZP-#3NS-100

b) CRC-2%A-#3NS-100



c) SCC-1.2%ZP-#3HS-100

d) CRC-2%A-#3HS-100

Figure 5-46 Mitigation of secondary fragmentation

CHAPTER 6. ANALYSIS & MODELS

6.1. CHAPTER OVERVIEW

The purpose of this chapter is to predict the response of the test slabs using dynamic inelastic single-degree-of-freedom (SDOF) analysis. This is achieved by implementing various material and strain-rate models for concrete and steel reinforcement bars to obtain resistance functions for the slabs. The resistance functions are then used in a dynamic analysis in order to compare analytical and experimental responses. Several static stress-strain models for plain concrete, SFRC, UHPFRC and steel reinforcement are presented in section 6.2; the variety of models will be used in different combinations to conduct a dynamic sensitivity analysis using the lumped inelasticity method described in section 6.4. To account for the effects of high strain rates on static material models, section 6.3 describes the dynamic increase factors which can be applied to stress-strain relationships in order to increase the strength of materials.

The results from the dynamic analysis are compiled in section 6.5 of this chapter. Discussions regarding the comparison of analytical responses against experimental measurements are divided within that section in function of the concrete type. It should be noted that the sensitivity analysis described in section 6.6 was applied to select the appropriate parameters for the dynamic analysis.

6.2. PARAMETER MODELS

The following section covers the various models used to define the concrete and steel stress-strain relationships which are used to conduct the dynamic analysis. In certain cases, multiple models are listed to allow for a sensitivity analysis to be completed.

6.2.1. PLAIN CONCRETE MODELS

Two models were selected in this analysis to describe unconfined plain concrete under static loading. The first model was proposed by Kent and Park (1971) and modifies Hognestad's (1951) parabola for the post-peak stress-strain relationship. The second model was suggested by Cusson and Paultre (1995) and is associated with equations developed by Popovics (1973) and Fafitis and Shah (1985). The tensile stress-strain curve for plain concrete was represented by a linear relationship with a slope equal to the modulus of elasticity until it reaches the tensile strength.

Kent & Park (1971)

The ascending branch is defined by Hognestad's (1951) second degree parabola as seen in the first part of equation (6-1) where f'_{cu} represents the concrete's unconfined compressive strength, ϵ_{cu} is the concrete strain and ϵ'_{cu} is the concrete strain when it reaches the peak stress. It should be noted that f'_{cu} is calculated as 85% of the compressive strength obtained from cylinder tests due to size factors (Saatcioglu et al. 2011), and that for this particular model, ϵ'_{cu} is replaced by 0.002 mm/mm. The descending branch of equation (6-1) was proposed as a linear function

relative to the concrete's compressive strength. Additional equations (6-2) and (6-3) define the coefficients required to complete the curve.

$$f_{cu}(\varepsilon_{cu}) := \begin{cases} f'_{cu} \cdot \left[\left(2 \cdot \frac{\varepsilon_{cu}}{\varepsilon'_{cu}} \right) - \left(\frac{\varepsilon_{cu}}{\varepsilon'_{cu}} \right)^2 \right] & \text{for } \varepsilon_{cu} \leq \varepsilon'_{cu} \\ f'_{cu} \cdot [1 - Z(\varepsilon_{cu} - \varepsilon'_{cu})] & \text{for } \varepsilon'_{cu} < \varepsilon_c \leq \varepsilon_{50cu} \end{cases} \quad (6-1)$$

$$Z = \frac{0.5}{\varepsilon_{50cu} - \varepsilon'_{cu}} \quad (6-2)$$

$$\varepsilon_{50cu} = \frac{3 + \varepsilon'_{cu} \cdot 145f'_{cu}}{145f'_{cu} - 1000} \quad \text{where } f'_{cu} \text{ is in MPa} \quad (6-3)$$

Cusson & Paultre (1995)

The model proposed by Cusson and Paultre (1995) is a combination of Popovics (1973) ascending branch and a modified Fafitis and Shah (1985) stress-strain relationship for the descending branch. Equation (6-4) covers both branches of the Cusson and Paultre (1995) concrete model. Coefficients k_u and k_{1u} can be obtained from equations (6-5) and (6-6), while coefficient k_{2u} is taken as 1.5 for unconfined concrete. The Young's modulus of elasticity is calculated from a formula by Carrasquillo et al. (1981) which is included below as equation (6-7). Finally, the peak strain ε'_{cu} can be determined using equation (6-8) while ε_{50cu} is assumed as 0.004 mm/mm. A comparative sample of both models was included in Figure 6-1.

$$f_{cu}(\varepsilon_{cu}) := \begin{cases} f'_{cu} \left[\frac{k_u(\varepsilon_{cu}/\varepsilon'_{cu})}{k_u - 1 + (\varepsilon_{cu}/\varepsilon'_{cu})^{k_u}} \right] & \text{for } \varepsilon_c \leq \varepsilon_{cu} \\ f'_{cu} \cdot \exp[k_{1u}(\varepsilon_{cu} - \varepsilon'_{cu})^{k_{2u}}] & \text{for } \varepsilon_c > \varepsilon_{cu} \end{cases} \quad (6-4)$$

$$k_u = \frac{E_c}{E_c - (f'_{cu}/\varepsilon'_{cu})} \quad (6-5)$$

$$k_{1u} = \frac{\ln(0.5)}{(\varepsilon_{50cu} - \varepsilon'_{cu})^{k_{2u}}} \quad (6-6)$$

$$E_c = 3320\sqrt{f'_{cu}} + 6900 \quad (6-7)$$

$$\varepsilon'_{cu} = 0.001684 + 0.000016f'_{cu} \quad (6-8)$$

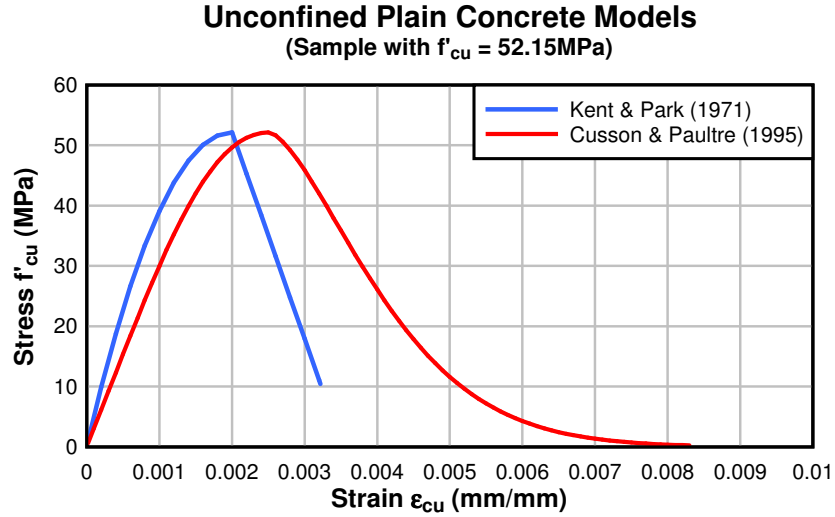


Figure 6-1 Unconfined plain concrete stress-strain models sample

6.2.2. STEEL FIBRE REINFORCED CONCRETE MODELS

The stress-strain curve for steel fibre reinforced concrete can be described by a combination of models from those listed in this section. Several models currently exist in literature for SFRC in compression; however, their accuracy is often limited to the data from their respective research (Bencardino et al. 2008). The model proposed by Aoude (2008), which modifies equations initially developed by Mansur et al. (1999) for high strength fibre reinforced concrete, was selected due to the nature of the materials tested in this research. The second model proposed by Ou et al. (2012), based on stress-strain equations by Carreira and Chu (1985), combined data from multiple researches with decent precision. Tensile model were obtained from research by Lok and Pei (1998), and from guidelines by Bekaert, the manufacturer of Dramix steel fibres (Nemegeer 1998).

Aoude (2008)

The model proposed by Aoude (2008) is the Mansur et al. (1999) stress-strain curve equations modified in order to represent normal strength concrete. This model is defined in equation (6-9). The ascending branch is similar to Cusson & Paultre's (1995) equation for plain concrete with a k_f factor defined by equation (6-10). The descending branch is composed of two coefficients k_{f1} and k_{f2} which vary in function of the strain and are calculated using equations (6-11) and (6-12). Constants used to calculate these coefficients are listed in Table 6-1. The variables v_f , L_f and d_f are respectively the fibre content, the fibres length and the fibres diameter, while the β_1 factor is identical to the previously mentioned k_f coefficient.

$$f_{cuf}(\varepsilon_{cuf}) := \begin{cases} f'_{cuf} \left[\frac{k_f(\varepsilon_{cuf}/\varepsilon'_{cuf})}{k_f - 1 + (\varepsilon_{cuf}/\varepsilon'_{cuf})^{k_f}} \right] & \text{for } \varepsilon_{cuf} \leq \varepsilon'_{cuf} \\ f'_{cuf} \left[\frac{k_{f1}\beta_1(\varepsilon_{cuf}/\varepsilon'_{cuf})}{k_{f1}\beta_1 - 1 + (\varepsilon_{cuf}/\varepsilon'_{cuf})^{k_{f2}\beta_1}} \right] & \text{for } \varepsilon_{cuf} > \varepsilon'_{cuf} \end{cases} \quad (6-9)$$

$$k_f = \frac{E_c}{E_c - (f'_{cuf}/\varepsilon'_{cuf})} = \beta_1 \quad (6-10)$$

$$k_{f1} = \left(\frac{C_a}{f'_{cuf}} \right)^{e_a} \left(1 + X_a \left(\frac{v_f L_f}{d_f} \right)^{e_{2a}} \right) \quad (6-11)$$

$$k_{f2} = \left(\frac{C_a}{f'_{cuf}} \right)^{e_b} \left(1 + X_b \left(\frac{v_f L_f}{d_f} \right)^{e_{2b}} \right) \quad (6-12)$$

Table 6-1 Constants for equations (6-11) and (6-12)

Constants	For $\varepsilon_{cuf} \leq 0.004$	For $\varepsilon_{cuf} > 0.004$
C_a	30.2	4.62
e_a	0.49	0.47
X_a	-0.22	-7.8×10^{-5}
e_{2a}	24.9	-6.4
e_b	0.52	0.51
X_b	0.11	0.83
e_{2b}	-1.09	-0.34

Ou et al. (2012)

The equations proposed by Ou et al. (2012) are based on the Carreira and Chu (1985) stress-strain curve defined as equation (6-13). The compressive strength, the strain at the peak stress and the beta coefficient are given as equations (6-14) through (6-16). The reinforcing index RI_v is calculated from the fibre content, fibre length and fibre diameter using equation (6-17). A sample of the compressive stress-strain relationship is shown in Figure 6-2(a).

$$f_{cuf}(\varepsilon_{cuf}) := f'_{cuf} \left[\frac{\beta \left(\frac{\varepsilon_{cuf}}{\varepsilon'_{cuf}} \right)}{\beta - 1 + \left(\frac{\varepsilon_{cuf}}{\varepsilon'_{cuf}} \right)^\beta} \right] \quad (6-13)$$

$$f'_{cuf} = f'_{cu} + 2.35(RI_v) \quad (6-14)$$

$$\varepsilon'_{cuf} = \varepsilon'_{cu} + 0.0007(RI_v) \quad (6-15)$$

$$\beta = 0.71(RI_v)^2 - 2.00(RI_v) + 3.05 \quad (6-16)$$

$$RI_v = v_f(l_f/d_f) \quad (6-17)$$

Lok & Pei (1998)

The simple, three-part model presented by Lok & Pei (1998) was selected as a model for the tensile stress-strain curve of SFRC. Prior to cracking, the curve follows a linear equation with a slope equal to the modulus of elasticity of the concrete. This remains valid until it reaches the cracking stress f_{ctf} which is calculated through equation (6-18). The post-cracking portion follows a bilinear relationship. At first, a linear equation connects the point of cracking to a stress and strain value defined by equations (6-19) and (6-20) as f_2^* and ε_2^* . The matrix bond strength τ_{bond} is described in equation (6-21) while E_{fp} is taken as 200 GPa. Once again, the variables v_f , L_f and d_f are respectively the fibre content, the fibre length and the fibre diameter. The second portion of the post-cracking curve is a linear equation descending from the previously mentioned f_2^* stress to a failure tensile stress of 0 MPa. A failure tensile strain of 0.02 mm/mm was suggested by Lok and Pei (1998).

$$f_{ctf} = 0.3(f'_{cu})^{2/3} \quad (6-18)$$

$$f_2^* = \frac{1}{2} v_f \tau_{bond} \frac{l_f}{d_f} \quad (6-19)$$

$$\varepsilon_2^* = \tau_{bond} \frac{l_f}{d_f} \frac{1}{E_{fp}} \quad (6-20)$$

$$\tau_{bond} = 0.6(f'_{cu})^{2/3} \quad (6-21)$$

Bekaert - Dramix guidelines

Bekaert developed a simplified three-part model in their design guidelines for Dramix fibres (Nemegeer 1998). According to these guidelines, the ascending branch follows a slope equal to the modulus of elasticity until it reaches the cracking stress f_{ctf} which can be estimated with equation (6-18). The two part descending branch is based on the values of $f_{fct,eq,150}$ and $f_{fct,eq,300}$ which are the equivalent flexural tensile stresses calculated using equations (6-22) and (6-23). Variables $F_{m,150}$ and $F_{m,300}$ represent the mean loads required to produce a deflection of $l/150$ and $l/300$ in a deformation-controlled bending test on a SFRC prism. The values of l , b and h are the dimensions of the prismatic fibre reinforced concrete beam samples.

$$f_{fct,eq,150} = \frac{F_{m,150} \cdot l}{bh^2} \quad (6-22)$$

$$f_{fct,eq,300} = \frac{F_{m,300} \cdot l}{bh^2} \quad (6-23)$$

Hence, for the descending branch, a linear slope connects the point of cracking to a point defined by a stress equal to $0.37f_{fct,eq,300}$ at a strain of 1%. Afterwards, from the previous point, it follows a linear slope until it reaches a stress of $0.37f_{fct,eq,150}$ at a strain of 10%. A sample of the tensile stress-strain relationship is shown in Figure 6-2(b).

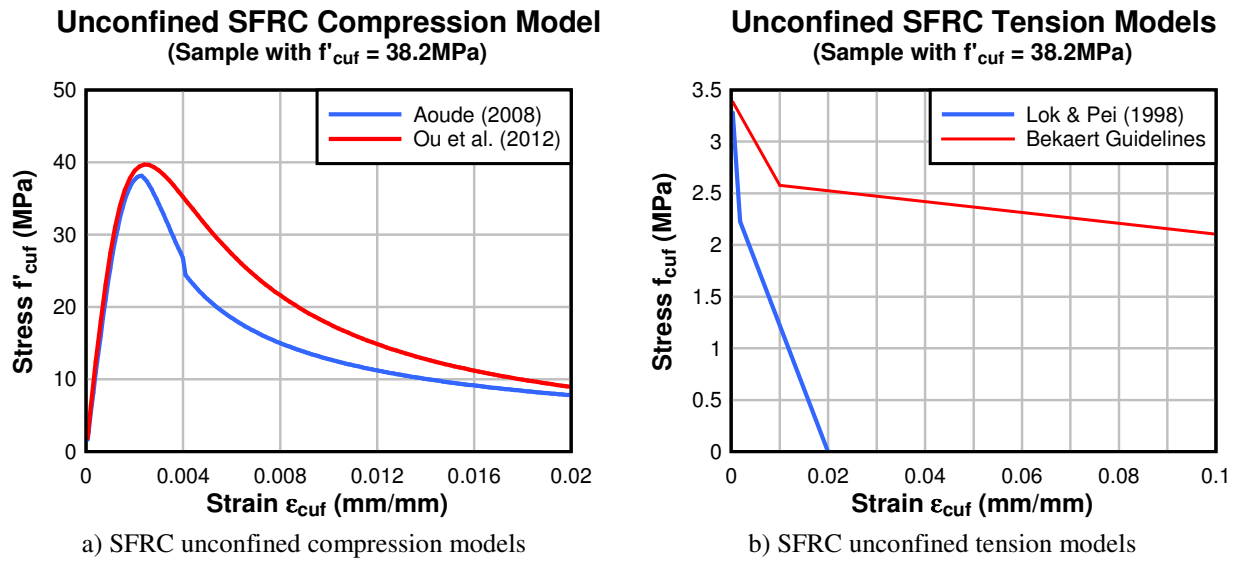


Figure 6-2 Unconfined SFRC stress-strain models sample

6.2.3. ULTRA-HIGH PERFORMANCE FIBRE REINFORCED CONCRETE MODELS

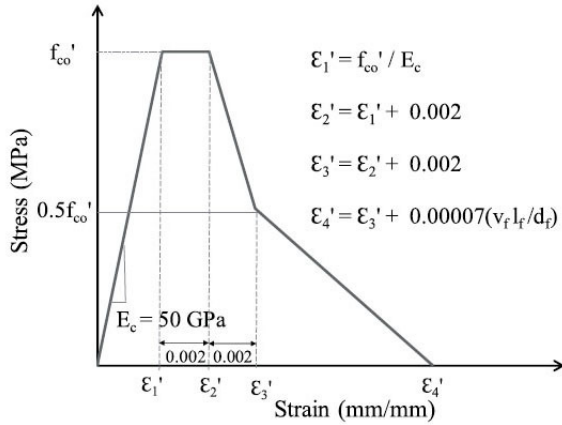
The stress-strain curve for unconfined UHPFRC is composed of two models. The simplified compression model proposed by Hosinieh et al. (2015) was selected along with the Lok and Pei (1998) tension model which was previously described for SFRC. Both models can be seen in Figure 6-3 with sample curves in Figure 6-4.

