

**PERFORMANCE OF ULTRA-HIGH PERFORMANCE FIBER REINFORCED
CONCRETE COLUMNS UNDER BLAST LOADING**

By

Frederic Dagenais

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Abstract

Recent attacks and accidental explosions have demonstrated the necessity of ensuring the blast resistance of critical buildings and infrastructure in Canada such as federal and provincial offices, military buildings and embassies. Of particular importance is the blast resistance of ground-story columns in buildings which must be properly detailed to provide the necessary strength and ductility to prevent progressive collapse. There exists a need to explore the use of innovative materials that can simultaneously improve the performance of such columns, while also allowing for a relaxation of required detailing to ease construction. Advancements in concrete material science have led to the development of ultra-high performance fiber reinforced concretes (UHPFRC) which show superior mechanical properties when compared to conventional concrete, such as increased compressive strength, tensile resistance and toughness. These enhanced properties make UHPFRC an attractive material for use in the blast design of reinforced concrete columns. This thesis presents the results of a research program examining the performance of UHPFRC columns under simulated blast loads. As part of the experimental program twelve half-scale UHPFRC specimens, six built with regular grade steel reinforcement and six built with steel high-strength steel reinforcement, are tested under blast loading using the University of Ottawa shock tube. The specimens were designed according to CSA A23.3 standard requirements for both seismic and non-seismic regions, using various fibre types, fibre amounts and longitudinal reinforcement ratios, allowing for an investigation of various design parameters on blast behaviour. The results demonstrate that the use of UHPFRC improves the blast performance of columns by reducing displacements, increasing resistance and enhancing damage tolerance. The results also indicate that fiber content, fiber properties, seismic detailing, longitudinal reinforcement ratio and longitudinal reinforcement strength are factors which can affect the behaviour and failure mode of UHPFRC columns. As part of the analytical study the response of the UHPFRC columns is predicted using dynamic inelastic analysis. The dynamic responses of the columns are predicted by generating dynamic load-deformation resistance functions for UHPFRC and conducting single-degree-of-freedom (SDOF) analysis using software RC-Blast.

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Notations

A	Area
A_g	Gross cross-sectional area of column
A_f	Cross-sectional area of fibre
A_s	Area of longitudinal reinforcing steel
d_b	Diameter of reinforcing steel
d_f	Diameter of fibre
DIF	dynamic increase factor accounting for material strength increase under high strain rates
D_{max}	Maximum mid-height column displacement
D_{res}	Residual mid-height column displacement
D_{anls}	Maximum mid-height displacement of analytical prediction
E_c	Modulus of elasticity of concrete
E_s	Modulus of elasticity of steel
f'_c	Peak static strength of concrete
f_{cc}	Confined concrete stress
f_{ct}	Concrete cracking stress
f_{cu}	Stress in the unconfined concrete at a strain equal to ϵ_{cu} .
f'_{cu}	Peak stress of unconfined concrete
f_{lf}	Steel hoops
f_1	Fibre confining pressure
f'_{le}	Effective confinement pressure
$f_{s/Du}$	Limiting value for compression reinforcing steel stress when considering buckling
f_{sh}	Steel strain-hardening stress

f_u	Static ultimate stress of reinforcing steel
f_y	Static yield stress of reinforcing steel
h	Height of column cross section
I_p	Reflected impulse over the positive phase
I	Incident impulse over the positive phase
k	Stiffness in dynamic equation of motion, rotational spring stiffness
K_{LM}	Load-mass factor for single degree of freedom conversion
K_L	Load factor for single degree of freedom conversion
K_m	Mass factor for single degree of freedom conversion
L	length of the column
l_f	Length of Fibre
L_{pl}	Plastic hinge length
$msec$	Millisecond
P	Axial load applied to column
P_o	concentric axial load capacity of column
s	Spacing of transverse steel ties
sec	Second
t	Time
u	Displacement in dynamic equation of motion
$\ddot{u}$	Acceleration in dynamic equation of motion
u_{st}	Equivalent static deflection
V_f	Amount of fibre by volume
w/c	Water-cement ratio
α	Fibre Orientation Factor
Δ	Distance of eccentric load from centre of column.
$\dot{\epsilon}$	Strain rate
ϵ_o	Peak strain