Hosinieh et al. (2015)

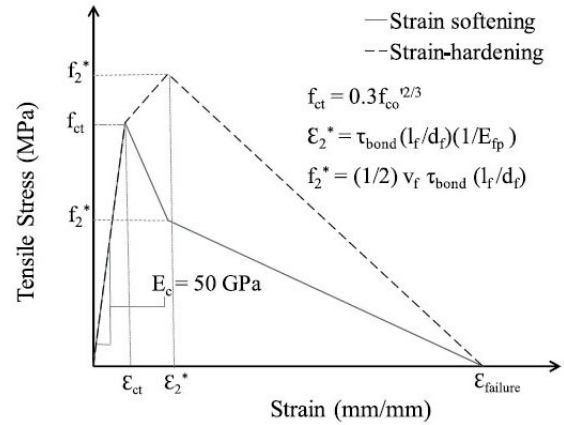
The model proposed by Hosinieh et al. (2015) is a simple four-part curve for which important strain formulas are compiled in equation (6-24). The ascending branch follows a linear slope of $E_c = 50 \text{ GPa}$ until it reaches the unconfined peak stress value f'_{cuf} at strain ϵ'_1 . Then, the stress-strain curve forms a plateau, remaining at the peak stress for a strain of 0.002 mm/mm where it will reach ϵ'_2 . Afterwards, the bi-linear descending branch first connects the previous point to a stress value equivalent to 50% of the peak stress at strain ϵ'_3 . Finally, a linear equation reduces the concrete stress to 0 MPa at strain ϵ'_4 . This final strain value is dependant of the properties of

the steel fibres incorporated in the concrete mix, including the fibre content, the fibres length and the fibres diameter.

$$\text{Strain values} \begin{cases} \varepsilon_1' = f_{cu}' / E_c \\ \varepsilon_2' = \varepsilon_1' + 0.002 \\ \varepsilon_3' = \varepsilon_2' + 0.002 \\ \varepsilon_4' = \varepsilon_3' + 0.00007 \left(\frac{v_f l_f}{d_f} \right) \end{cases} \quad (6-24)$$



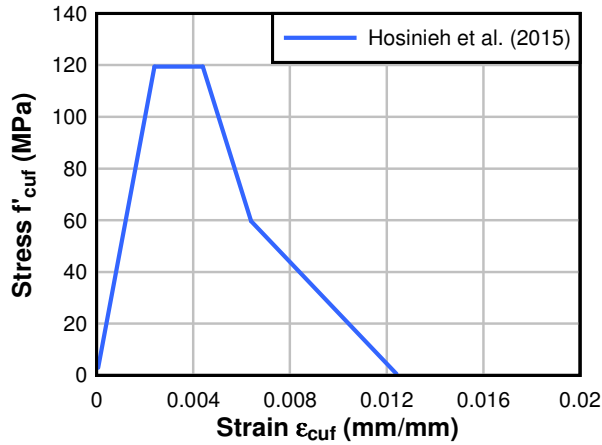
a) UHPFRC unconfined compression model



b) UHPFRC unconfined tension model

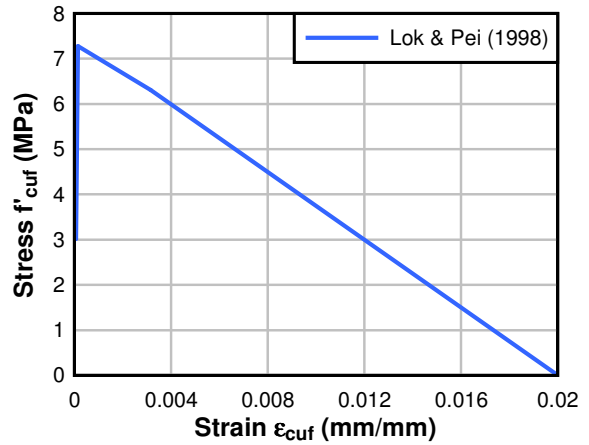
Figure 6-3 Unconfined UHPFRC stress-strain models (Hosinie et al. 2015)

Unconfined SFRC Compression Model
(Sample with $f'_{cuf} = 120 \text{ MPa}$)



a) SFRC unconfined compression model

Unconfined SFRC Tension Model
(Sample with $f'_{cuf} = 120 \text{ MPa}$)



b) SFRC unconfined tension model

Figure 6-4 Unconfined SFRC stress-strain models sample

6.2.4. REGULAR GRADE DEFORMED STEEL REINFORCEMENT BAR MODELS

In the case of regular grade deformed steel reinforcement bars, two tensile stress-strain curve models were taken into consideration for the dynamic analysis. The first is based on the experimental data obtained from steel coupon tests. The second is a model integrated within the

RCBlast (2014) blast load simulation software that was described by Jacques et al. (2012) as equations (6-25) through (6-28). Variable f_s is the static steel stress. Variables f_y , f_{sh} and f_u are defined as the static yield, strain hardening and ultimate stresses. Finally, variables ε_y , ε_{sh} and ε_u represent the steel strains at yield, strain hardening and ultimate.

$$f_s = E_s \varepsilon_s \quad \text{for } \varepsilon_s \leq \varepsilon_y \quad (6-25)$$

$$\varepsilon_y = f_y / E_s \quad (6-26)$$

$$f_s = f_y + (\varepsilon_s - \varepsilon_y) \left(\frac{f_{sh} - f_y}{\varepsilon_{sh} - \varepsilon_y} \right) \quad \text{for } \varepsilon_y < \varepsilon_s \leq \varepsilon_{sh} \quad (6-27)$$

$$f_s = f_y + (f_u - f_y) \left[2 \left(\frac{\varepsilon_s - \varepsilon_{sh}}{\varepsilon_u - \varepsilon_{sh}} \right) - \left(\frac{\varepsilon_s - \varepsilon_{sh}}{\varepsilon_u - \varepsilon_{sh}} \right)^2 \right] \quad \text{for } \varepsilon_{sh} < \varepsilon_s \leq \varepsilon_u \quad (6-28)$$

The data inputs in the model were coherent with the values obtained from the experimental curves. Both models with their respective dynamic increase factors are included in Figure 6-5 below; the experimental and RCBlast (Jacques 2014) models slightly differ during the early strain hardening portion of the stress-strain curves. It should also be noted that the dynamic increase factors depreciates gradually from 1.30 to 1.10 between the yield plateau and the ultimate stress; further information on these dynamic increase factors has been included later on in this chapter.

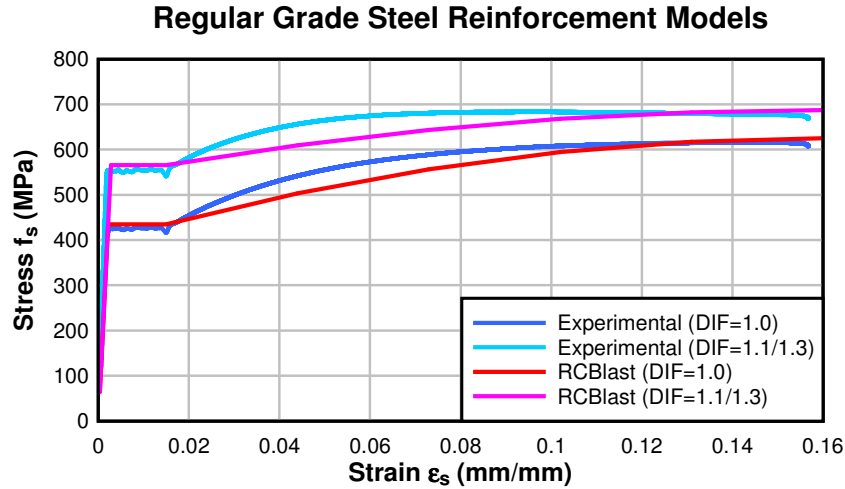


Figure 6-5 Regular grade steel reinforcement stress-strain models

In regards to compression models, it is assumed that the steel reinforcement bars will not buckle as it did not happen during the experiment. To be conservative, the static compression stress-strain curve for all steel models is assumed to be a bilinear relationship defined by an ascending

branch with a slope equal to the steel modulus of elasticity, followed by a plateau equal to the yielding stress.

6.2.5. REGULAR GRADE NON-DEFORMED STEEL REINFORCEMENT BAR MODELS

For the non-deformed 6.3 mm steel reinforcement bars which were used in smaller slab specimens, no specific analytical model was taken into consideration other than the experimental data obtained from steel coupon tests. The tensile stress-strain curves used for the dynamic analysis are included in Figure 6-6 along with their dynamic increase factors. Note that due to the lack of a proper yield plateau, the curve was modified with a single dynamic increase factor equal to 1.10. Further information on the dynamic increase factor has been included later on in this chapter.

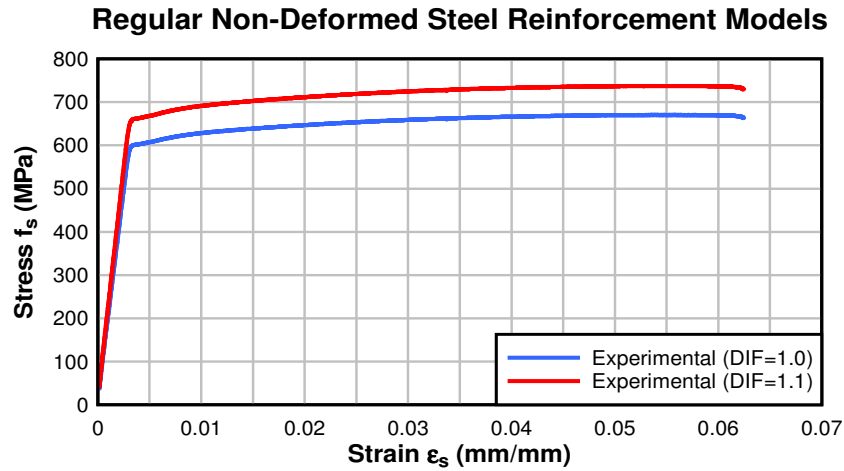


Figure 6-6 Regular non-deformed steel reinforcement models

6.2.6. HIGH STRENGTH STEEL (MMFX₂) MODELS

In the case of high strength steel reinforcement bars, two tensile stress-strain curve models were taken into consideration for the dynamic analysis. The first is based on the experimental data obtained from steel coupon tests. The second model is a set of three equations developed by the ACI Task Group ITG-6 (2010) for ASTM A1035 steel. The set of equations is included below as equation (6-29). Both models with their respective dynamic increase factors are included in Figure 6-7 below. Note that due to the lack of a proper yield plateau, the curve was modified with a single dynamic increase factor equal to 1.10 (in addition, analysis was also conducted neglecting dynamic increase factor for this steel type, with a DIF equal to 1.00). Further information on the dynamic increase factor has been included later on in this chapter.

$$f_s(\epsilon_s) := \begin{cases} 200,000\epsilon_s(MPa) & \text{for } \epsilon_s \leq 0.0024 \\ 1170 - \frac{2.96}{\epsilon_s + 0.0019}(MPa) & \text{for } 0.0024 < \epsilon_s \leq 0.02 \\ 1040(MPa) & \text{for } 0.02 < \epsilon_s \leq 0.06 \end{cases} \quad (6-29)$$

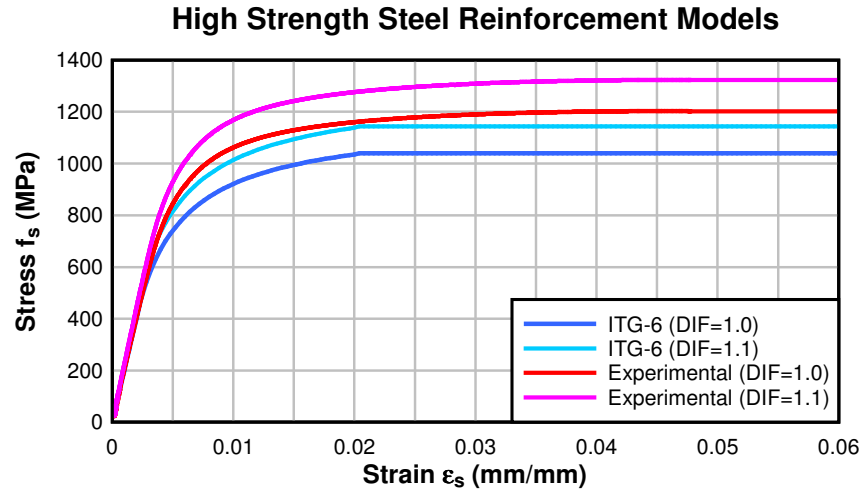


Figure 6-7 High strength steel reinforcement models

6.2.7. STAINLESS STEEL MODEL

Due to the current lack of specific model for high ductility stainless steel rebar, the experimental data from coupons will be used to conduct the dynamic analysis of specimen CRC-2%A-#3SS-100. It should be noted that the experimental curve could be fit using a modified Ramberg-Osgood function however it wasn't considered beneficial for this particular study. The tensile stress-strain relationship used for this analysis is included in Figure 6-8. Note that due to the lack of a proper yield plateau, the curve was modified with a single dynamic increase factor equal to 1.10 (in addition, analysis was also conducted neglecting dynamic increase factor for this steel type, with a DIF equal to 1.00). Further information on the dynamic increase factor has been included later on in this chapter.

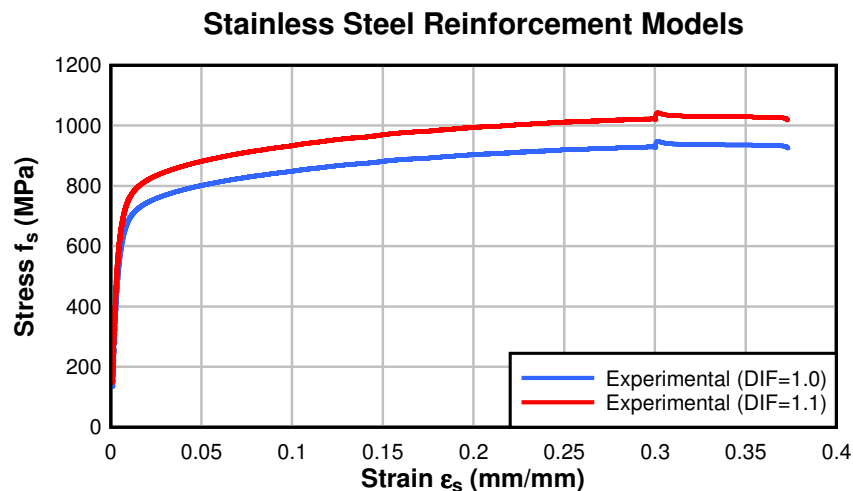


Figure 6-8 Stainless steel reinforcement models

6.3. DYNAMIC INCREASE FACTORS

This section covers the various dynamic increase factors (DIF) that were used to modify the static stress-strain curves obtained from the previously described models. It should be noted that a lower bound strain rate $\dot{\epsilon} = 1 \text{ s}^{-1}$ was assumed for the specimens tested through the shock-tube (Saatcioglu et al. 2011).

6.3.1. DYNAMIC INCREASE FACTORS FOR CONCRETE MODELS

Research has shown that plain concrete, SFRC and UHPFRC material properties are influenced by the strain rate which increases strength under dynamic compressive, tensile and flexural loads. The dynamic increase factors used to modify the static concrete models vary in function of the strain rate and the unconfined compressive concrete strength. Three sets of equations were selected to describe the factors: CEB Model Code 90 (1990) for plain concrete in compression, Zhang and Mindness (2011) for SFRC and UHPFRC in compression, and Malvar and Ross (1998) for plain concrete, SFRC and UHPFRC in tension. In addition, the UFC-3-340-02 (2008) proposes design dynamic increase factors which are not dependent of the strain rate.

CEB Model Code 90 (1990)

Equations (6-30) and (6-31) from the Euro-international Concrete Committee were selected to define the dynamic increase factor DIF_c for plain concrete in compression. The variable $\dot{\epsilon}$ represents the strain rate while $\dot{\epsilon}_s$ is a constant for the static strain rate.

$$DIF_c = \begin{cases} \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^{1.026\alpha} & \text{for } \dot{\epsilon} \leq 30\text{s}^{-1} \\ \gamma \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^{1/3} & \text{for } \dot{\epsilon} > 30\text{s}^{-1} \end{cases} \quad (6-30)$$

$$\log \gamma = 6.156\alpha - 2, \quad \alpha = 1/(5 + 9f'_{cu}/10), \quad \dot{\epsilon}_s = 30 \cdot 10^{-6}\text{s}^{-1} \quad (6-31)$$

Zhang & Mindness (2011)

Equations (6-32) and (6-33) were proposed by Zhang and Mindness (2011) to estimate the dynamic increase factor DIF_c for SFRC and UHPFRC in compression. The variable $\dot{\epsilon}_{BLT}$ represents the bi-linear equation transition point.

$$DIF_c = \begin{cases} (\phi\dot{\epsilon})^\alpha & \text{for } \dot{\epsilon} < \dot{\epsilon}_{BLT} \\ \beta(\dot{\epsilon})^{1/3} & \text{for } \dot{\epsilon} \geq \dot{\epsilon}_{BLT} \end{cases} \quad (6-32)$$

$$\alpha = \frac{\ln(\beta \cdot \dot{\epsilon}_{BLT}^{1/3})}{\ln(\phi \cdot \dot{\epsilon}_{BLT})}, \quad \beta = \frac{5}{9} \cdot \exp\left(\frac{-f'_{cuf}}{230}\right), \quad \phi = 10^5, \quad \dot{\epsilon}_{BLT} = 25 \cdot \exp\left(\frac{f'_{cuf}}{130}\right) \quad (6-33)$$

Malvar & Ross (1998)

Due to the limited number of studies conducted on the effects of high strain rates on UHPFRC in dynamic tension there is currently no well-established model to estimate the tensile dynamic increase factor of SFRC or UHPFRC. Hence, the equations (6-34) and (6-35) proposed by Malvar and Ross (1998) applied normally to estimate the dynamic increase factor DIF_t for regular concrete in tension were selected to also account for the effects of high strain rates of SFRC and UHPFRC under dynamic tension. It should be noted that the equations for Malvar and Ross (1998) tend to produce lower-bound, conservative dynamic increase factors for UHPFRC (Millard et al. 2010).

$$DIF_t = \begin{cases} \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^\delta & \text{for } \dot{\epsilon} \leq 1s^{-1} \\ \beta^* \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^{1/3} & \text{for } \dot{\epsilon} > 1s^{-1} \end{cases} \quad (6-34)$$

$$\delta = \frac{1}{\left(5 + \frac{8 \cdot f'_{cuf}}{f'_{cr}}\right)}, \quad \log \beta^* = 6\delta - 2, \quad f'_{cr} = 10\text{MPa}, \quad \dot{\epsilon}_s = 10^{-6}s^{-1} \quad (6-35)$$

UFC-3-340-02 (2008)

The dynamic increase factors proposed by UFC-3-340-02 (2008) are acceptable for the design of blast-resistant reinforced concrete elements due to their conservative nature. The DIF values do not rely on the strain rate and are applied on the totality of the concrete or steel stress-strain curves. In the case of this study, the DIF for bending at far range would be applied. These values can be found in Table 6-2 below.

Table 6-2 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

6.3.2. DYNAMIC INCREASE FACTORS FOR STEEL MODELS

The dynamic increase factors used to modify the static steel models vary in function of the type of steel and the strain rate. The expressions developed by Saatcioglu et al. (2011) were selected to define these factors.

Saatcioglu et al. (2011)

Equation (6-36) was proposed by Saatcioglu et al. (2011) to define the dynamic increase factors of steel at yield and at ultimate.

$$\begin{aligned} \text{at yield} &\rightarrow DIF_y = 0.034\ln(\dot{\epsilon}) + 1.30 \geq 1.0 \\ \text{at ultimate} &\rightarrow DIF_u = 0.0101\ln(\dot{\epsilon}) + 1.10 \geq 1.0 \end{aligned} \quad (6-36)$$

It was previously mentioned that a strain rate of $\dot{\epsilon} = 1 \text{ s}^{-1}$ would be assumed for slabs tested with the shock-tube; by replacing this value into the equation, the dynamic increase factors of reinforcing bars are obtained as $DIF_y = 1.30$ and $DIF_u = 1.10$. For regular steel reinforcement, the dynamic increase factor would gradually depreciate between the strain of the yield plateau and the ultimate strain. In the case of steel reinforcement which lacked a distinctive yield plateau, or which was of a higher grade, it was assumed that a dynamic increase factor of 1.10 would be applied to the entire stress-strain curve.

6.4. DYNAMIC ANALYSIS USING LUMPED INELASTICITY APPROACH

The dynamic analysis was conducted by solving the single-degree-of-freedom (SDOF) equation of motion given as equation (6-37) below:

$$K_{LM}(u(t))m\ddot{u}(t) + R(u(t)) = AP_r(t) \quad (6-37)$$

where $u(t)$ is the deflection at mid-height of the slab specimen, $\ddot{u}(t)$ is the acceleration at mid-height, $K_{LM}(u(t))$ is the load mass transformation factor in function of the deflection, $R(u(t))$ is the resistance of the member in function of the deflection, m is the total mass of the system, A is the area impacted by the blast pressure and $P_r(t)$ is the reflected pressure in function of time. The total mass of the system includes both the load transfer device and the specimen; it is estimated at 398 kg for 80 mm slabs and 445 kg for 100 mm slabs. The frame opening of the shock-tube serves as the area and is measured as 4.13 m^2 .

The lumped inelasticity approach used to obtain resistance functions for this dynamic analysis has been described in detail by Jacques et al. (2012) and integrated in the software RCblast (Jacques 2014). To conduct this analysis, a few inputs are required including the specimen cross-sectional dimensions, the arrangement of steel reinforcement, and the definition of material stress-strain curves at high strain rates. Afterwards, the moment-curvature relationships are generated from these characteristics. To determine the resistance functions and load mass transformation factors the slab is idealized as half-span symmetric linear elastic flexural member

with a non-linear rotational spring at the mid-span critical section such as demonstrated in Figure 6-9. The flexural rigidity EI is determined from the pre-yield slope of the moment-curvature relationship, while a post-yield sectional analysis defines the moment-rotation relationship of the rotational spring. For the purpose of this analysis, the plastic hinge length was taken as the distance between the tension steel and the extreme compression fibre. It is also assumed that the spring stiffness is very large prior to reaching the yielding point and that it is only allowed to rotate past yielding.

The force-deflection equation for the flexural member can be derived by solving the Euler-Bernoulli bending equation with the appropriate loading and support conditions. At the bottom support, it is assumed the displacement and moments are equal to zero. At mid-span, located at a distance of L_o from the bottom support, the boundary conditions are in function of properties of the rotational spring such as the spring's rotation θ_s and the moment $k_s\theta_s$. The resistance functions and equivalent SDOF transformation factors are obtained by solving this force-equation at various load stages.

To calculate the mid-span displacement for a particular blast, a simplified triangular pressure-time history is defined based on the peak reflected pressure and impulse recorded during the experiment. The average acceleration numerical method is then applied to solve the equation of motion and obtain the maximum displacement.

The dynamic analysis was conducted using the software RCblast (Jacques 2014). This software was proven to be accurate when used with proper material models (Jacques et al. 2012). Results are summarized and discussed in the next sub-section of this thesis.

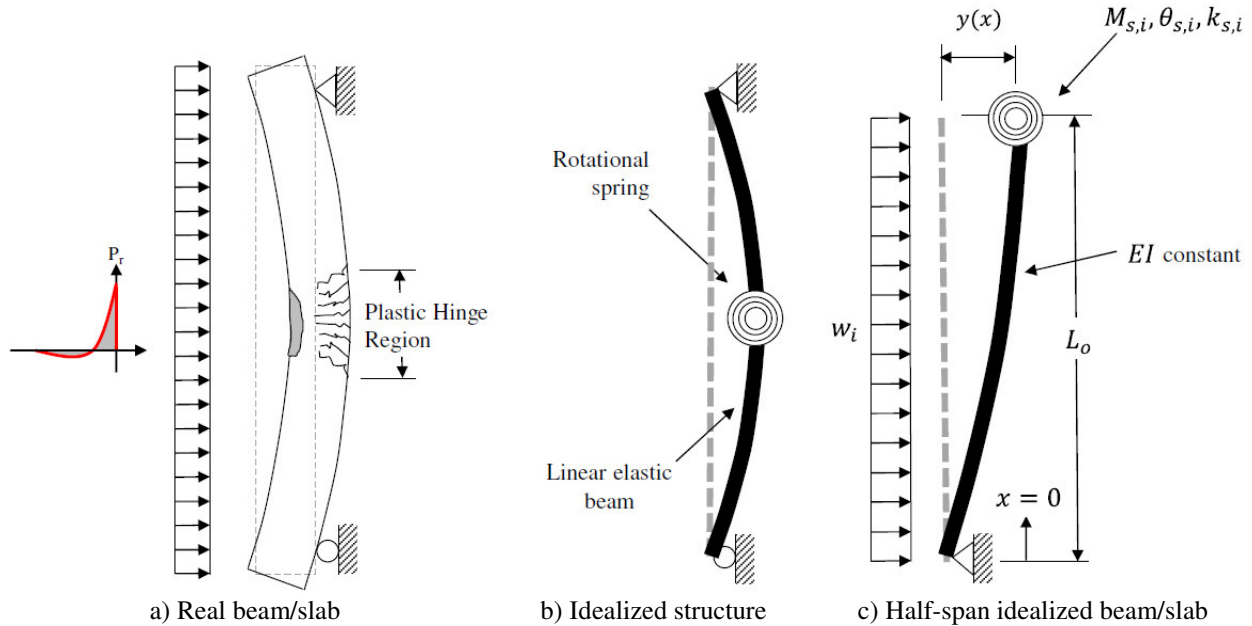


Figure 6-9 Reinforced concrete slab; lumped inelasticity approach (Jacques et al. 2012)

6.5. DYNAMIC ANALYSIS RESULTS

The results from the SDOF dynamic analysis were compiled in several tables in function of the specimen's concrete type. These tables can be found in section 6.5.4 of this document.

Summary tables include the specimen names, the blast test number and associated reflected pressure, reflected impulse and idealized positive phase duration. They also contain experimental and analytical displacements, analytical over experimental displacement ratios, and the error percentage.

Statistics tables include blast test ranges, mean analytical over experimental displacement ratios, standard deviations, coefficients of variance and mean error percentages. The dynamic analysis will be considered to be accurate if the average of the ratios is close to 1.00, while the precision of the analysis is defined in function of how small is the coefficient of variance. Graphics demonstrating the spread and variation between the analytical and experimental values are also included in section 6.5.4.

Regarding the analytical displacements, these were obtained from a sensitivity analysis where different combinations of concrete and steel models with their respective dynamic increase factors were compared in terms of accuracy after conducting a dynamic analysis. The most reliable combination of concrete and steel models was then selected for each specimen and summarized in Table 6-3. The analytical responses resulting from these cases were included in this section for discussion; additional information on the sensitivity analysis can be found in section 6.6 of this chapter.

Table 6-3 Selected model combinations; results from the sensitivity analysis

Specimen	Concrete models		Steel models
	Compression	Tension	Tension
SCC-0%-6.3NS-80	Cusson & Paultre (1995)	Linear slope	Experimental
SCC-0%-#3NS-100	Cusson & Paultre (1995)	Linear slope	Jacques et al. (2012)
SCC-0%-#3HS-100	Cusson & Paultre (1995)	Linear slope	ACI Group ITG-6 (2010)
SCC-1.2%ZP-6.3NS-80	Aoude (2008)	Lok & Pei (1998)	Experimental
SCC-0.6%ZP-#3NS-100	Aoude (2008)	Lok & Pei (1998)	Jacques et al. (2012)
SCC-1.2%ZP-#3NS-100	Aoude (2008)	Lok & Pei (1998)	Jacques et al. (2012)
SCC-0.6%BP-#3NS-100	Aoude (2008)	Lok & Pei (1998)	Jacques et al. (2012)
SCC-1.2%ZP-#3HS-100	Aoude (2008)	Lok & Pei (1998)	ACI Group ITG-6 (2010)
CRC-2%B-6.3NS-80	Hosinie et al. (2015)	Lok & Pei (1998)	Experimental
CRC-2%B-#3NS-80	Hosinie et al. (2015)	Lok & Pei (1998)	Jacques et al. (2012)
CRC-2%B-#3NS-100	Hosinie et al. (2015)	Lok & Pei (1998)	Jacques et al. (2012)
CRC-2%A-#3NS-100	Hosinie et al. (2015)	Lok & Pei (1998)	Jacques et al. (2012)
CRC-2%A-#3HS-100	Hosinie et al. (2015)	Lok & Pei (1998)	ACI Group ITG-6 (2010)
CRC-2%A-#3SS-100	Hosinie et al. (2015)	Lok & Pei (1998)	Experimental

6.5.1. DYNAMIC ANALYSIS RESULTS – PLAIN CONCRETE SPECIMENS

Comparative analytical and experimental data for plain concrete specimens are compiled in Table 6-4 with a statistical analysis in Table 6-5. The accuracy of the SDOF model is visually described by Figure 6-10.

For all blast tests conducted on plain concrete specimens, the average displacement ratio ($\delta_{\text{anls}}/\delta_{\text{exp}}$) is 0.76 with a percentage error of 23.6%, a standard deviation of 0.13 and a coefficient of variance of 17.1%. This indicates that the dynamic analysis generally over-estimates the resistance of the plain concrete specimens. Prior to permanent deformations, such as during blast 1 for SCC-0%-6.3NS-80 and SCC-0%-#3NS-100 or during blast 1, 2 and 3 for SCC-0%-#3HS-100, the dynamic analysis was capable of obtaining displacement values with 15.4% and less error margin. The high strength steel specimen was also at a mere 0.3% from the actual experimental value during blast 1. The increased inaccuracy of later shots can be associated with the fact that the plain concrete specimens were weakened quickly by previous blast loads; ideally, such specimens should be tested on a single blast test.

6.5.2. DYNAMIC ANALYSIS RESULTS – SFRC SPECIMENS

Comparative analytical and experimental data for SFRC specimens are compiled in Table 6-6 with the statistical analysis in Table 6-7. The accuracy of the SDOF model is illustrated by Figure 6-11.

For all blast tests conducted on SFRC specimens, the average displacement ratio ($\delta_{\text{anls}}/\delta_{\text{exp}}$) is 0.63 with a percentage error of 36.8%, a standard deviation of 0.14 and a coefficient of variance of 22.0%. This indicates that the dynamic analysis generally over-estimates significantly the resistance of SFRC specimens. The model combinations selected for the dynamic analysis were only capable of estimating with decent accuracy the displacements of specimen SCC-1.2%ZP-#3HS-100 during blasts 1 and 2; the error percentages were 1.48% and 16.53% respectively. The responses of SCC-1.2%ZP-6.3NS-80 and SCC-0.6%ZP-#3NS-100 during blast 1 were also predicted within a 25% margin.

The accuracy shown by the SDOF dynamic analysis for the high strength steel specimen SCC-1.2%ZP-#3HS-100 could be associated with the importance of this steel type on the response of the slab. It should also be noted that the selection of an SFRC stress-strain model is quite complicated as there is no well-established model applicable to all fibre types and content. As previously indicated, the accuracy of SFRC compression models is often limited to the contents of their respective research (Bencardino et al. 2008).

6.5.3. DYNAMIC ANALYSIS RESULTS – UHPFRC SPECIMENS

Comparative analytical and experimental data for UHPFRC specimens are compiled in Table 6-8 with the statistical analysis in Table 6-9 through Table 6-11. The accuracy of the SDOF model is visually represented by Figure 6-12.

For all blast tests conducted on UHPFRC specimens, the average displacement ratio ($\delta_{\text{anls}}/\delta_{\text{exp}}$) is 0.81 with a percentage error of 26.2%, a standard deviation of 0.25 and a coefficient of variance of 31.0%. This indicates that the response is generally under-estimated by the dynamic analysis; however, a few tests resulted in a higher analytical displacement when compared to their experimental counterpart. The coefficient of variance is also quite high meaning the ratios differ greatly from one test to another. It should be noted that specimens with the A-type fibres seem to allow better predictions than B-type, as ratios are 0.84 against 0.75 and error percentages are 24.2% against 29.1%, but this might be due to the nature of the slabs as A-type fibres specimens include the high strength steel and high ductility stainless steel specimens.

In terms of specific blast tests, the dynamic analysis on blast 1 and 2 provided closer results however the spread of data reaches 19% for the coefficient of variance meaning that while the average of the displacement ratio is close to 1.00, the results differ greatly between different specimen configurations. As for later blasts, the slabs are weakened from repeated shots and the responses cannot be predicted accurately with the current models. Furthermore, the lack of distinct plastic hinge formation at failure in some UHPFRC specimens may be incompatible with the assumptions made in the lumped inelasticity approach.

6.5.4. DYNAMIC ANALYSIS RESULTS – SUMMARY OF RESULTS

The tables in this section summarize the dynamic analysis results and the comparisons between analytical and experimental responses. Comprehensive charts are included to visualize the accuracy of the dynamic analysis procedure incorporated to this research.

Table 6-4 Summary of analysis for plain concrete specimens

Specimen	Idealized Shockwave				Maximum Mid-Span Displacement			
	Blast #	P _r kPa	T _d ms	I _r kPa-ms	δ_{exp} mm	δ_{anls} mm	$\delta_{anls}/\delta_{exp}$	% Error %
SCC-0%-6.3NS-80	-1	18.7	13.6150	127.3	43.2	36.80	0.85	14.8%
	-2	32.5	12.9231	210	100.6	67.85	0.67	32.6%
	-3	46.1	11.9306	275	176.4	100.76	0.57	42.9%
SCC-0%-#3NS-100	-1	18.9	14.6032	138	23.5	20.59	0.87	12.5%
	-2	34.6	13.7572	238	52.8	37.74	0.71	28.6%
	-3	45.3	14.0839	319	84.9	56.28	0.66	33.7%
	-4	53.2	13.1955	351	110.2	65.00	0.59	41.0%
	-5	-	-	-	No Data	N/A	N/A	N/A
SCC-0%-#3HS-100	-1	19	13.7895	131	18.9	18.95	1.00	0.3%
	-2	35.7	12.4370	222	36.3	32.33	0.89	10.9%
	-3	46	14.0435	323	56.6	47.86	0.85	15.4%
	-4	57	13.4035	382	76.1	58.04	0.76	23.7%
	-5	63.3	13.9021	440	93.9	68.58	0.73	27.0%
	-6	73.9	13.4777	498	No Data	N/A	N/A	N/A

Table 6-5 Analysis statistics for plain concrete specimens per blast

Blast #	Mean	Standard deviation	Coefficient of variance	% Error
1-6	0.76	0.13	17.1%	23.6%
1	0.91	0.08	8.9%	9.2%
2	0.76	0.12	15.1%	24.0%
3	0.69	0.14	20.1%	30.7%
4	0.68	0.10	14.7%	32.4%
5	0.73	N/A	N/A	27.0%

Table 6-6 Summary of analysis for SFRC specimens

Specimen	Idealized Shockwave				Maximum Mid-Span Displacement			
	Blast #	P _r kPa	T _d ms	I _r kPa-ms	δ_{exp} mm	δ_{anls} mm	$\delta_{anls}/\delta_{exp}$	% Error
SCC-1.2%ZP-6.3NS-80	-1	24.7	12.63	156	47.1	36.98	0.79	21.5%
	-2	36.7	12.59	231	97.6	63.13	0.65	35.3%
	-3	48.8	12.62	308	152.8	104.49	0.68	31.6%
	-4	56.4	12.55	354	180.7	135.69	0.75	24.9%
SCC-0.6%ZP-#3NS-100	-1	20.1	13.73	138	20.7	15.82	0.76	23.6%
	-2	36.2	12.87	233	48.1	27.81	0.58	42.2%
	-3	45.8	13.23	303	76.2	40.45	0.53	46.9%
	-4	52.5	15.43	405	122.1	65.60	0.54	46.3%
	-5	64.9	12.23	397	181.6	65.15	0.36	64.1%
SCC-1.2%ZP-#3NS-100	-1	19.7	13.81	136	23.9	15.59	0.65	34.8%
	-2	34.7	13.72	238	44.2	28.34	0.64	35.8%
	-3	46.4	13.92	323	72.9	44.55	0.61	38.9%
	-4	55.5	13.30	369	105.1	56.47	0.54	46.3%
	-5	62.9	13.29	418	155.9	70.73	0.45	54.6%
SCC-0.6%BP-#3NS-100	-1	No Data	No Data	No Data	22.6	N/A	N/A	N/A
	-2	36.6	14.10	258	52.0	31.60	0.61	39.2%
	-3	46.6	13.65	318	73.1	43.95	0.60	39.9%
	-4	55.8	12.76	356	99.2	53.87	0.54	45.7%
	-5	75	10.61	398	158.6	66.85	0.42	57.9%
SCC-1.2%ZP-#3HS-100	-1	21.2	13.49	143	18.3	18.03	0.99	1.5%
	-2	33.9	14.93	253	38.0	31.72	0.83	16.5%
	-3	48	14.08	338	61.3	43.56	0.71	28.9%
	-4	62.7	13.01	408	80.9	54.78	0.68	32.3%
	-5	68.8	14.19	488	109.3	69.06	0.63	36.8%

Table 6-7 Analysis statistics for SFRC specimens per blast

Blast #	Mean	Standard deviation	Coefficient of variance	% Error
1-5	0.63	0.14	22.0%	36.8%
1	0.80	0.14	17.4%	20.3%
2	0.66	0.10	15.2%	33.8%
3	0.63	0.07	11.4%	37.2%
4	0.61	0.10	16.3%	39.1%
5	0.47	0.12	25.1%	53.4%

Table 6-8 Summary of analysis for UHPFRC specimens

Specimen	Idealized Shockwave				Maximum Mid-Span Displacement			
	Blast #	P _r kPa	T _d ms	I _r kPa-ms	δ_{exp} mm	δ_{anls} mm	$\delta_{anls}/\delta_{exp}$	% Error
CRC-2%B-6.3NS-80	-1	18.4	14.5652	134	20.3	24.92	1.23	22.8%
	-2	32.4	14.2593	231	44.8	45.22	1.01	1.0%
	-3	42.6	13.8498	295	92.3	72.00	0.78	22.0%
	-4	51.3	12.4756	320	111.2	85.26	0.77	23.3%
CRC-2%B-#3NS-80	-1	21.9	12.6027	138	21.4	20.61	0.96	3.7%
	-2	34.6	13.9306	241	45.3	36.35	0.80	19.8%
	-3	49.6	13.4274	333	98.2	59.99	0.61	38.9%
	-4	56	13.6071	381	194.3	75.89	0.39	60.9%
CRC-2%B-#3NS-100	-1	18.7	15.1872	142	15.9	12.55	0.79	21.1%
	-2	34.2	14.0351	240	31.1	21.72	0.70	30.2%
	-3	46.7	13.7045	320	No Data	N/A	N/A	N/A
	-4	54.9	12.3497	339	59.2	36.22	0.61	38.8%
	-5	64.3	13.0327	419	104.8	54.29	0.52	48.2%
CRC-2%A-#3NS-100	-1	16.5	15.8788	131	12.1	15.46	1.28	27.8%
	-2	39.5	13.5190	267	33.9	32.01	0.94	5.6%
	-3	47.4	13.6287	323	49.7	38.76	0.78	22.0%
	-4	56.5	12.9204	365	67.3	45.55	0.68	32.3%
	-5	67.4	13.0564	440	213.1	63.78	0.30	70.1%
CRC-2%A-#3HS-100	-1	17.3	15.4913	134	13.0	13.26	1.02	2.0%
	-2	35.4	13.7288	243	27.8	24.44	0.88	12.1%
	-3	46.7	14.9893	350	45.1	35.07	0.78	22.2%
	-4	58.3	14.4425	421	59.4	43.30	0.73	27.1%
	-5	68.5	13.6934	469	71.9	50.10	0.70	30.3%
	-6	87.2	12.2706	535	128.5	62.37	0.49	51.5%
CRC-2%A-#3SS-100	-1	18.6	14.4086	134	14.5	18.42	1.27	27.0%
	-2	35.8	14.1341	253	30.2	34.83	1.15	15.3%
	-3	45.5	14.9451	340	45.8	46.61	1.02	1.8%
	-4	57.8	13.5986	393	59.9	54.29	0.91	9.4%
	-5	63.2	14.5886	461	81.8	66.57	0.81	18.6%
	-6	71.3	14.0252	500	119.1	75.69	0.64	36.4%

Table 6-9 Analysis statistics for UHPFRC specimens per blast

Blast #	Mean	Standard deviation	Coefficient of variance	% Error
1-6	0.81	0.25	30.3%	25.6%
1	1.09	0.20	18.2%	17.4%
2	0.91	0.16	17.5%	14.0%
3	0.79	0.15	18.3%	21.4%
4	0.68	0.17	25.4%	32.0%
5	0.58	0.14	23.9%	41.8%
6	0.56	0.11	18.9%	44.0%

Table 6-10 Analysis statistics for UHPFRC specimens per blast (A-type fibres only)

Blast #	Mean	Standard deviation	Coefficient of variance	% Error
1-6	0.84	0.26	30.8%	24.2%
1	1.19	0.15	12.3%	18.9%
2	0.99	0.14	14.4%	11.0%
3	0.86	0.14	16.1%	15.3%
4	0.77	0.12	15.6%	22.9%
5	0.60	0.27	44.7%	39.7%
6	0.56	0.11	18.9%	30.0%

Table 6-11 Analysis statistics for UHPFRC specimens per blast (B-type fibres only)

Blast #	Mean	Std. Dev.	COV	% Error
1-6	0.76	0.23	29.7%	27.6%
1	0.99	0.22	22.2%	15.8%
2	0.84	0.16	19.0%	17.0%
3	0.70	0.12	17.2%	30.5%
4	0.59	0.19	32.1%	41.0%
5	0.52	N/A	N/A	48.2%