$\dot{\epsilon}_s$	Quasi-static strain rate
ϵ_{cc}	Confined concrete strain
ϵ_{ccf}	Confined steel fibre reinforced concrete strain
ϵ_{cu}	Unconfined compressive strain of concrete
ϵ'_{cu}	Peak unconfined compressive strain of concrete
ϵ'_{cuf}	Peak unconfined compressive strain of steel fibre reinforced concrete
ϵ_{axial}	Initial axial strain in column due to axial load
$\epsilon_{s/Du}$	Limiting value of strain when considering compression reinforcing steel buckling
ϵ_{sh}	Steel strain at strain-hardening
ϵ_{tot}	Total maximum strain at mid-span of column
ϵ_u	Ultimate strain
ϵ_y	Yield strain
σ_c	Concrete stress
σ_{st}	material strength under static loading
τ_{bond}	Bond shear strength between the fibre and the matrix
θ	Rotation at the supports
μ	Ductility factor - the ratio of maximum displacement to yield displacement
φ	Non-dimensional mode shape
ϕ	Curvature

Acronyms

ASTM	American Society for Testing and Materials
BLS	Blast load simulator
C.o.V	Coefficient of Variation
CBC	Canadian Broadcasting Corporation
CFRP	Carbon Fibre Reinforced Polymer
CFST	concrete filled steel tube columns
CRC	Compact Reinforced Composite
CSA A23.3	Canadian Standards Association – Design of Concrete Structures
DAC	Data Acquisition Computer
DIF	Dynamic Increase Factor
DLF	Dynamic Load Factor
EB	Externally bonded
HSC	High strength concrete
HFPB FE	high-fidelity physics-based finite element
HSLA-V	High strength low alloy vanadium
LTD	Load Transfer Device
LVDT	Linear Variable Differential Transducer
NSC	Normal strength concrete
PFRC	Polymeric Fibre Reinforced Concrete
RC	Reinforced Concrete
RPC	Reactive Powder Concrete
rpm	Revolutions per minute
RUHPFRC	Reinforced Ultra high performance fibre reinforced concrete
SCC	Self Consolidating Concrete

SCFRC	Self Consolidating Fibre Reinforced Concrete
SDOF	Single Degree of Freedom
SFRC	Steel Fibre Reinforced Concrete
SRP	Steel Reinforced Polymer
TNT	Tri-Nitro Toluene – an explosive composition
UHPRC	Ultra high performance fibre reinforced concrete
VR	Vanadium reinforcement
WCM	Winfrith Concrete Model

1 Introduction

1.1 General

Incidents such as the Commonwealth avenue collapse in Boston and the Bailey's crossroads collapse in Virginia have shown that failure of primary members in reinforced concrete structures can lead to devastating and sudden structural failures (Delatte, 2009). Other events, such as the Oklahoma City bombing have also brought attention to the vulnerability of reinforced concrete structures against blast hazards. In particular, these events have highlighted the potential of blast-induced progressive collapse caused by the failure of ground-story columns. To prevent such sudden failures, ground story columns must be properly detailed to provide sufficient strength and ductility to prevent failure under extreme blast pressures. This detailing can lead to heavily congested members that are difficult to construct; there exists a need to explore the use of innovative materials that can simultaneously improve the blast performance of such columns, while also allowing for a relaxation of required detailing leading to ease of construction.