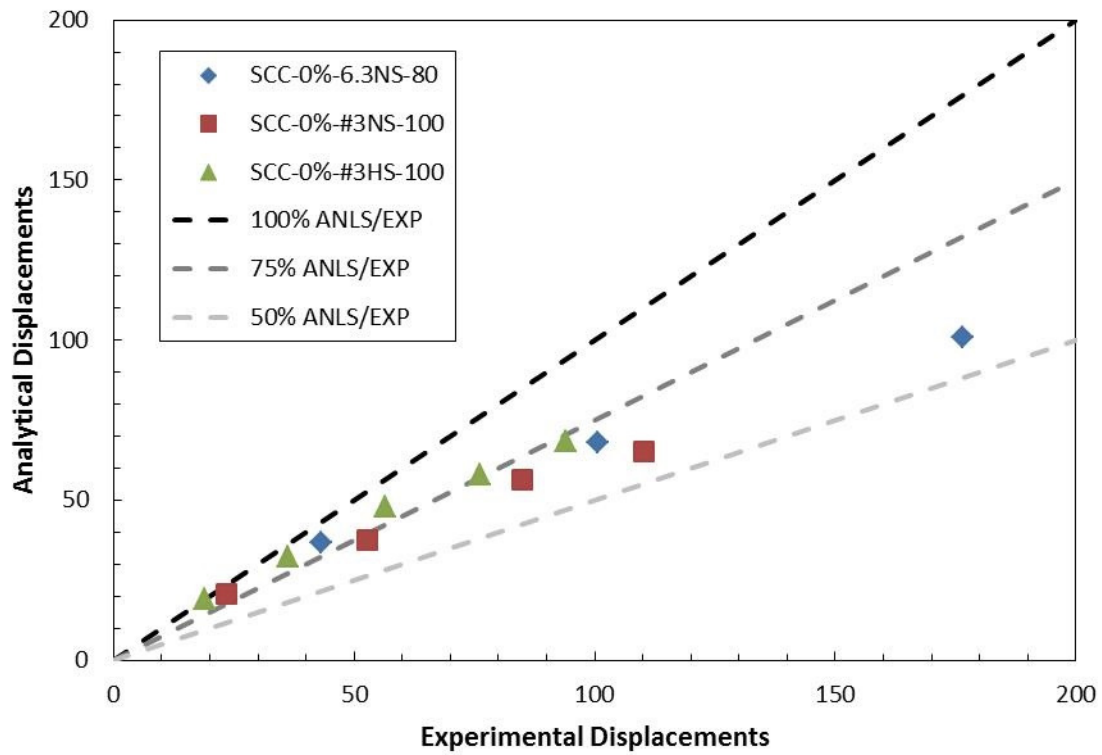


Figure 6-10 Experimental vs. Analytical Displacements - Plain concrete specimens

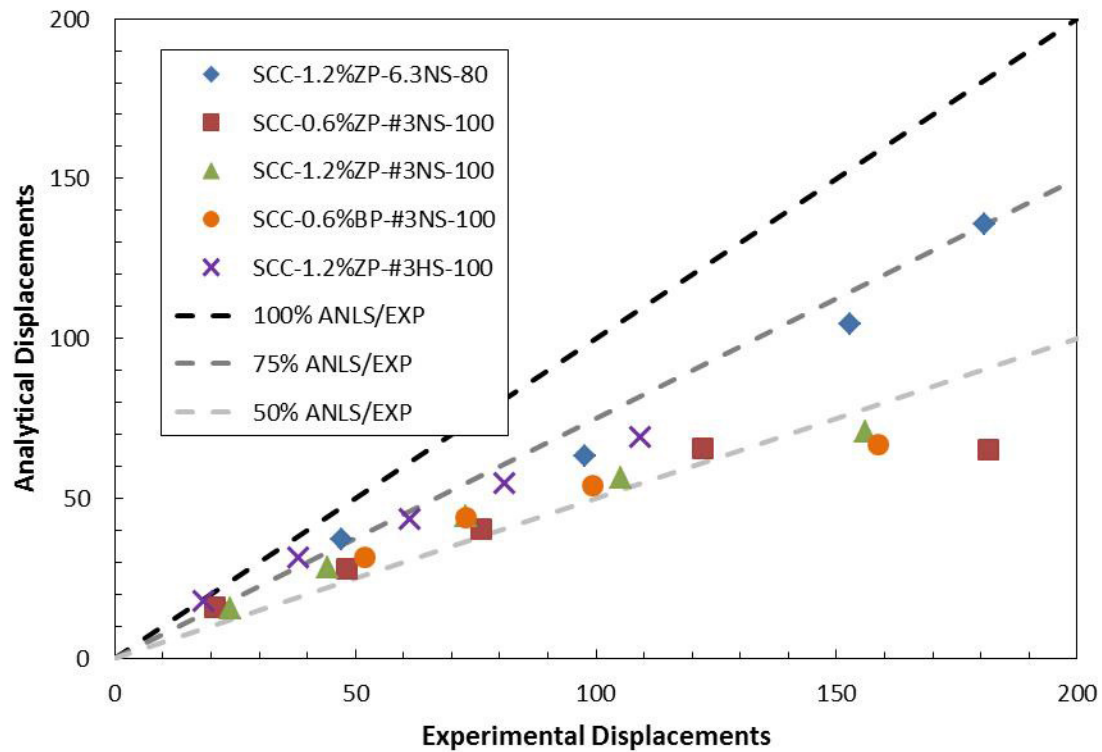


Figure 6-11 Experimental vs. Analytical Displacements - SFRC concrete specimens

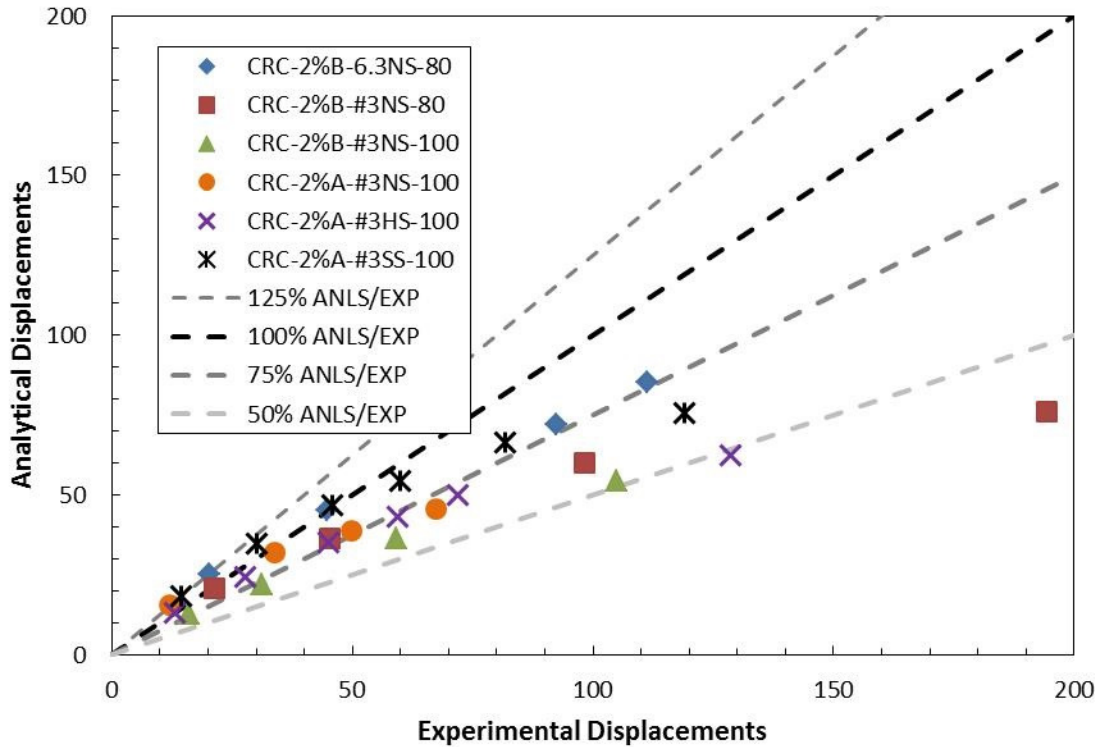


Figure 6-12 Experimental vs. Analytical Displacements - UHPFRC concrete specimens

6.6. SENSITIVITY ANALYSIS

A sensitivity analysis was required to select an appropriate combination of concrete and steel models with their respective dynamic increase factors. It can also be used to examine the influence of certain parameters on the SDOF predictions.

Each specimen was analyzed using different combinations of material models and dynamic increase factors. The dynamic analysis was conducted using the software RCBlas (Jacques 2014) and responses were compared with the displacement data obtained from the experimental program. The results were compiled within three categories: plain concrete, SFRC and UHPFRC specimens.

6.6.1. SENSITIVITY ANALYSIS - MODEL SELECTION (PLAIN CONCRETE SPECIMENS)

For plain concrete specimens, including SCC-0%-6.3NS-80, SCC-0%-#3NS-100 and SCC-0%-#3HS-100, two concrete models were analyzed: Kent and Park (1971) and Cusson and Paultre (1995). Dynamic increase factors were applied to both concrete models based on the equations by the Euro-international Concrete Committee (1990) for concrete in compression, and Malvar and Ross (1998) for concrete in tension. Steel models for this category include experimental stress-strain curves, the Jacques et al. (2012) model for regular steel rebar and the ACI Group ITG-6 (2010) high strength steel model. A dynamic increase factor was applied to steel for half of the combination cases. The factor is based on equations by Saatcioglu et al. (2011).

For each analysis, a different combination was selected. Case-A and Case-B are combinations which include the Kent and Park (1971) equations while Case-C and Case-D include the Cusson and Paultre (1995) equations. Experimental steel stress-strain curves were used for Case-A and Case-C while other steel models were implemented in Case-B and Case-D, depending on the composition of the specimen. Cases followed by an apostrophe have their steel dynamic increase factor set to a conservative 1.00 value. Table 6-12 summarizes these combinations.

Table 6-12 Sensitivity analysis combination cases - Plain concrete

		Steel Models	
		<i>Experimental</i>	<i>Jacques et al. (2012)</i> <i>ACI Group ITG-6 (2010)</i>
Concrete Models	<i>Kent & Park (1971)</i>	CASE-A & CASE-A'	CASE-B & CASE-B'
	<i>Cusson & Paultre (1995)</i>	CASE-C & CASE-C'	CASE-D & CASE-D'

Based on Figure 6-13 through Figure 6-15, the Cusson & Paultre (1995) equations from Case-C and Case-D generally provide a better estimate of the maximum displacements than the other concrete model when both have dynamic increase factors applied. As for steel reinforcement, the use of analytical models in the dynamic analysis of Case-D led to closer results than the experimental stress-strain curves of Case-C. Finally, the dynamic increase factor of steel reinforcement was observed to have a major effect on the prediction of the response of reinforced concrete slab with regular strength steel (Case-D vs. Case-D'). However, the properties of reinforcement steel at high strain rates shouldn't be neglected. Therefore, Case-D was selected, when applicable, as the final combination for this analysis. Case-C was selected for specimen SCC-0%-6.3NS-80 as no analytical model could be implemented for the steel reinforcement model.

6.6.2. SENSITIVITY ANALYSIS – MODEL SELECTION (SFRC SPECIMENS)

For steel fibre reinforced concrete, the concrete stress-strain curve is defined by the models proposed by either Aoude (2008) or Ou et al. (2012) for concrete in compression and Lok and Pei (1998) for concrete in tension. Dynamic increase factors were applied to both concrete models based on the equations by Zhang and Mindness (2011) for concrete in compression, and Malvar and Ross (1998) for concrete in tension. Steel models for this category include experimental stress-strain curves, the Jacques et al. (2012) model and the ACI Group ITG-6 high strength steel model. For steel models, a dynamic increase factor has been applied to half the combination cases based on equations by Saatcioglu et al. (2011).

For each analysis, a different combination was selected. Case-A and Case-B are combinations which include the Aoude (2008) equations while Case-C and Case-D include the Ou et al. (2012) equations. The experimental steel stress-strain curves were used for Case-A and Case-C, while the Jacques et al. (2012) and ITG-6 (2010) equations were implemented for Case-B and Case-D

when applicable. Cases followed by an apostrophe have their steel dynamic increase factor set to a conservative 1.00 value. Table 6-13 summarizes these combinations.

Table 6-13 Sensitivity analysis combination cases - SFRC concrete

		Steel Models	
		<i>Experimental</i>	<i>Jacques et al. (2012)</i> <i>ACI Group ITG-6 (2010)</i>
Concrete Models	<i>Aoude (2008)</i> <i>Lok & Pei (1998)</i>	CASE-A & CASE-A'	CASE-B & CASE-B'
	<i>Ou et al. (2012)</i> <i>Lok & Pei (1998)</i>	CASE-C & CASE-C'	CASE-D & CASE-D'

Similarly to the plain concrete sensitivity analysis, Figure 6-16 through Figure 6-20 represent the different analysis cases for each SFRC specimen. The dynamic analysis responses for cases with the Aoude (2008) compression model were generally slightly closer to the measured experimental values when compared to the Ou et al. (2012) cases. For ZP-fibre specimens with regular reinforcement, SCC-0.6%ZP-#3NS-100 and SCC-1.2%ZP-#3NS-100, the analytical steel model in Jacques et al. (2012) provided a better estimate of the response. For the BP-fibre specimen, SCC-0.6%BP-#3NS-100, both steel models resulted in similar displacement predictions. As for the high strength steel specimen SCC-1.2%ZP-#HS-100, the analytical model from ITG-6 (2010) had closer results when an increase factor was applied. Finally, the analysis with a steel DIF set to 1.00 generally provided better predictions; however, the dynamic increase factor for steel during high strain rate events shouldn't be neglected from the analysis. Hence, Case-B will be selected when available for the final analysis results. Case-A will be selected in the case of specimens without analytical steel models such as SCC-1.2%ZP-6.3NS-80.

6.6.3. SENSITIVITY ANALYSIS – MODEL SELECTION (UHPFRC SPECIMENS)

For ultra-high performance fibre reinforced concrete, the concrete stress-strain curve is defined by the models proposed by Hosinieh et al. (2015) for concrete in compression and Lok and Pei (1998) for concrete in tension. Dynamic increase factors were applied to both concrete models based on the equations by Zhang and Mindness (2011) for concrete in compression, and Malvar and Ross (1998) for concrete in tension. Steel models for this category include experimental stress-strain curves, the RCBlast (2014) model and the ACI Group ITG-6 (2010) high strength steel model. For steel models, a dynamic increase factor has been applied to half the combination cases based on equations by Saatcioglu et al. (2011).

For each analysis, a different combination was selected. The concrete models were applied to each case along with their respective dynamic increase factor. The experimental steel stress-strain curves were used for Case-A, while the Jacques et al. (2012) and ITG-6 (2010) equations were implemented for Case-B, when applicable. Cases followed by an apostrophe have their steel dynamic increase factor set to 1.00. Table 6-14 summarizes these combinations.

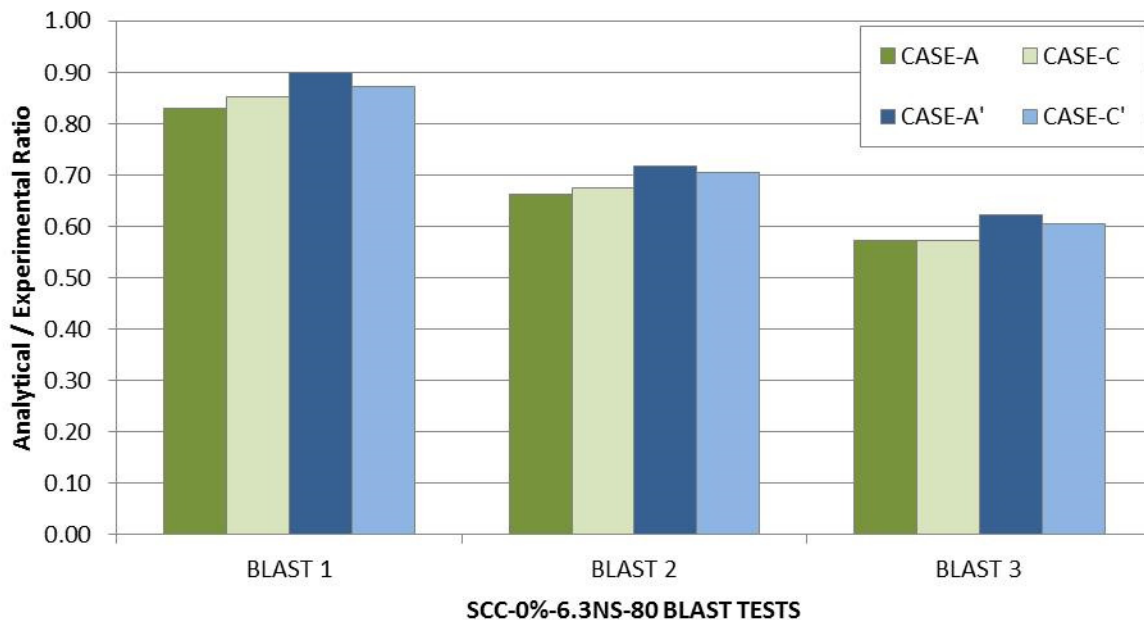
Table 6-14 Sensitivity analysis combination cases - UHPFRC concrete

		Steel Models	
		<i>Experimental</i>	<i>Jacques et al. (2012)</i> <i>ACI Group ITG-6 (2010)</i>
Concrete Models	Hosinieh et al. (2015) <i>Lok & Pei (1998)</i>	CASE-A & CASE-A'	CASE-B & CASE-B'

Similarly to the plain concrete sensitivity analysis, Figure 6-21 through Figure 6-26 represent the different analysis cases for each UHPFRC specimen. For four of six specimens, the steel dynamic increase factor barely influenced the magnitude of the predicted response; specimens CRC-2%B-#3NS-80 and CRC-2%B-#3NS-100 saw improvements in the prediction of the maximum displacement when the DIF was set to 1.00. Again, for the purpose of this research, it was decided that the dynamic increase factor of steel should not be neglected and that it will be included in the final analysis results. As for the steel model, the models by Jacques et al. (2012) and ITG-6 (2010) were slightly more favorable during the later explosions. Hence, Case-B will be selected when available for the final analysis results. Case-A will be selected in the case of specimens without analytical steel models such as CRC-2%B-6.3NS-80 and CRC-2%A-#3SS-100.

6.6.4. SENSITIVITY ANALYSIS – SUMMARY OF CASES FOR MODEL SELECTION

The following section regroups all fourteen bars charts of the sensitivity analysis discussed in the previous section. Each bar chart includes analytical over experimental displacement ratios, the specimen's name, case names and blast test numbers.

**Figure 6-13 Sensitivity analysis - SCC-0%-6.3NS-80**

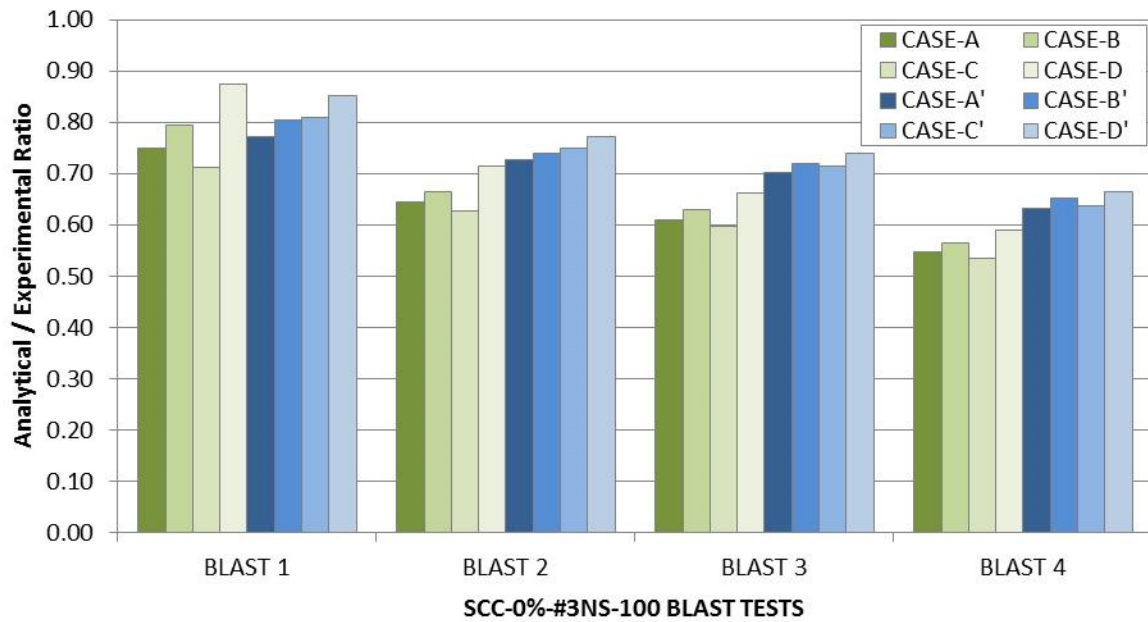


Figure 6-14 Sensitivity analysis - SCC-0% -#3NS-100

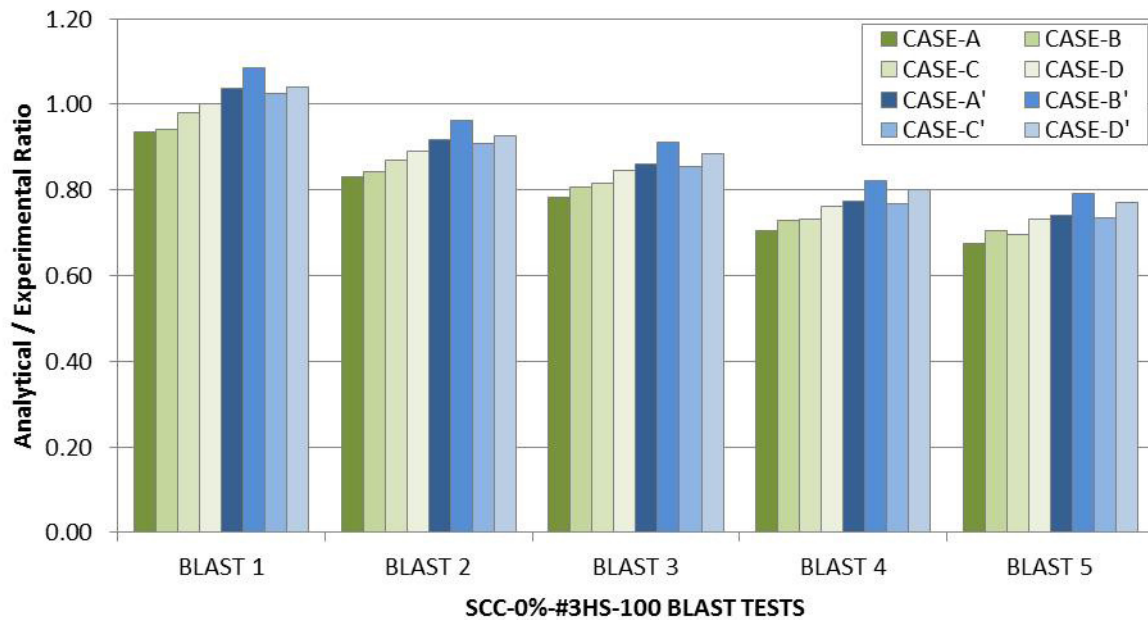


Figure 6-15 Sensitivity analysis - SCC-0% -#3HS-100

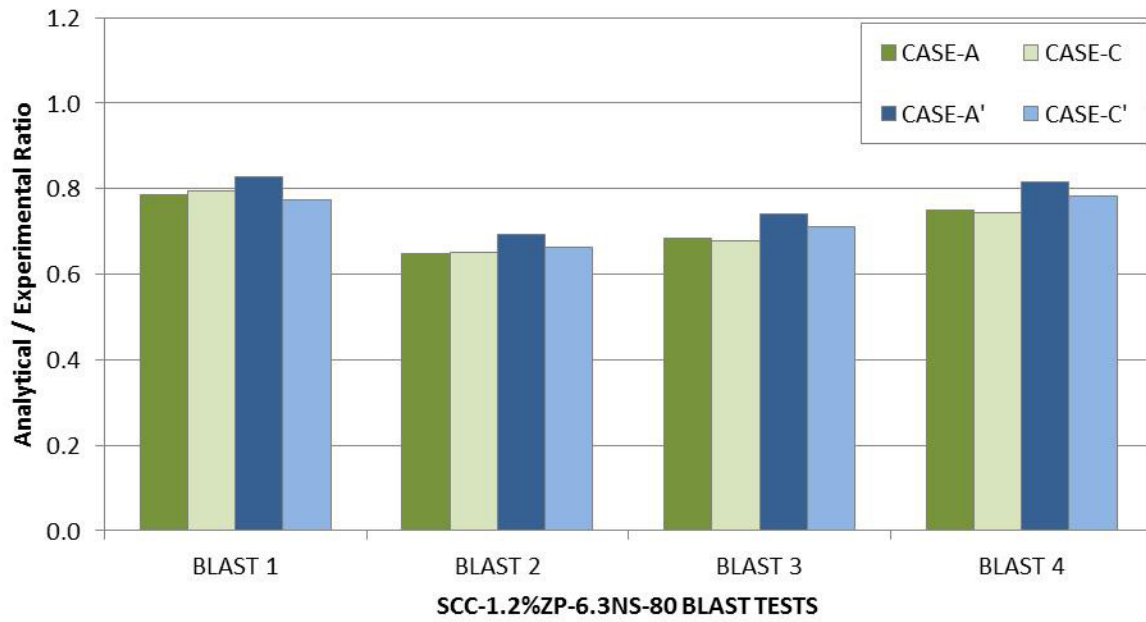


Figure 6-16 Sensitivity analysis - SCC-1.2% ZP-6.3NS-80

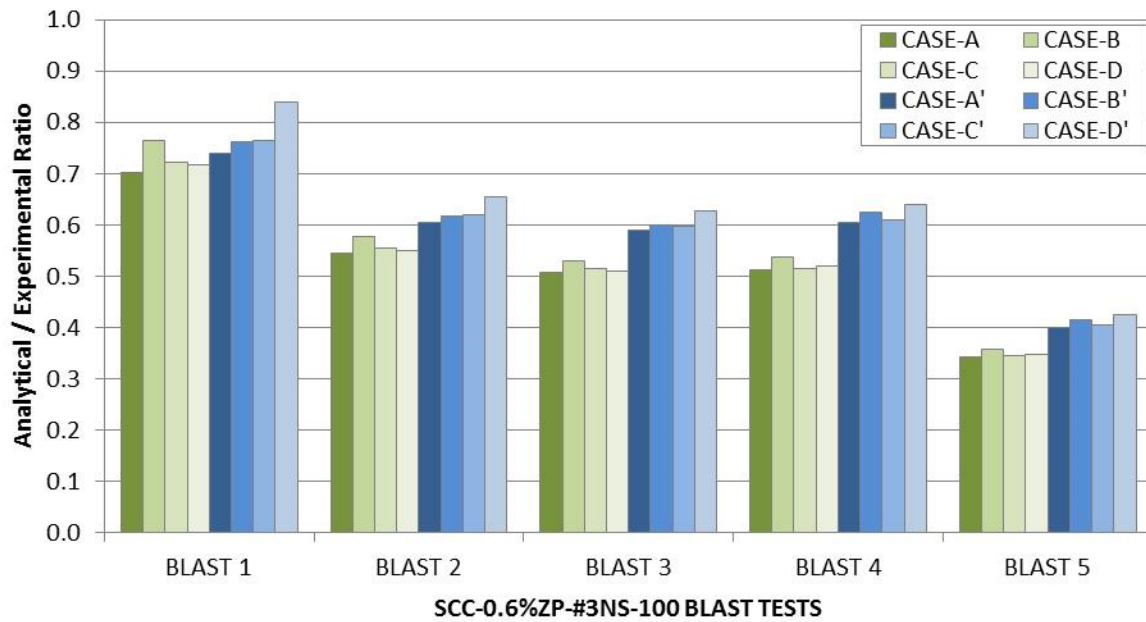


Figure 6-17 Sensitivity analysis - SCC-0.6% ZP-#3NS-100

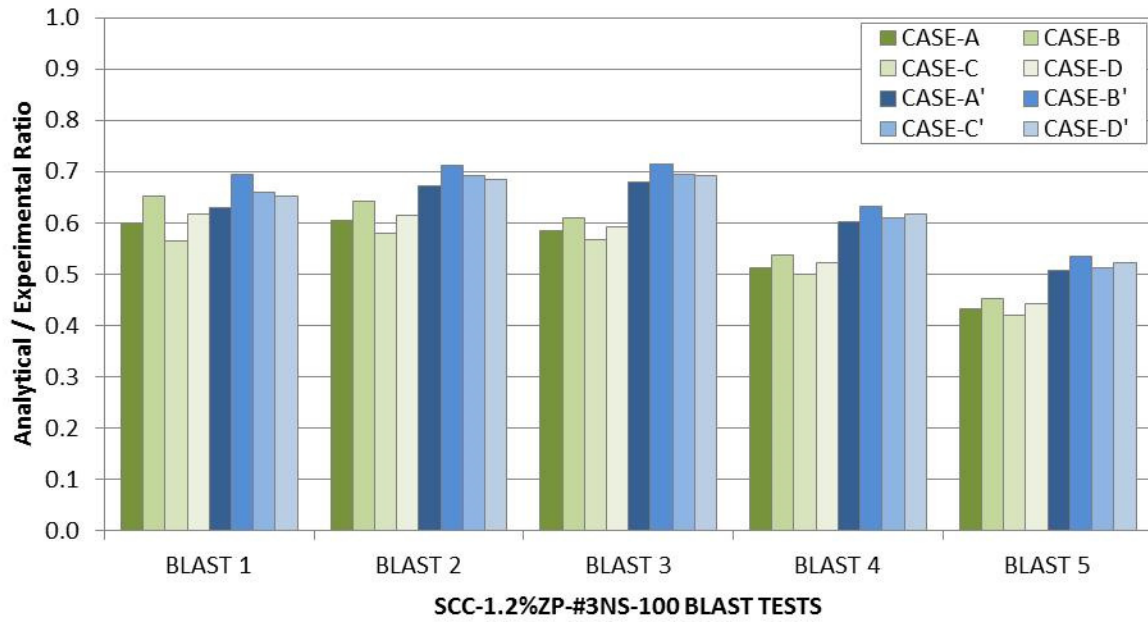


Figure 6-18 Sensitivity analysis - SCC-1.2% ZP-#3NS-100

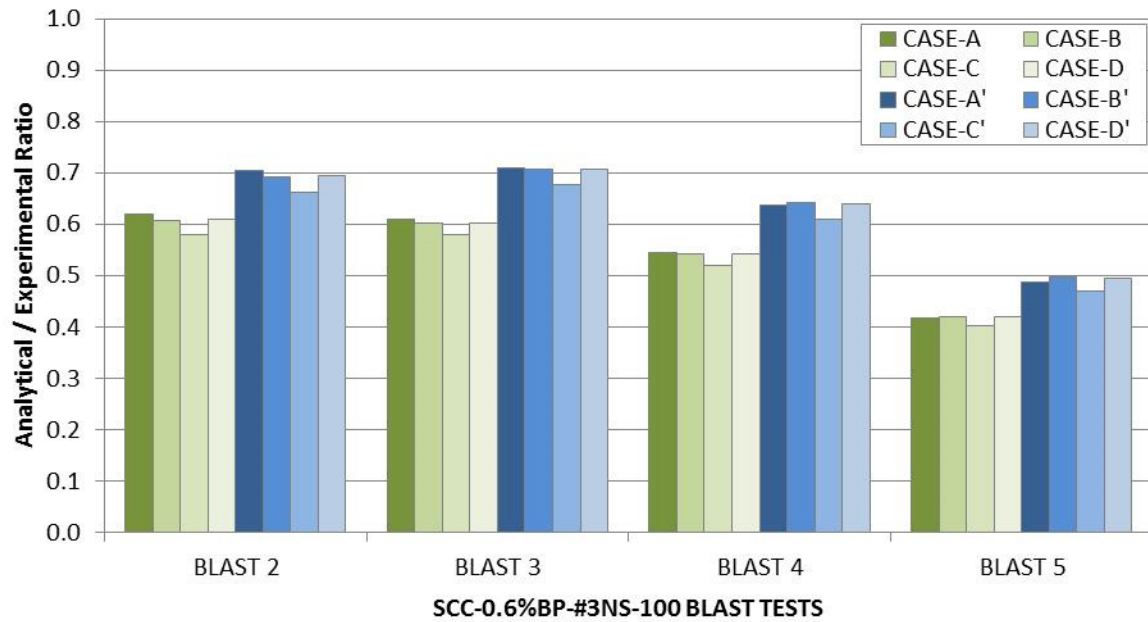


Figure 6-19 Sensitivity analysis - SCC-0.6% BP-#3NS-100

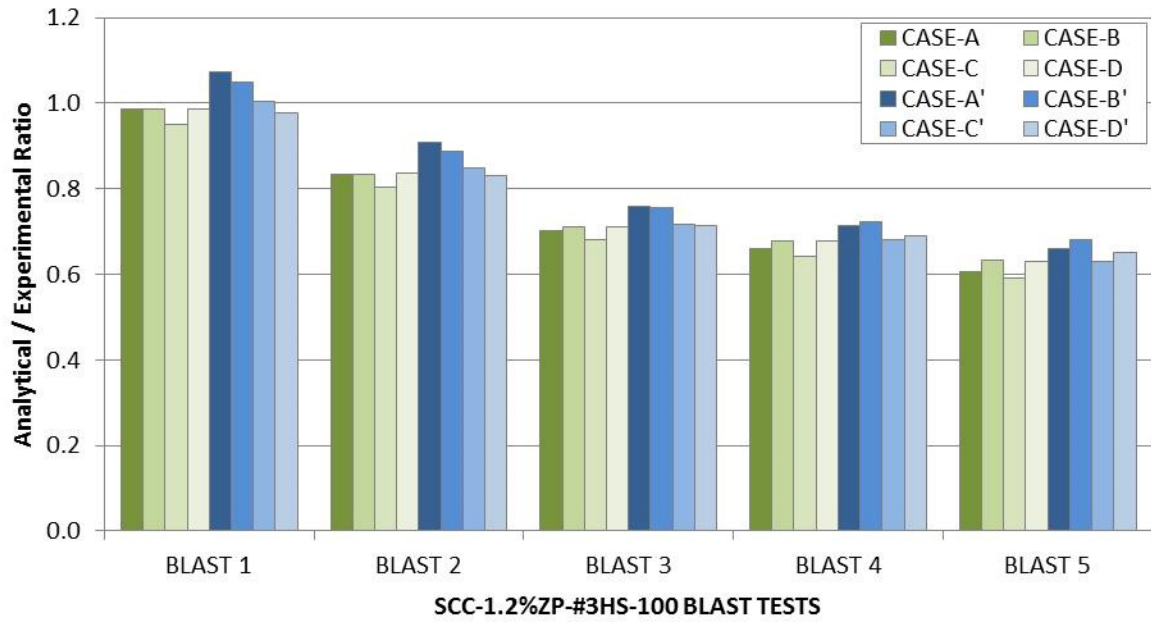


Figure 6-20 Sensitivity analysis - SCC-1.2% ZP-#3HS-100

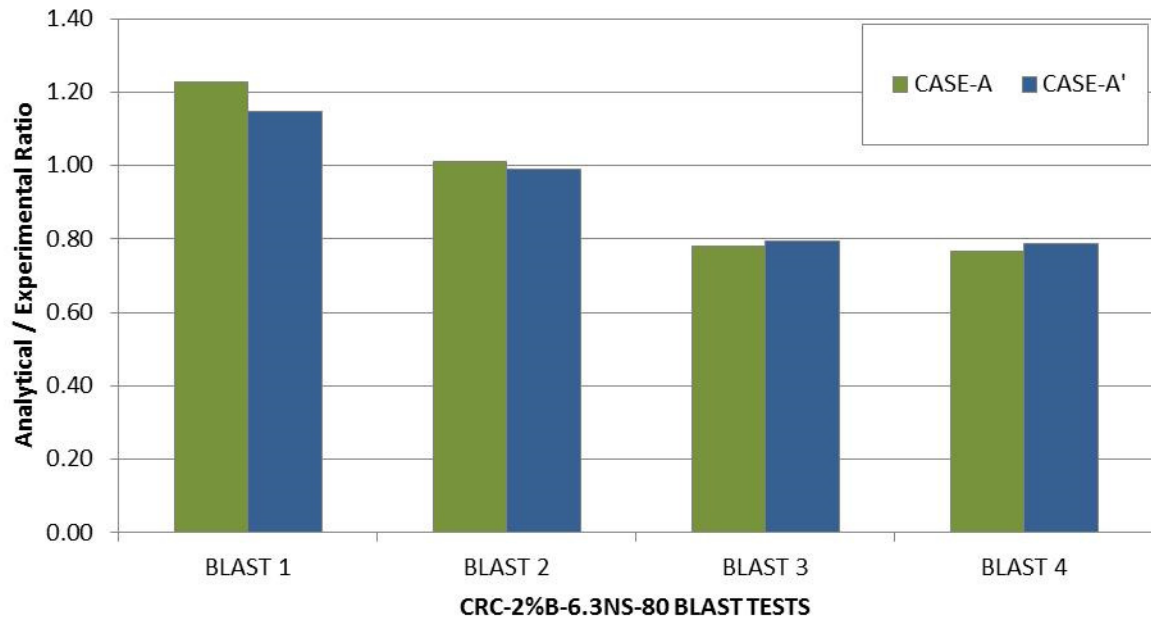


Figure 6-21 Sensitivity analysis - CRC-2%B-6.3NS-80

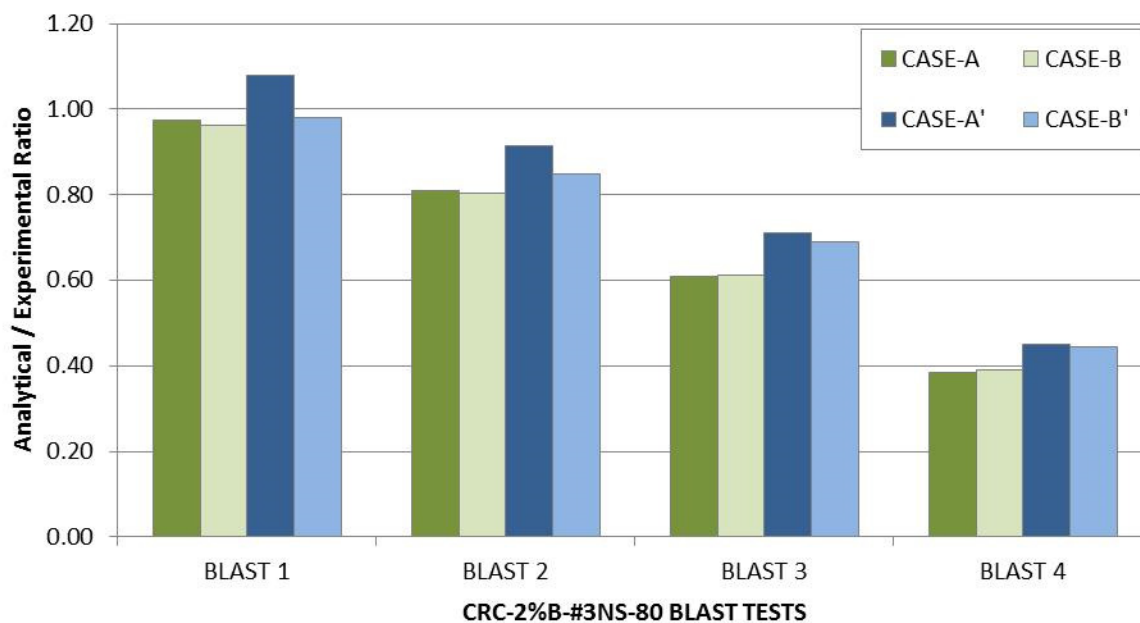


Figure 6-22 Sensitivity analysis - CRC-2%B-#3NS-80

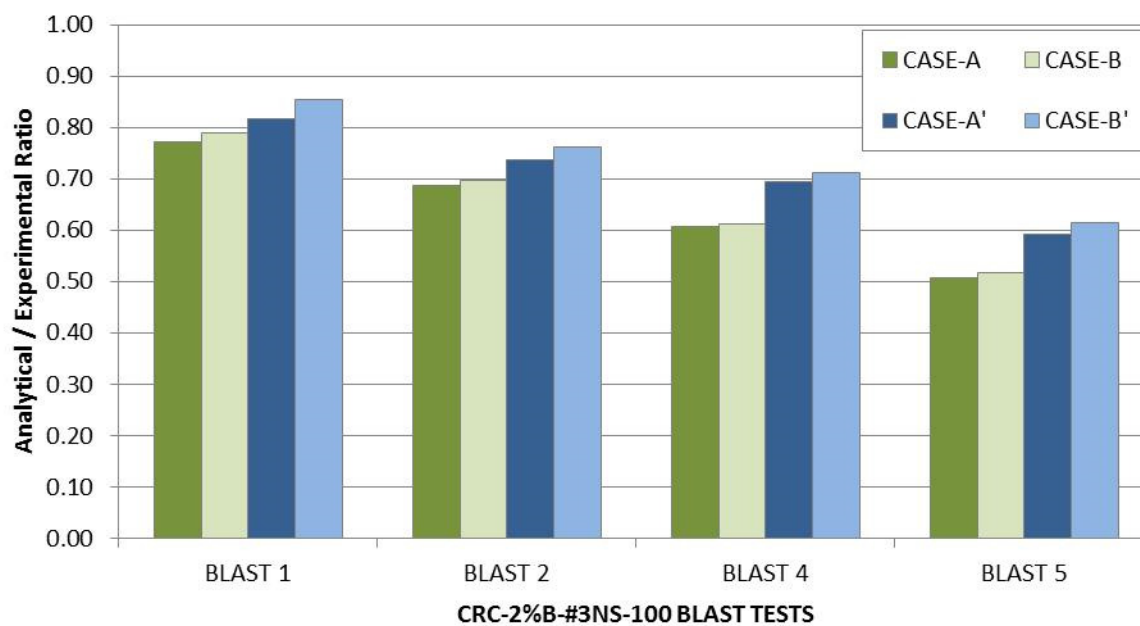


Figure 6-23 Sensitivity analysis - CRC-2%B-#3NS-100

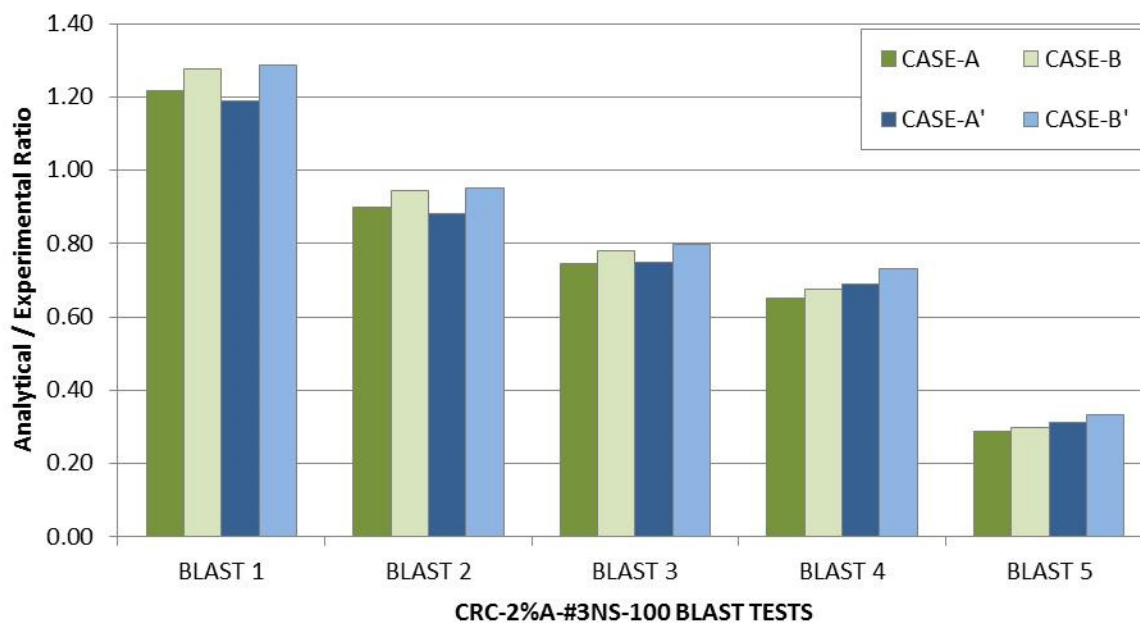


Figure 6-24 Sensitivity analysis - CRC-2% A-#3NS-100

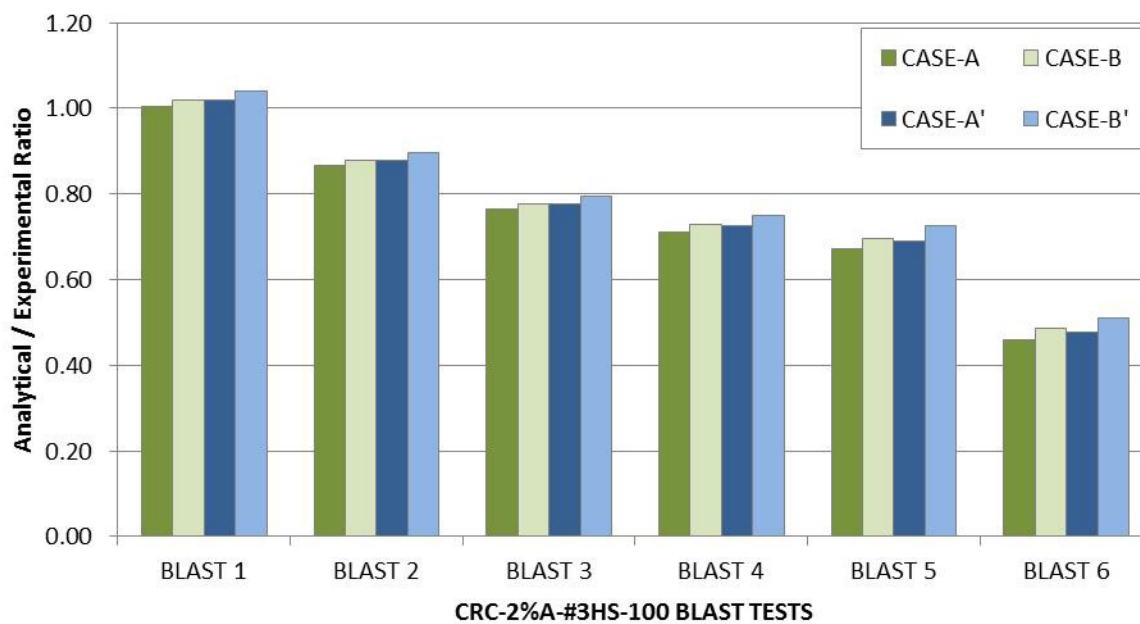


Figure 6-25 Sensitivity analysis - CRC-2% A-#3HS-100

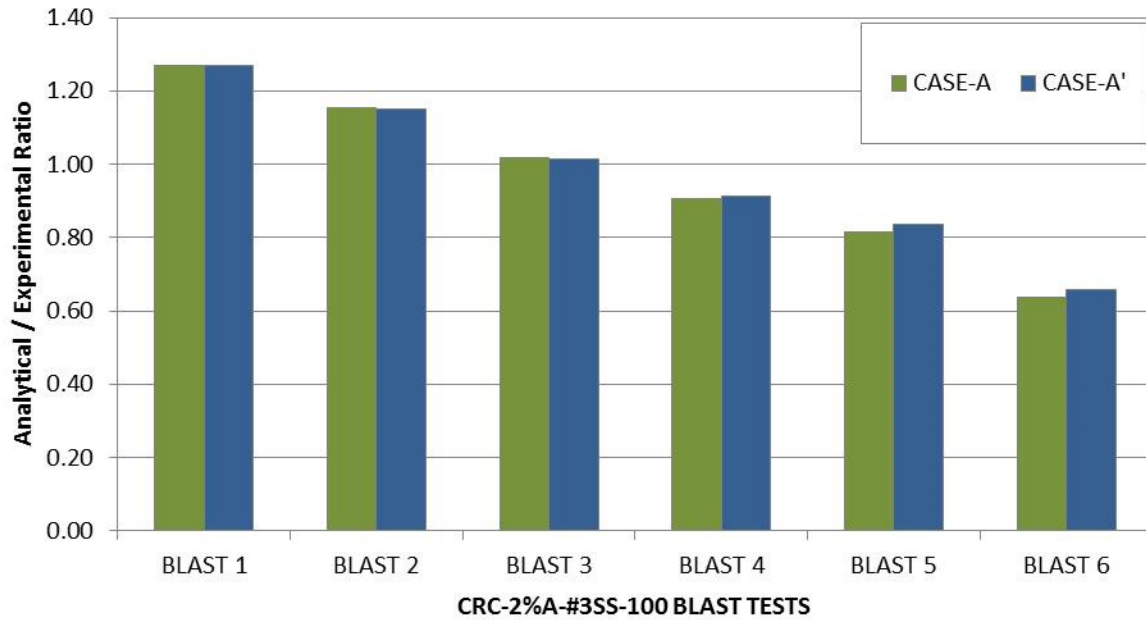


Figure 6-26 Sensitivity analysis - CRC-2%A-#3SS-100

6.6.5. SENSITIVITY ANALYSIS – DYNAMIC INCREASE FACTORS

The dynamic increase factor is a parameter which has the potential to greatly influence the analytical response obtained from a dynamic analysis. While some models were extracted from various studies such as the Zhang and Mindness (2011) and Malvar and Ross (1998) equations for calculating the DIF of concrete in compression and tension, other models commonly used in blast analysis and in the design of blast-resistant reinforced concrete structures have been recommended such as the CEB model and the UFC-3-340-02 (2008) design factors from Table 6-2. To observe the effects of the dynamic increase factor on the results of a dynamic analysis, three different cases were analyzed for specimen SCC-0%-#3NS-100. The concrete model used for this sensitivity analysis was developed by Cusson & Paultre (1995) while the steel model was defined by Jacques et al. (2012). The various cases with their respective DIF are compiled in Table 6-15.

Table 6-15 Sensitivity analysis – Dynamic increase factors for SCC-0%-#3NS-100

	Dynamic increase factors		
	Concrete in compression	Concrete in tension	Steel reinforcement
CASE-A	CEB (1990) $DIF_c = 1.23$	Malvar & Ross (1998) $DIF_{ct} = 1.34$	Saatcioglu et al. (2011) $DIF_y = 1.30, DIF_u = 1.10$
CASE-B	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-C	$DIF_c = 1.00$	$DIF_{ct} = 1.00$	$DIF_y = 1.00, DIF_u = 1.00$

The analytical against experimental displacement ratios obtained from the dynamic analysis of all three cases are compiled in Figure 6-27. Unsurprisingly, the results show that the selection of

a dynamic increase factor can influence significantly the value of the analytical response, where a lower dynamic increase factor suggests a greater deflection. During blast 4, the displacement difference between Case-A and Case-C was nearly 13%; however, during blast 1, this behaviour was not observed.

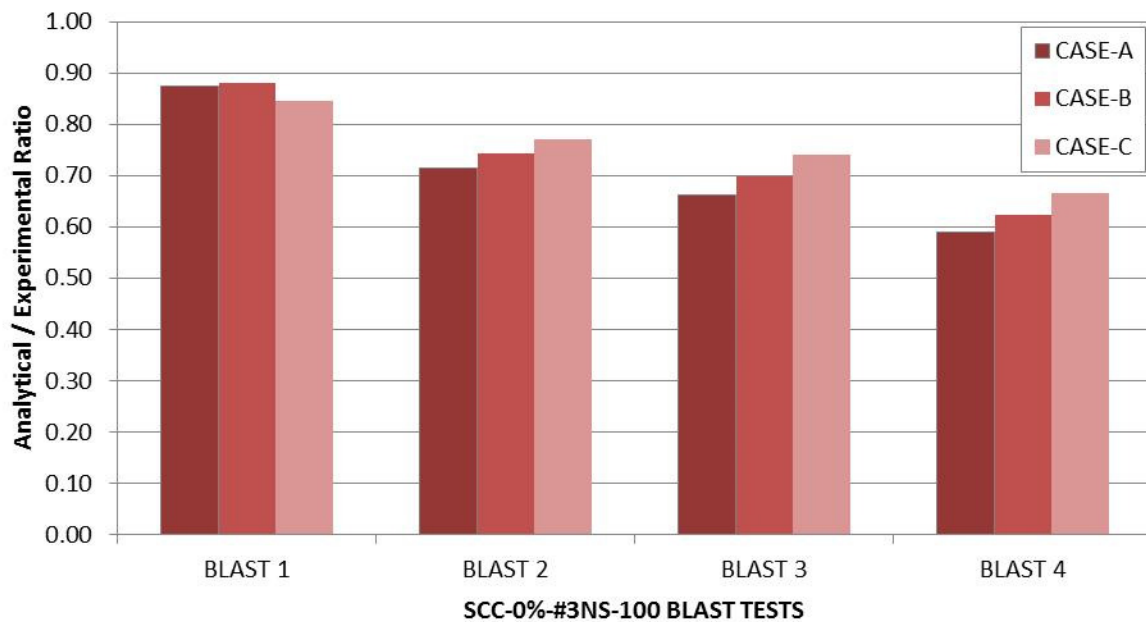


Figure 6-27 Sensitivity analysis - Dynamic increase factors, SCC-0%-#3NS-100

Additional cases can be computed for SFRC specimens. A sensitivity analysis was conducted on specimen SCC-1.2%ZP-#3NS-100 with four different cases of dynamic increase factors. These cases are listed in Table 6-16. The concrete model used for this sensitivity analysis was developed by Aoude (2008) for SFRC in compression and Lok and Pei (1998) for SFRC in tension. The steel model was defined by Jacques et al. (2012).

Table 6-16 Sensitivity analysis – Dynamic increase factors for SCC-1.2%ZP-#3NS-100

	Dynamic increase factors		
	Concrete in compression	Concrete in tension	Steel reinforcement
CASE-A	Zhang & Mindness (2011) $DIF_c = 1.38$	Malvar & Ross (1998) $DIF_{ct} = 1.47$	Saatcioglu et al. (2011) $DIF_y = 1.30, DIF_u = 1.10$
CASE-B	CEB (1990) $DIF_c = 1.31$	Malvar & Ross (1998) $DIF_{ct} = 1.47$	Saatcioglu et al. (2011) $DIF_y = 1.30, DIF_u = 1.10$