Recent advancements in concrete material science have led to the development of ultra-high performance fiber reinforced concretes (UHPFRC) which show superior mechanical properties when compared to conventional concrete. Dense particle packing in UHPFRC allows for superior compressive strength, while the provision of steel fibers allows for enhanced tensile resistance and toughness leading to increased damage tolerance (Li, 2002). These enhanced properties make UHPFRC an attractive material for use in the blast design of reinforced concrete structures (Banthia, 2008). In columns, UHPFRC could possibly be used to reduce required detailing leading to improved constructability without compromising performance. Figure 1.1 shows a comparison of the compressive stress-strain response of UHPFRC and conventional concrete and demonstrates the significantly increased compressive strength and ductility that can be obtained with UHPFRC when compared to conventional concrete. Such properties could be used to improve the resistance of structural members under blast loading.

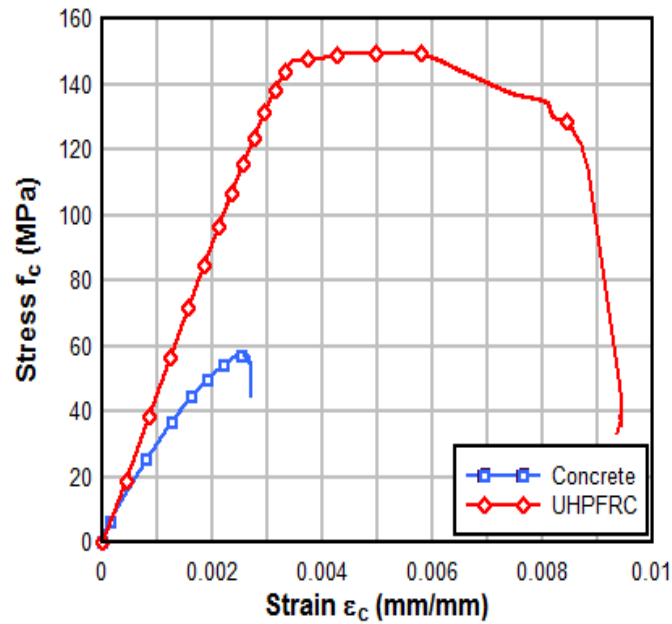


Figure 1.1 Stress-strain relationship of UHPFRC and conventional concrete in compression

Previous limited research at the University of Ottawa has shown that the use of UHPFRC can enhance the blast performance of reinforced concrete columns by better controlling displacements, increasing blast resistance and enhancing and improving damage tolerance (Burrell, 2012). There is a need for further research data related to the behaviour of UHPFRC columns. There is also a need to explore the effect of various design parameters that can influence the blast behaviour of such columns (e.g. choice of fiber properties, fiber content, reinforcement detailing, etc.)

The research has also shown that UHPFRC columns are susceptible to failure due to rupture of tensile steel reinforcement. High-strength steel, shown in Figure 1.2, can have tensile strength up to two or three times compared to regular grade reinforcement; combining the high tensile strength of high-performance steel with the high compressive strength of UHPFRC can potentially enhance column performance and lead to greater blast resistance.

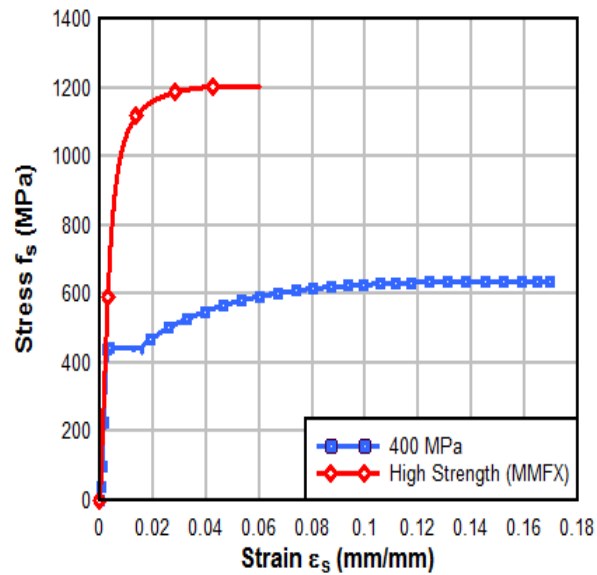


Figure 1.2 Stress-strain relationship of high-strength and regular grade steel

1.2 Research Objectives

This research program was undertaken to study the benefits associated with the use of UHPFRC and high-strength steel in columns subjected to blast loads. More precisely, the experimental program aims at studying the influence of various design parameters that can influence the performance of such columns:

- The effect of concrete type (UHPFRC vs. SCC)
- The effect of fibre properties
- The effect of fibre content
- The effect of seismic detailing
- The effect reinforcement ratio
- The effect of steel type (MMFX)

The analytical investigation aims at examining the suitability of using dynamic inelastic single-degree-of-freedom (SDOF) analysis to predict the blast response of the columns tested in the study. The details of the experimental and analytical programs are summarized below.

1.2.1 Experimental Program

As part of the experimental program twelve half-scale columns constructed with compact reinforced composite (CRC), a proprietary UHPFRC, are tested under simulated blast loading using the University of Ottawa shock tube. The columns are subdivided into two series; Series 1 includes six UHPFRC specimens constructed with regular grade longitudinal steel reinforcement, while Series 2 includes six UHPFRC specimens constructed with high-strength longitudinal steel reinforcement (MMFX). Design parameters considered in this study include the effect of: fiber properties, fiber content, transverse reinforcement spacing, longitudinal reinforcement ratio and longitudinal reinforcement strength.

1.2.2 Analytical Program

The analytical program involves numerical analysis aimed at predicting the response of the UHPFRC columns tested in the experimental program. This is achieved by generating dynamic load-deformation resistance functions for the UHPFRC columns and using single degree-of-freedom dynamic analysis software, RC Blast.

1.3 Thesis Organisation

The thesis consists of seven sections:

- Section 1 introduces the thesis and the research program objectives;
- Section 2 presents a literature review on the thesis subject matter including a review of previous research related to the blast behaviour of RC columns, UHPFRC slabs, UHPFRC columns and reinforced concrete elements built with high-strength steel reinforcement;
- Section 3 summarizes the details of the experimental program, including: specimen details, material properties, specimen construction, instrumentation, test setup and test procedure;
- Section 4 presents the experimental results for the twelve UHPFRC columns tested in the experimental program;
- Section 5 discusses and compares the experimental data to study the influence of various parameters on the blast response of the tested specimens;

- Section 6 presents an analytical study aimed at predicting the dynamic response of UHPFRC columns under blast loads. The section compares analytical results to experimental results obtained during this research program;
- Section 7 presents concluding remarks regarding the research program and provides some recommendations for future research.

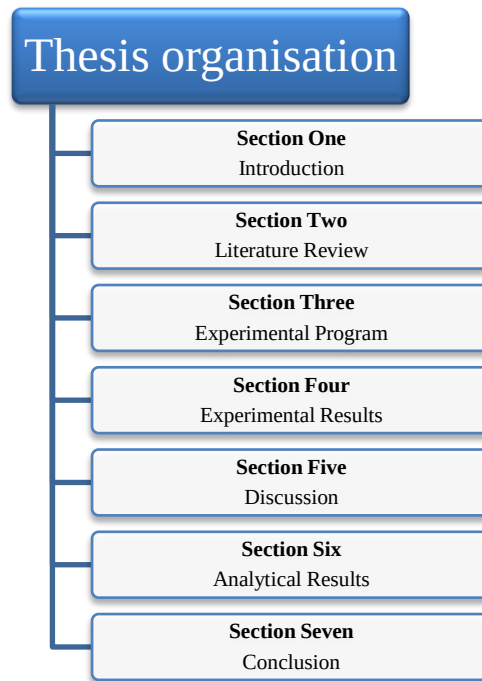


Figure 1.3 Thesis Organisation

2 Literature Review

This Section of the thesis presents a literature review related to the thesis topic. In order to put the current project into context, the section begins with historical examples of explosion disasters, followed by a review of existing research related to the blast load behaviour of conventional reinforced concrete (RC) columns, with emphasis on studies which have investigated the effect of seismic detailing on blast performance. The third part of the section summarizes existing research on UHPFRC under extreme loads; beginning with a short review of impact studies, followed by a review of existing research related to the blast performance of UHPFRC slabs and columns. Six of the specimens in this research project are constructed with high-strength steel reinforcement; therefore the section ends with a review of research that has investigated the effect of high strength steel reinforcement on the blast performance of reinforced concrete structural components.