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	$DIF_c = 1.00$	$DIF_{ct} = 1.00$	$DIF_y = 1.00, DIF_u = 1.00$

The analytical against experimental displacement ratios obtained from the dynamic analysis of all four cases were compiled in Figure 6-28. Again, the results show that the analytical response

of SFRC slabs is quite sensitive to the selection of the dynamic increase factor models. For instance, during blasts 3 to 5, the displacement of Case-D was nearly 26% greater than Case-A.

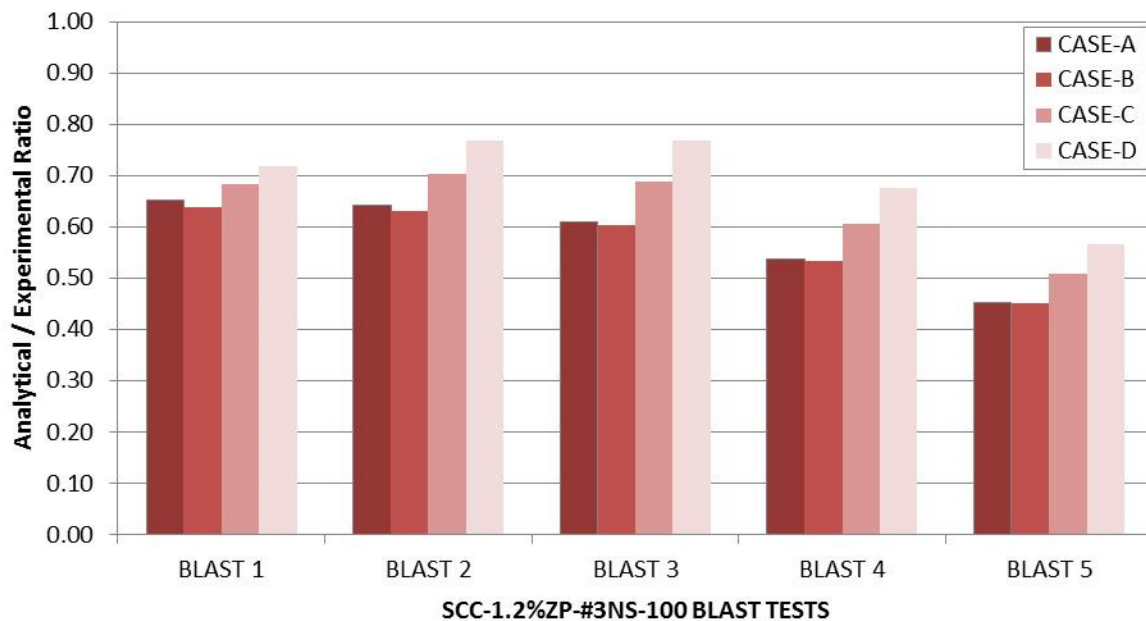


Figure 6-28 Sensitivity analysis - Dynamic increase factors, SCC-1.2%ZP-#3NS-100

The same cases were also applied for a sensitivity analysis with the UHPFRC specimen CRC-2%A-#3NS-100. The dynamic increase factors are tabulated in Table 6-17; it should be noted that no particular DIF model is well-established for UHPFRC. The concrete model used for this sensitivity analysis was proposed by Hosinieh et al. (2015) for UHPFRC in compression and Lok and Pei (1998) for UHPFRC in tension. The steel model was defined by Jacques et al. (2012).

Table 6-17 Sensitivity analysis – Dynamic increase factors for CRC-2%A-#3NS-100

	Dynamic increase factors		
	Concrete in compression	Concrete in tension	Steel reinforcement
CASE-A	Zhang & Mindness (2011) $DIF_c = 1.21$	Malvar & Ross (1998) $DIF_{ct} = 1.14$	Saatcioglu et al. (2011) $DIF_y = 1.30, DIF_u = 1.10$
CASE-B	CEB (1990) $DIF_c = 1.09$	Malvar & Ross (1998) $DIF_{ct} = 1.14$	Saatcioglu et al. (2011) $DIF_y = 1.30, DIF_u = 1.10$
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	$DIF_c = 1.00$	$DIF_{ct} = 1.00$	$DIF_y = 1.00, DIF_u = 1.00$

The analytical against experimental displacement ratios obtained from the dynamic analysis of all four cases were compiled in Figure 6-29. Once again, the results show that the analytical response of UHPFRC slabs is sensitive to the selection of the dynamic increase factor models.

The percentage difference for predicting deflections between Case-A and Case-D is about 10% for the early blasts and can reach up to 22% for the later tests.

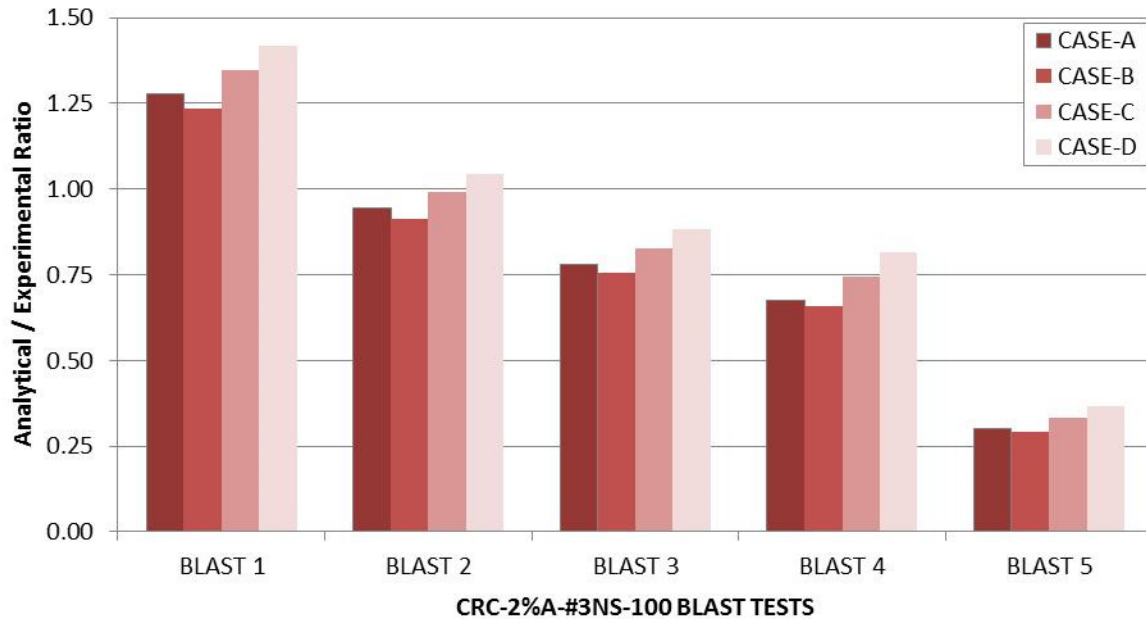


Figure 6-29 Sensitivity analysis - Dynamic increase factors, CRC-2%A-#3NS-100

6.6.6. SENSITIVITY ANALYSIS – CONCRETE IN TENSION MODEL

Unlike conventional concrete, the provision of fibres in SFRC and UHPFRC can result in important tension resistance which can influence blast response. Therefore, the selection of a proper concrete in tension stress-strain model for such materials can significantly influence the analytical response obtained from a dynamic analysis. A sensitivity analysis was conducted on four cases in order to observe the effects of the tensile stress-strain models on the analytical displacements of the SFRC slabs. In the case of slab SCC-1.2%ZP-#3NS-100, the concrete in compression model used for this sensitivity analysis was developed by Aoude (2008) while the steel model was described by Jacques et al. (2012). Dynamic increase factors were estimated using equations by Zhang and Mindness (2011) for concrete in compression ($DIF_c = 1.38$), and Saatcioglu et al. (2011) for steel ($DIF_y = 1.30$, $DIF_u = 1.10$). These dynamic increase factors were kept identical in all four cases described in Table 6-18.

The analytical over experimental displacement ratios obtained from the dynamic analysis of all four cases were compiled in Figure 6-30. In Case-A and Case-B, the dynamic increase factor for concrete in tension remains the same however the stress-strain model varies between Lok and Pei (1998) or the Bekaert guidelines. Based on the dynamic analysis, it seems that Lok and Pei (1998) provides a closer prediction than the Bekaert model; this could be associated with an over-estimation of the concrete toughness in tension by the Bekaert model. Between Case-A and

Case-C, the dynamic increase factor was reduced to a value of 1.00 leading to a slightly lower resistance. As for Case-D, ignoring the action of concrete in tension increased the analytical displacement by 16 to 23% when compared with Case-A. Such observations can lead to question the actual contribution of steel fibres to the flexural behaviour of conventional SFRC slabs when subjected to dynamic loads.