2.1 Historical Examples of Explosion disasters

2.1.1 Barajas International Airport, Madrid, Spain, December 30th, 2006

This malicious terror attack involved approximately 500kg of explosive set off in the concrete parking structure of Terminal 4 of the Madrid-Barajas International Airport. The incident involved a van parked within the concrete structure containing ammonium nitrate and hexogen mixed explosive. As shown in Figure 2.1, the explosion nearly demolished all five floors creating around 40 tonnes of debris. Two Ecuadorian citizens were killed while 36 people were subjected to injuries. A total cost of 30 million Euros was estimated for the damages from this event (Santamaria, 2008).



Figure 2.1 Effects of the 500kg explosive on the RC parking structure of Terminal 4

(Source: http://en.wikipedia.org/wiki/2006_Madrid-Barajas_Airport_bombing)

2.1.2 Arndale Shopping Centre, Manchester, United Kingdom, June 15th, 1996

This terrorist attack happened the morning of Saturday June 15th, 1996. The explosion was caused by explosives in a cargo truck which was parked outside of the Arndale Shopping Centre in the city of Manchester, UK. The truck contained roughly 1500kg of explosive combining semtex, ammonium nitrate fertilizer and a military grade plastic explosive. The powerful shockwave was said to be felt up to 8km away while windows within a range of 500m were shattered. The blast from the explosion was strong enough to generate major structural damage to a dozen buildings including two multi-story parking garages, while 6 buildings required full demolition. Major damage to roads occurred in proximity of the explosion, which also affected the city's infrastructure. The direct costs from this event were estimated to be £100 million (GBP). Warnings to the authorities led to mass evacuation which prevented any casualties from happening (Williams et al., 2000).