Table 6-18 Sensitivity analysis – Tensile models and dynamic increase factors for SCC-1.2%ZP-#3NS-100

	Tensile models & dynamic increase factors	
	Concrete in tension model	Dynamic increase factor
CASE-A	Lok & Pei (1998)	Malvar & Ross (1998) $DIF_{ct} = 1.47$
CASE-B	Bekaert Dramix Guidelines (Nemegeer 1998)	Malvar & Ross (1998) $DIF_{ct} = 1.47$
CASE-C	Lok & Pei (1998)	$DIF_{ct} = 1.00$
CASE-D	Concrete in tension is ignored.	

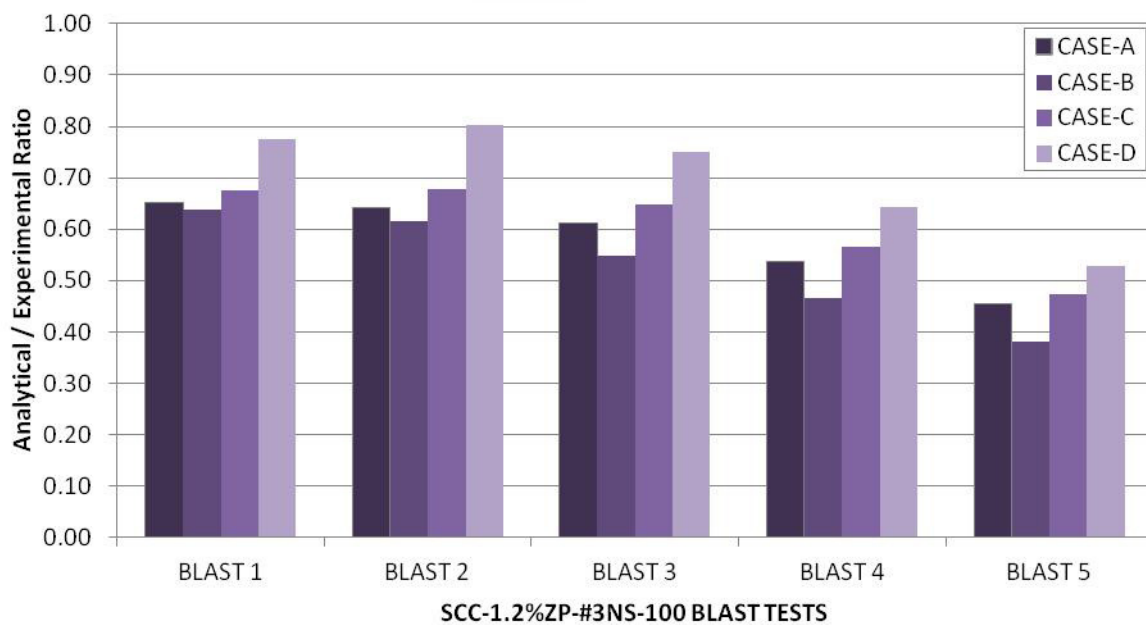


Figure 6-30 Sensitivity analysis - Concrete in tension models, SCC-1.2% ZP-#3NS-100

6.6.7. SENSITIVITY ANALYSIS – FULL OR PARTIAL MASS PARTICIPATION

When conducting the dynamic analysis, it was assumed that the mass of the load transfer device (210 kg) should be combined with the mass of the specimen (188 kg for 80 mm specimens or 235 kg for 100 mm specimens) as both components seem to deflect together during blast tests. However, as slabs are permanently deformed at later blasts, this composite behaviour can be questioned as the specimen isn't resting on the load transfer device when the shock-tube's blast load is initiated similarly to Figure 6-31(a) where the slab had a total residual deflection of 83 mm after three tests. During blast 4, the load transfer device only reaches the control

specimen SCC-0%-#3NS-100 in Figure 6-31(b) after 12 milliseconds. The peak is then achieved after 44 milliseconds as illustrated through Figure 6-31(c).

The effects of full and partial mass participation on the maximum analytical displacements were investigated using a sensitivity analysis. In the case of full mass participation (Case-A), both the mass of the load transfer device and the specimens were considered for the dynamic analysis. For partial mass participation (Case-B), only the mass of the specimen was considered. Concrete and steel models were taken as listed in Table 6-3 with their respective dynamic increase models. Cases are summarized in Table 6-19. Results from the sensitivity analysis are compiled in function of their concrete type in Table 6-20, Table 6-21 and Table 6-22.

The analysis of full and partial mass participation provided upper and lower bound maximum displacement predictions. These values are presented in function of their concrete type in Figure 6-33, Figure 6-35 and Figure 6-37 along with average markers to differentiate each specimen. Sample analytical over experimental ratios for a plain concrete specimen (SCC-0%-#3NS-100), an SFRC specimen (SCC-1.2%ZP-#3NS-100) and a UHPFRC specimen (CRC-2%A-#3HS-100) are shown in Figure 6-32, Figure 6-34 and Figure 6-36 respectively.

From the results, it is noted that conducting the dynamic analysis using full mass participation provided respectively close predictions for the initial blast tests (e.g. Blast 1 and 2) of SCC-0%-#3NS-100 and CRC-2%A-#3HS-100. On the other hand, analytical displacements obtained from partial mass participation cases are significantly closer to the experimental values during later blast tests for all three samples (e.g. Blasts 3 and above). The same trend is observed for most SCC, SFRC and CRC specimens. This leads to the hypothesis that the mass of the dynamic system could be influenced by the interaction between the load transfer device and the concrete panels, especially when conducting repeated blast tests. Therefore when specimens have permanent deformation prior to blast application, as is the case at later blasts, analysis using partial mass participation may be more appropriate - further research is recommended to study this effect in specimens tested under repeated shock-tube loading.

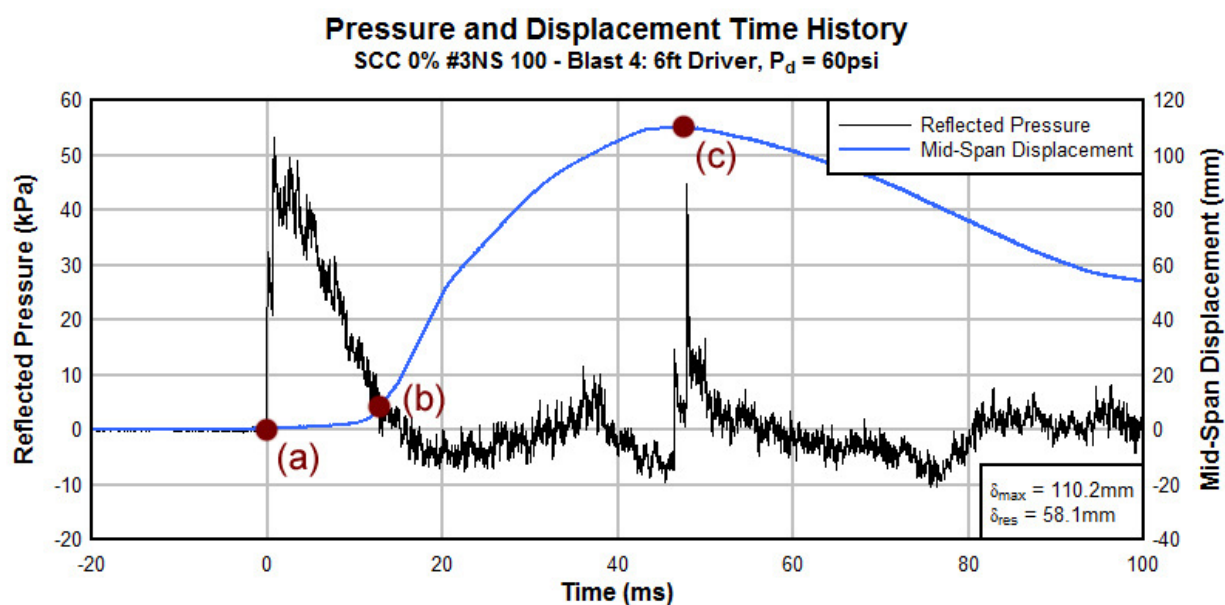
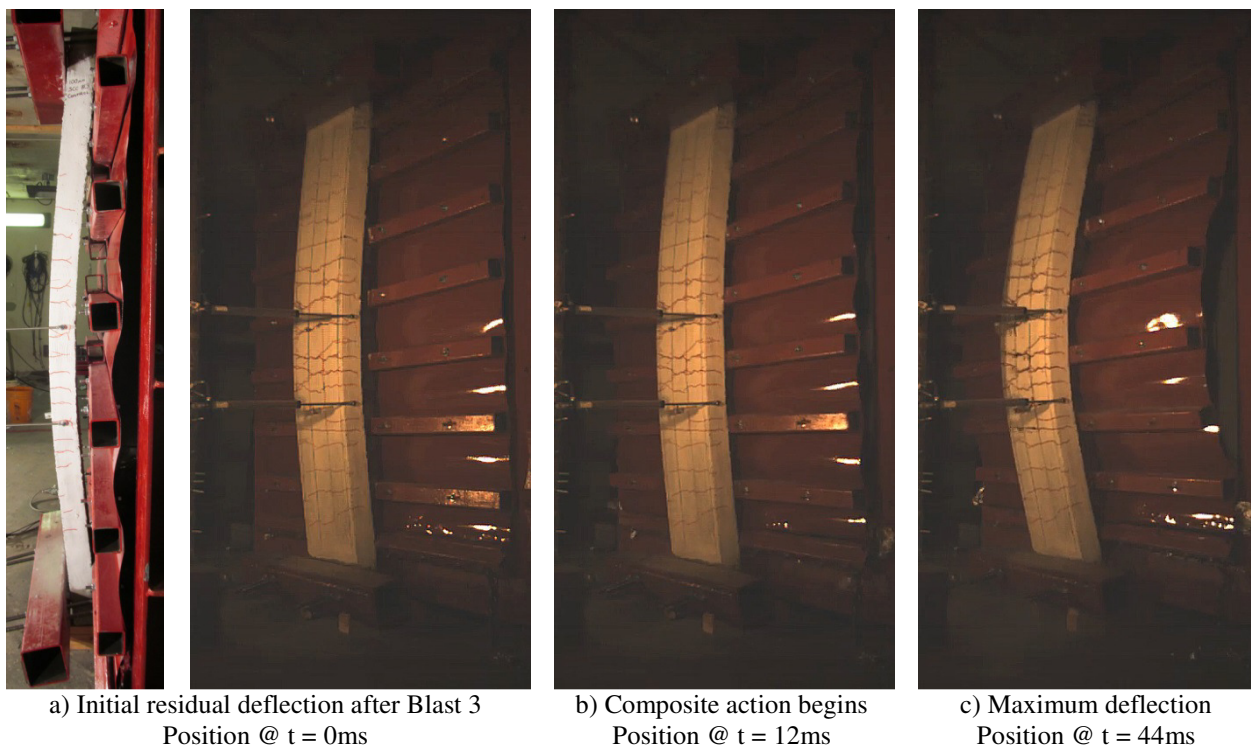


Figure 6-31 SCC-0% -#3NS-100, Blast 4, delay between LTD and specimen deflection

Table 6-19 Sensitivity analysis – Cases for full and partial mass participation

Cases	80 mm specimens	100 mm specimens
Full Mass Participation (Case-A)	398 kg	445 kg
Partial Mass Participation (Case-B)	188 kg	235 kg

Table 6-20 Summary of sensitivity analysis for plain concrete specimens in regards to mass participation

Specimen	Blast #	δ_{exp} mm	Full Mass Participation			Partial Mass Participation		
			δ_{anls} mm	$\delta_{anls}/\delta_{exp}$	% Error %	δ_{anls} mm	$\delta_{anls}/\delta_{exp}$	% Error %
SCC-0%-6.3NS-80	-1	43.2	36.80	0.85	14.8%	56.58	1.31	31.0%
	-2	100.6	67.85	0.67	32.6%	117.90	1.17	17.2%
	-3	176.4	100.76	0.57	42.9%	191.29	1.08	8.4%
SCC-0%-#3NS-100	-1	23.5	20.59	0.87	12.5%	26.65	1.13	13.2%
	-2	52.8	37.74	0.71	28.6%	55.08	1.04	4.2%
	-3	84.9	56.28	0.66	33.7%	87.21	1.03	2.7%
	-4	110.2	65.00	0.59	41.0%	103.52	0.94	6.1%
	-5	No Data	N/A	N/A	N/A	N/A	N/A	N/A
SCC-0%-#3HS-100	-1	18.9	18.95	1.00	0.3%	25.33	1.34	34.0%
	-2	36.3	32.33	0.89	10.9%	44.19	1.22	21.7%
	-3	56.6	47.86	0.85	15.4%	67.01	1.18	18.4%
	-4	76.1	58.04	0.76	23.7%	84.18	1.11	10.6%
	-5	93.9	68.58	0.73	27.0%	103.10	1.10	9.8%
	-6	No Data	N/A	N/A	N/A	N/A	N/A	N/A

Table 6-21 Summary of sensitivity analysis for SFRC specimens in regards to mass participation

Specimen	Blast #	δ_{exp} mm	Full Mass Participation			Partial Mass Participation		
			δ_{anls} mm	$\delta_{anls}/\delta_{exp}$	% Error %	δ_{anls} mm	$\delta_{anls}/\delta_{exp}$	% Error %
SCC-1.2%ZP-6.3NS-80	-1	47.1	36.98	0.79	21.5%	59.90	1.27	27.2%
	-2	97.6	63.13	0.65	35.3%	119.91	1.23	22.9%
	-3	152.8	104.49	0.68	31.6%	210.39	1.38	37.7%
	-4	180.7	135.69	0.75	24.9%	222.81	1.23	23.3%
SCC-0.6%ZP-#3NS-100	-1	20.7	15.82	0.76	23.57%	20.83	1.01	0.6%
	-2	48.1	27.81	0.58	42.18%	42.05	0.87	12.6%
	-3	76.2	40.45	0.53	46.92%	66.93	0.88	12.2%
	-4	122.1	65.60	0.54	46.27%	113.53	0.93	7.0%
	-5	181.6	65.15	0.36	64.12%	113.42	0.62	37.5%
SCC-1.2%ZP-#3NS-100	-1	23.9	15.59	0.65	34.77%	20.51	0.86	14.2%
	-2	44.2	28.34	0.64	35.82%	42.82	0.97	3.0%
	-3	72.9	44.55	0.61	38.89%	74.51	1.02	2.2%
	-4	105.1	56.47	0.54	46.27%	97.08	0.92	7.6%
	-5	155.9	70.73	0.45	54.63%	123.57	0.79	20.7%
SCC-0.6%BP-#3NS-100	-1	22.6	N/A	N/A	N/A	N/A	N/A	N/A
	-2	52.0	31.60	0.61	39.23%	49.51	0.95	4.8%
	-3	73.1	43.95	0.60	39.88%	73.60	1.01	0.7%
	-4	99.2	53.87	0.54	45.70%	92.44	0.93	6.8%
	-5	158.6	66.85	0.42	57.85%	116.87	0.74	26.3%
SCC-1.2%ZP-#3HS-100	-1	18.3	18.03	0.99	1.48%	23.94	1.31	30.8%
	-2	38.0	31.72	0.83	16.53%	42.65	1.12	12.2%
	-3	61.3	43.56	0.71	28.94%	61.49	1.00	0.3%
	-4	80.9	54.78	0.68	32.29%	82.17	1.02	1.6%
	-5	109.3	69.06	0.63	36.82%	103.77	0.95	5.1%

Table 6-22 Summary of sensitivity analysis for UHPFRC concrete specimens in regards to mass participation

Specimen	Blast #	δ_{exp} mm	Full Mass Participation			Partial Mass Participation		
			δ_{anls} mm	$\delta_{anls}/\delta_{exp}$	% Error %	δ_{anls} mm	$\delta_{anls}/\delta_{exp}$	% Error %
CRC-2%B-6.3NS-80	-1	20.3	24.92	1.23	22.8%	35.16	1.73	73.2%
	-2	44.8	45.22	1.01	1.0%	88.22	1.97	97.0%
	-3	92.3	72.00	0.78	22.0%	148.19	1.61	60.6%
	-4	111.2	85.26	0.77	23.3%	178.43	1.60	60.5%
CRC-2%B-#3NS-80	-1	21.4	20.61	0.96	-3.7%	28.94	1.35	35.2%
	-2	45.3	36.35	0.80	-19.8%	61.02	1.35	34.7%
	-3	98.2	59.99	0.61	-38.9%	112.75	1.15	14.8%
	-4	194.3	75.89	0.39	-60.9%	148.25	0.76	23.7%
CRC-2%B-#3NS-100	-1	15.9	12.55	0.79	21.1%	15.86	1.00	-0.3%
	-2	31.1	21.72	0.70	30.2%	30.30	0.97	-2.6%
	-3	No Data	N/A	N/A	N/A	N/A	N/A	N/A
	-4	59.2	36.22	0.61	38.8%	63.30	1.07	6.9%
	-5	104.8	54.29	0.52	48.2%	98.21	0.94	-6.3%
CRC-2%A-#3NS-100	-1	12.1	15.46	1.28	27.8%	19.39	1.60	60.2%
	-2	33.9	32.01	0.94	5.6%	41.81	1.23	23.3%
	-3	49.7	38.76	0.78	22.0%	58.57	1.18	17.8%
	-4	67.3	45.55	0.68	32.3%	75.91	1.13	12.8%
	-5	213.1	63.78	0.30	70.1%	111.19	0.52	47.8%
CRC-2%A-#3HS-100	-1	13.0	13.26	1.02	2.0%	16.97	1.31	30.5%
	-2	27.8	24.44	0.88	12.1%	31.89	1.15	14.7%
	-3	45.1	35.07	0.78	22.2%	46.95	1.04	4.1%
	-4	59.4	43.30	0.73	27.1%	63.35	1.07	6.6%
	-5	71.9	50.10	0.70	30.3%	78.28	1.09	8.9%
	-6	128.5	62.37	0.49	51.5%	91.59	0.71	28.7%
CRC-2%A-#3SS-100	-1	14.5	18.42	1.27	27.0%	24.49	1.69	68.9%
	-2	30.2	34.83	1.15	15.3%	46.35	1.53	53.5%
	-3	45.8	46.61	1.02	1.8%	64.27	1.40	40.3%
	-4	59.9	54.29	0.91	9.4%	81.98	1.37	36.9%
	-5	81.8	66.57	0.81	18.6%	105.69	1.29	29.2%
	-6	119.1	75.69	0.64	36.4%	122.91	1.03	3.2%