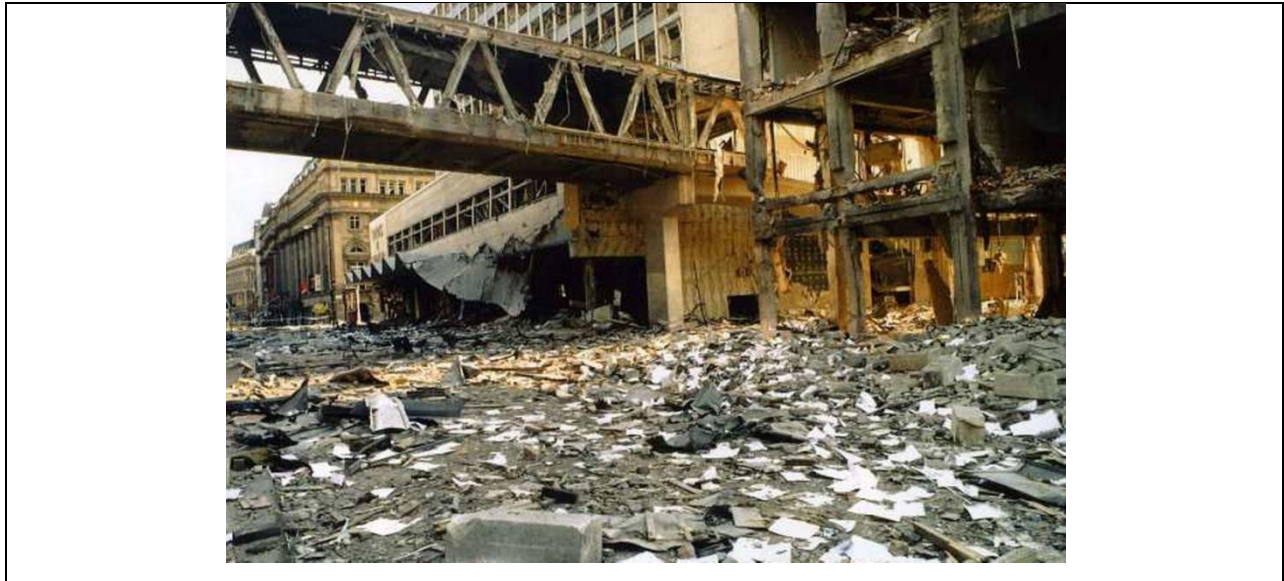


Figure 2.2 Effect of 1500kg explosive, Arndalte shopping center

(Source: <http://huddled.co.uk/18-years-ira-manchester-bomb/>)

2.1.3 Alfred P. Murrah Building, Oklahoma City, April 19th, 1995

The failure of the nine-story Murrah building provides one of the most clear examples of the vulnerability of reinforced concrete buildings to progressive collapse from explosions. The failure of the building was caused by the detonation of 1800 kg of equivalent TNT explosive loaded in a transport truck and placed approximately 4m in front of a main support column of the building on April 19th, 1995. The event resulted in the death of 168 people, with another 680 injuries, and more than \$652 million (USD) in damage. Post-blast surveys revealed that the support column was fully destroyed by the proximity of the blast resulting in the collapse of four above bays due to lack of alternative load path. Twenty bays in the footprint of the structure collapsed while ten of these occurred over the full height of the building due to progressive collapse (see Figure 2.3). It is noted that 90% of the casualties were directly associated with the structure's progressive collapse (Schmidt webinar, 2010). This incident led to development of new structural design guidelines to resist progressive collapse (Osteraas, 2006).



Figure 2.3 Aftermath of the Murrah Building bombing

(Source: http://en.wikipedia.org/wiki/Oklahoma_City_bombing#cite_note-UnderstandingTerror106-4)

2.2 Previous Research on Blast Behaviour of RC Columns

Previous research has demonstrated that closely-spaced and well-detailed transverse reinforcement improves the ductility of RC columns under seismic loads. More recently, researchers have examined the effectiveness of seismic detailing in columns subjected to blast loads. Li et al. (2012) tested a series of reinforced concrete columns using a hydraulic-blast simulator, which applies lateral and vertical loads using a series of horizontal actuators and vertical actuators, respectively. The columns with seismic detailing were found to have improved blast resistance, with reduced displacements and increased damage tolerance, when compared to companion columns with conventional detailing. Post-blast axial tests also showed that seismic detailing allowed for increased residual axial capacity.

Fujikura and Bruneau (2011) conducted live explosive tests on quarter-scale RC bridge columns with seismic and non-ductile detailing. The seismically-detailed columns did not show ductile behaviour under blast loads and suffered brittle failure due to direct shear at the base. It is noted that the columns in this test program were subjected to close-in blast loads and were tested without the application of axial load. In a related study, Fujikura et al. (2008) tested the blast

performance of a "multi-hazard" bridge pier design concept which consisted of a multi-column pier bent with concrete-filled circular steel tubes (CFST). The CFST system was found to provide ductile response under close-in blasts.

Williamson et al. (2011) investigated the blast response of ten half-scale RC bridge columns tested under close-in blasts. Parameters studied included effect of standoff distance, column aspect-ratio, splice location, and transverse reinforcement detailing. Increasing the transverse reinforcement ratio was found to improve blast performance, with post-damage evaluations showing blast designs and seismic designs to perform better than non-seismic designs. However, as reported by Fujikura and Bruneau (2011), the failure mode under close-in blasts was found to be direct shear at the base. Based on this study, guidelines for blast-resistant design of bridge columns were provided in NCHRP report 645 (2010).