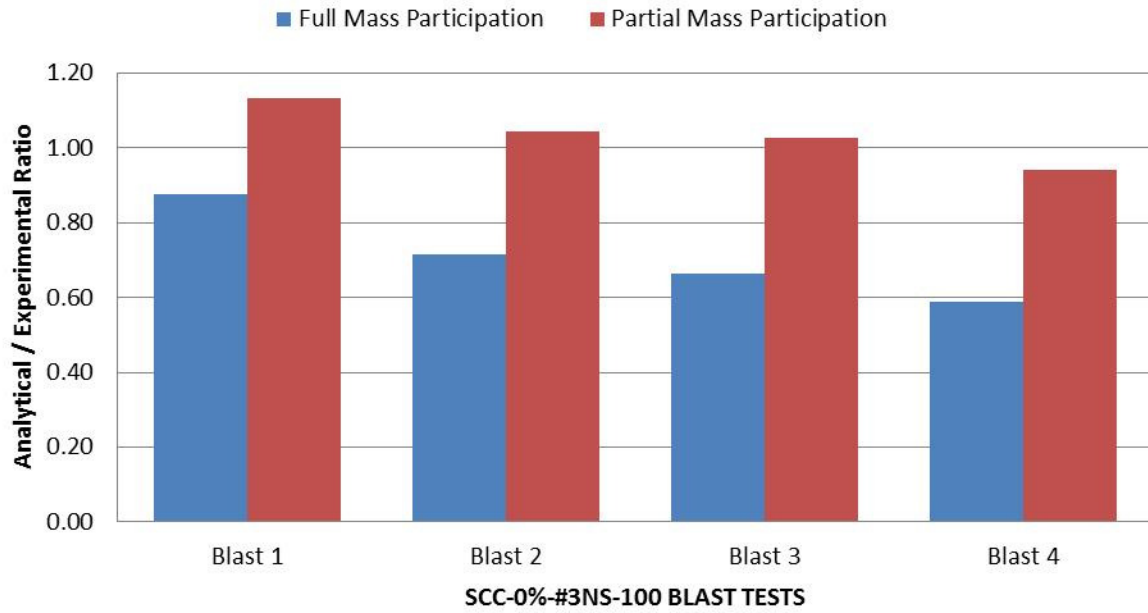


Figure 6-32 Sensitivity analysis – Mass participation, SCC-0%-#3NS-100

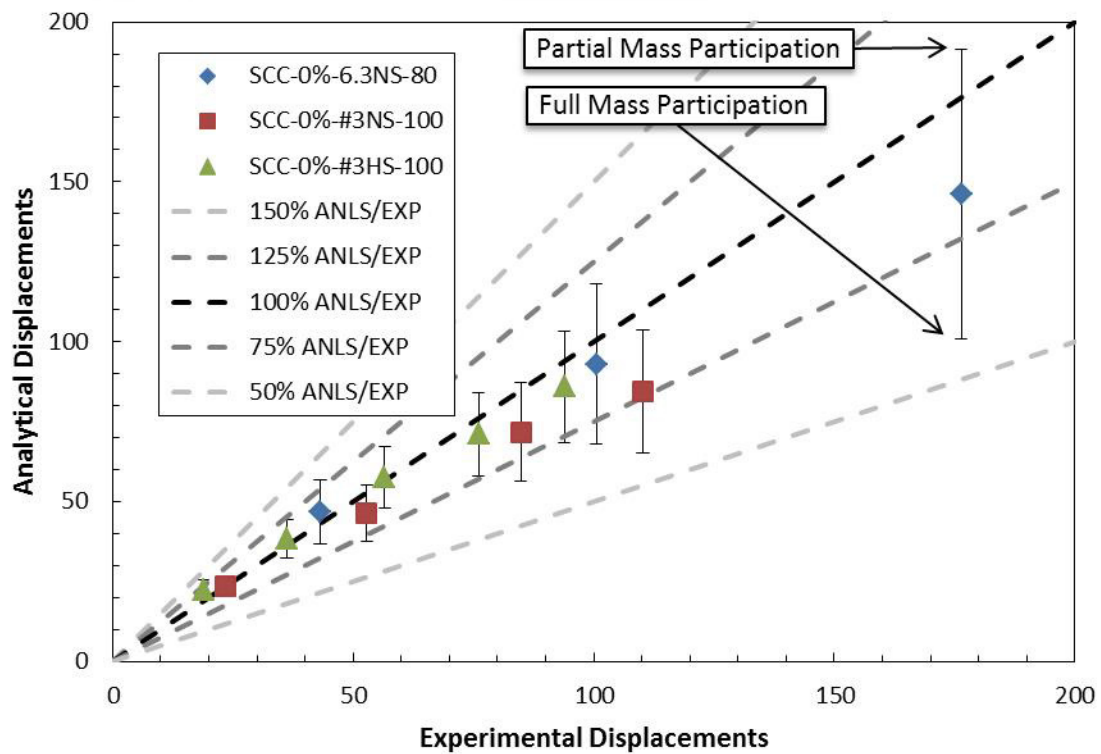


Figure 6-33 Sensitivity analysis – Mass participation, Plain concrete specimens

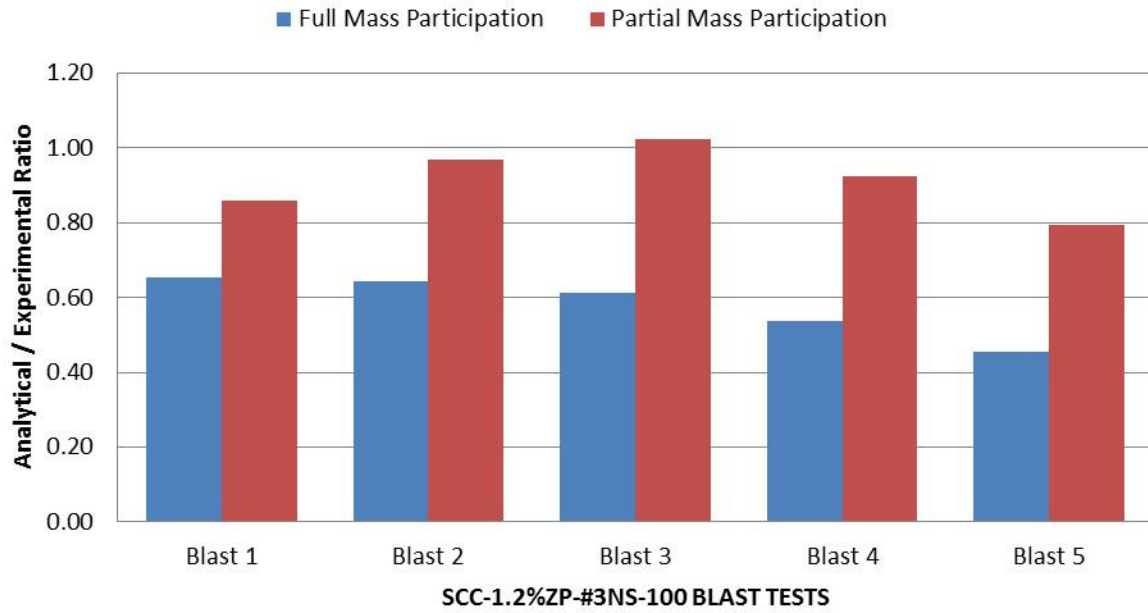


Figure 6-34 Sensitivity analysis – Mass participation, SCC-1.2%ZP-#3NS-100

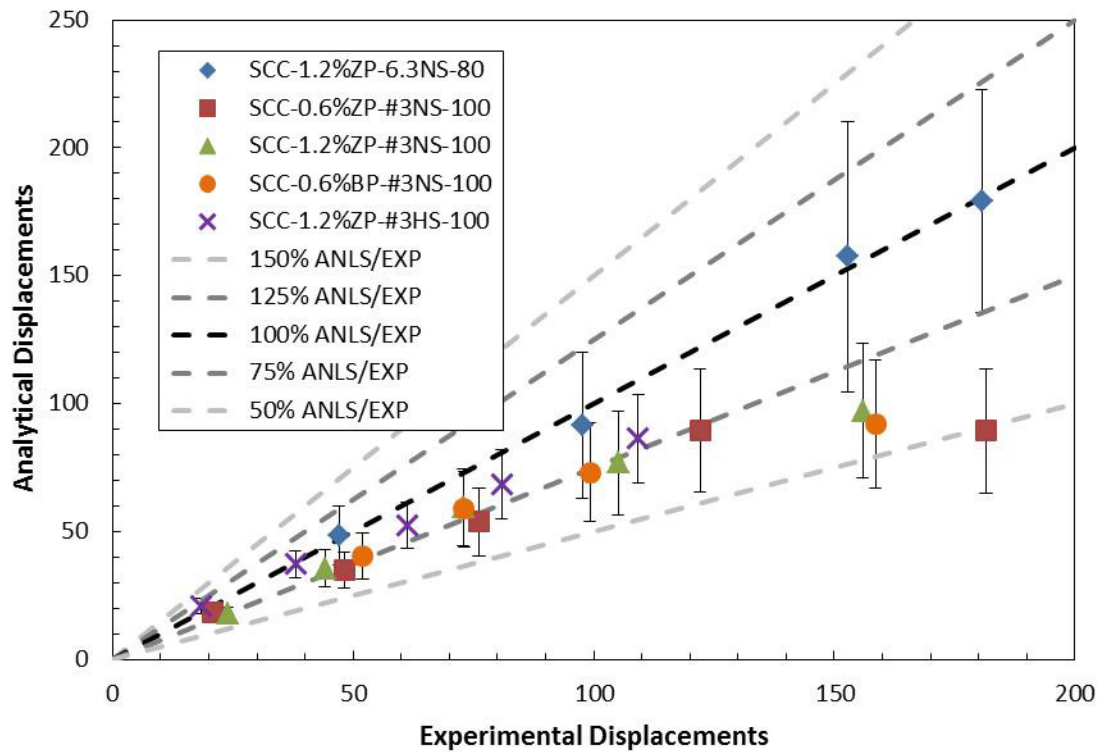


Figure 6-35 Sensitivity analysis – Mass participation, SFRC specimens

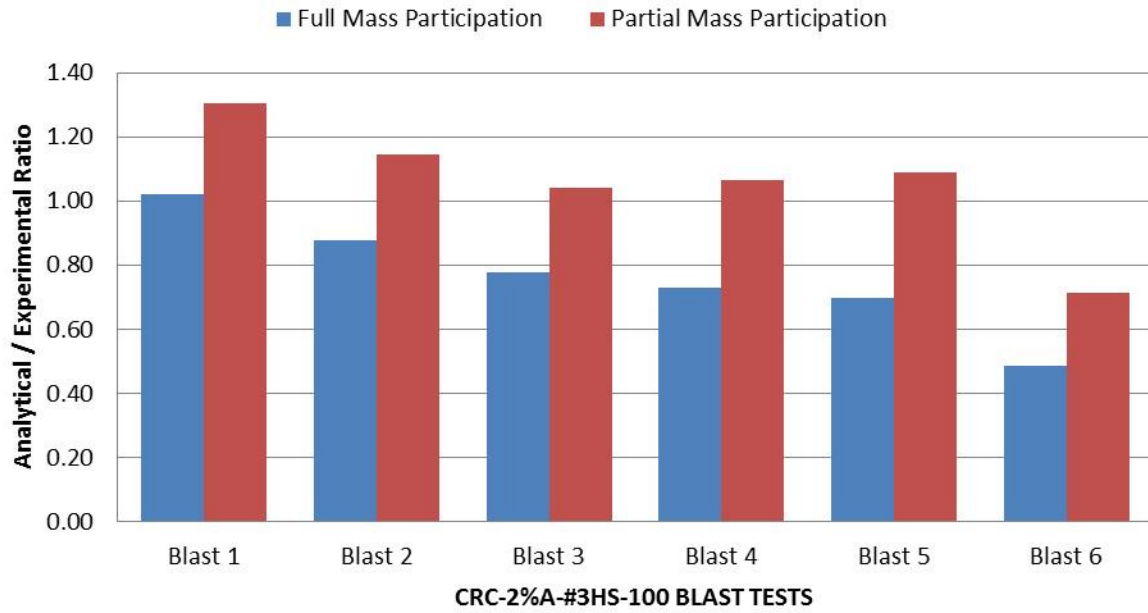


Figure 6-36 Sensitivity analysis – Mass participation, CRC-2%A-#3HS-100

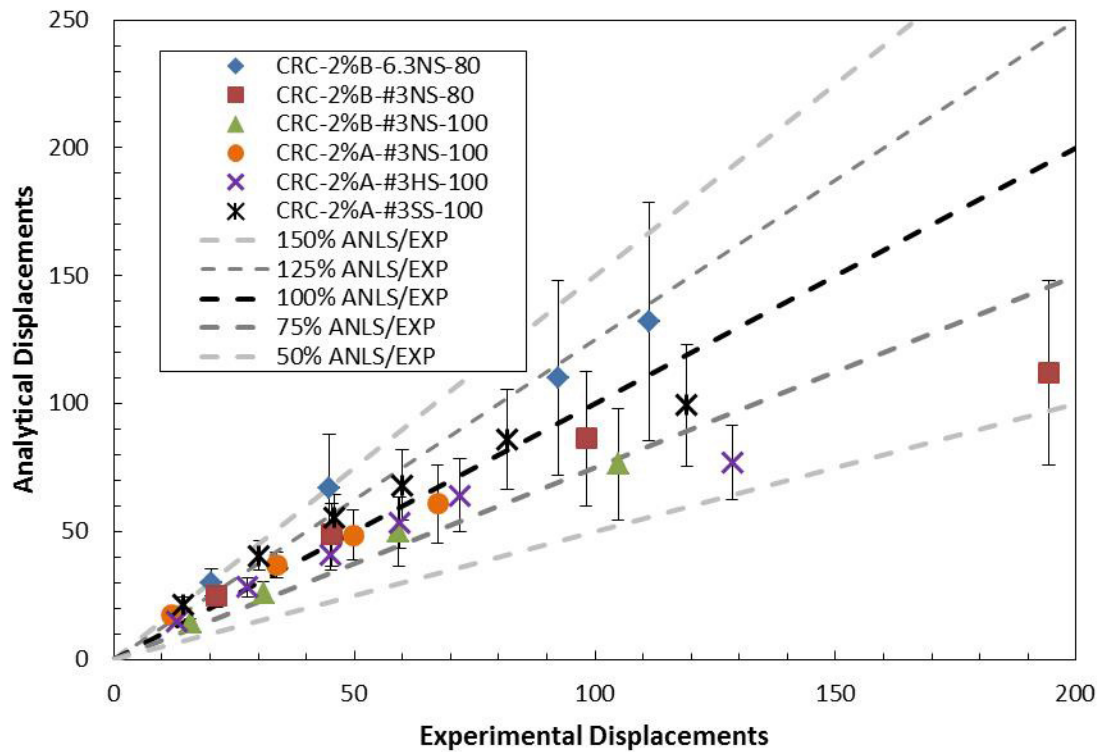


Figure 6-37 Sensitivity analysis – Mass participation, UHPFRC specimens

6.7. DYNAMIC ANALYSIS POTENTIAL SOURCES OF ERROR

The maximum analytical responses of all fourteen specimens were predicted using RCblast (Jacques 2014). Often, the deflections were underestimated meaning the blast-resistant properties of the reinforced concrete slabs were overestimated due to the software input. This section covers a few of the potential sources of error which could have influenced the final results of this dynamic analysis.

One of the main factors which could have contributed to discrepancies between the analytical and experimental displacements is the selection of a representative concrete and steel model, especially in the case of SFRC and UHPFRC. A study by Bencardino et al. (2008) noted that SFRC compressive stress-strain models are often limited to the data of their respective research, probably due to the random and varied nature of steel fibres. It should also be noted that there are only a small number of researches on the stress-strain relationship of UHPFRC. As for tension models, there are only a limited number of such models available in literature, and a few authors reported that the contribution of steel fibres to flexural toughness may be overestimated during high strain rates (Bindiganavile et al. 2002; Lok and Zhao 2004; Wang et al. 2007).

Furthermore, in the concrete models, the peak compressive strength of concrete was taken as the average of three concrete cylinder tests and was reduced by an empirical 15% to account for the size factor. When considering the variation between results from the concrete cylinder tests and the calculated average compressive strength, one could argue that the actual strength of each concrete mix could slightly differ from these measurements.

As for the application of dynamic increase factors to the stress-strain models when materials are subjected to high strain rates, the sensitivity analysis has shown that the dynamic analysis can be significantly influenced by a variation in the DIF models, especially when conservative models are used such as UFC-3-340-02 (2008). The assumption of an average strain rate for shock-tube testing could also lead to inaccuracies in strain rate dependent models. There is also a need for reliable strain-rate models for concretes incorporating fibres, particularly in the case of response under dynamic tension.

Regarding later blasts, the displacement ratios showed a substantial gap between the response obtained by the dynamic analysis and the deflection measured during the experimental program. The underestimation of the displacement for these blasts can be clarified by the strength decay of reinforced concrete slabs when subjected to repeated blasts. This kind of behaviour was also observed in a research by Kadhom et al. (2012) on reinforced concrete columns where specimens were subjected to either a single or multiple simulated blast loads; repeated loading resulted in approximately a 15% increase in column deflection during failure loads.

In addition, it was observed through the sensitivity analysis that the assumption of partial mass participation, essentially ignoring the mass of the load transfer device in the SDOF model, provided analytical results closer to the experimental maximum deflections for later blast tests.

This was associated to the delayed interaction of the load transfer device with a permanently deformed reinforced concrete panel.

Finally, manual inputs such as the selection of yield points on the moment-curvature relationship in RCblast can encourage the analytical response to diverge from the experimental deflections. It should also be noted that some assumptions such as the plastic hinge length being equal to the effective depth or the SDOF idealized behaviour assumed in the lumped inelasticity approach (Figure 6-9) may not be appropriate for UHPFRC flexural members, particularly at failure. For instance, Schumacher et al. (2009) have observed that steel fibre reinforced concrete structural components can show decreased plastic hinge length when compared to conventional reinforced concrete members. In their research, it was noted that difference of the column rotation at ultimate minus the rotation at the onset of steel yielding was reduced when steel fibres were incorporated to the concrete mix.

CHAPTER 7. CONCLUSION & RECOMMENDATIONS

7.1. CONCLUSIONS

This research investigated the performance of a series of one-way reinforced concrete slabs constructed with innovative concrete and steel material under simulated blast loads. The objective was to conduct an experimental parametric study in order to determine the various benefits of implementing high-performance materials such as SFRC, UHPFRC and high strength steel reinforcing bars to the construction of blast-resistant reinforced concrete slabs and panels. These benefits were extracted from comparisons between the dynamic behaviour of each specimen tested.

Fourteen one-way slabs with different configurations of concrete, steel reinforcement and steel fibres were designed and tested at the University of Ottawa Shock-Tube Facility under repeated simulated blast loads with gradually increasing reflected pressure and impulse until failure. Important performance criteria included the failure blast load and failure mode, the maximum and residual mid-span deflections, tensile cracking control, and secondary fragmentation mitigation. These criteria were used in different comparisons to investigate the effects of the concrete type, the effects of the steel reinforcement type, and the effects of steel fibre content and steel fibre type. From these experimental comparisons, the following conclusions can be drawn:

- The use of ultra-high performance fibre reinforced concrete (UHPFRC) in flexural members such as slabs and panels can greatly reduce the maximum and residual deflections sustained from blast loads for a structural component of similar size. It can however lead to more brittle failures such as the rupture of steel reinforcement if the rebar lacks the ductility and strength necessary for the reinforced concrete slab to fully benefit from the enhanced strength and toughness of UHPFRC.
- The implementation of steel fibre reinforced concrete (SFRC) did not provide any significant improvements for the maximum and residual deflections of the slabs. On the other hand, the incorporation of steel fibres to the concrete mix helps to reduce spalling, tensile cracking and secondary fragmentation. Additionally, higher fibre content and aspect ratios improved the control of cracking though weren't optimized in this research. This behaviour is further enhanced when the specimen is cast with UHPFRC.
- High strength steel and high ductility stainless steel reinforcement both improved significantly the response of the slabs, reducing the maximum and residual deflections as well as allowing the slabs to resist higher blast loads. Furthermore, the enhanced ductility and toughness of stainless steel reinforcement prevented the rupture of tension reinforcement during the highest blast load tested in this research, unlike the high strength steel specimen which ruptured in the UHPFRC specimen due to its brittle nature.
- The combination of both UHPFRC and high performance steel reinforcement leads to further improvements in terms of response and damage control.

A dynamic analysis was conducted in conjunction with the experimental program. This analysis was conducted using a lumped inelasticity approach in order to solve a single-degree-of-freedom equation of motion. Several concrete, steel and dynamic increase factor models were defined to model the response of the reinforced concrete slabs and were evaluated using a sensitivity analysis to obtain the most accurate responses for the specimens. A few observations were noted regarding the dynamic analysis:

- The SDOF model was capable of predicting with moderate accuracy the response of normal strength concrete specimens during the initial blast load as well as the response of high strength steel specimens during the first few repeated blast tests.
- The SFRC models had a tendency of over-predicting the blast resistance of SFRC slabs. This could be associated to the actual contribution of steel fibres to flexural toughness during high strain rate events.
- The UHPFRC models over-predicted the early displacements of most specimens. At later blasts, the displacements were significantly under-predicted. This could be linked to the current lack of proper dynamic increase factor models for UHPFRC or an incorrect assumption of the specimen's shape before and during failure.
- The response of the reinforced concrete slabs during later blasts was significantly under-predicted by the dynamic analysis due to the strength degrading after each blast test.
- Factors contributing to potential errors in the SDOF dynamic analysis include critical inputs such as the manual selection of yield points in the blast analysis software, the uncertainty of SFRC and UHPFRC stress-strain models, the lack of well-established dynamic increase factor models for fibre reinforced concrete, and inaccuracies associated with the assumptions from the lumped inelasticity approach (plastic hinge lengths, shape of specimen at failure).

7.2. RECOMMENDATIONS FOR FUTURE RESEARCH

The following recommendations are suggested for future studies in order to improve the understanding of the behaviour of SFRC and UHPFRC slabs under blast loads:

- Further investigate the behaviour of SFRC and UHPFRC panels in order to validate and potentially quantify the performance benefits observed in this research. This should include two-way slabs, walls with axial loads, and tests with different boundary conditions.
- Complete additional research regarding the use of high strength steel and high ductility stainless steel reinforcement bars in reinforced concrete slabs and other structural components.
- Evaluate the stress-strain models of SFRC and UHPFRC under high strain rates to better understand the contribution of steel fibres.
- Study the effects of high strain rates on UHPFRC to produce a dynamic increase factor model which could be used for design purposes.

- Determine the optimal steel fibre content for SFRC and UHPFRC in function of the concrete mix and properties of the steel fibres in order to fully benefit from their implementation.

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