Research has shown that retrofits with FRP composites can significantly improve the strength and ductility of seismically-deficient RC columns. More recently, the effectiveness of using FRP retrofit to improve blast behavior of columns has been demonstrated numerically and through experimental research (Crawford, 2011). Other retrofitting techniques considered in the literature include the use of steel-jackets (Morrill et al., 2001) and SRP wraps (Carriere et al., 2009).

In summary, important experimental research has been conducted to study the blast performance of conventional reinforced concrete columns. Data in the literature is conflicting on the effectiveness of seismic detailing, particularly in the case of columns subjected to intense close-in blasts. Other approaches for mitigating blast effects reported in the literature include the use of FRP retrofits, SRP wraps, and steel-jacketing. Table 2.1 lists some of the research done on blast behaviour of RC columns.

Table 2.1 Published research on blast behaviour of RC columns

Authors	Type of testing	Main conclusions
Li et al. (2012)	Blast loading of seismic and non-seismic RC columns.	- Higher axial load increase stiffness resulting in smaller deflected profile. - Higher transverse reinforcement ratio provided additional shear strength resulting in lower lateral residual deflection.
Fujikura and Bruneau (2011), (2012)	- Blast testing conducted on ¼ scale ductile RC columns and steel jacketed retrofitted RC column. - Analytical investigation on blast-response and behaviour of bridge piers having circular-shaped concrete-filled steel tube columns (CFST).	- Ductile seismically designed RC column and steel jacket retrofitted RC column show no ductility under blast loading. - Failure of columns by direct shear at their base rather than by flexural yielding. - Seismic and steel jacketing codes known to be effective in seismic performance show ineffectiveness in blast loading. - Structural response using SDOF dynamic analysis was sensitive to load-mass factors. - Rayleigh damping with fibre based model can significantly reduce structural response under blast loading due to high frequency mode effects.
Williamson et al. (2011)	Blast loaded half scale bridge columns	Higher performance of columns with spirals or ties with long hooks under blast loading
NCHRP report 645 (2010)	- Small-scale blast test on square and round non-responding columns - Close-in blast tests on half-scaled RC columns	- Decrease design threat by providing sufficient standoff distance from columns. - Circular columns are more effective to decrease blast pressures and impulse when compared to square and rectangular shape. - Continuous spiral reinforcement performed better than discrete hoops for small standoff treats.
Crawford (2011)	Blast effect on RC columns using FRP	- FRP offers o remarkable capability to enhance the blast resistance of existing RC columns - HFPB FE models are necessary to capture some forms of blast response
Morrill et al. (2001)	Analytical retrofitting techniques using steel jacketing and FRP wraps	Simple windows-based codes were completed to assess the resistance of existing RC columns and walls to blast.
Carriere et al. (2009)	- Scaled RC members tested under blast with SRF wraps - Numerical models created in AUTODYN for testing validation	- SRF reduces concrete crushing and spalling at plastic hinges - SRP can withstand blast from 50kg of explosive at 2m without debonding. - Quantitative failure can be predicted using appropriate hydrocode.

2.3 Previous Research on Impact Behaviour of UHPFRC

There has been important research demonstrating that UHPFRC exhibits excellent performance under high-strain rates, mostly from impact testing. Studies have been conducted using drop-weight and drop hammer testing by Parent et al., (2007), Millard et al. (2010), and Habel & Gauvreau (2008). The strain-rate behaviour of UHPFRC in direct tension has also been studied by Wile et al. (2012). Testing under higher strain rates using Split-Hopkin Pressure Bar (SHPB) has been conducted by Rong et al. (2010), Tai (2010), and others. Overall, this research has demonstrated the superior performance of UHPFRC under extreme dynamic loading when compared to conventional concrete.

Limited research exists on the behaviour of Compact Reinforced Composite (CRC; the proprietary UHPFRC used in the current project) at higher strain-rates. The material's behaviour under impact loads was studied by Bindiganavile et al. (2002). In the test program, quasi-static and impact tests were conducted on standard flexural beams using a drop-weight testing machine, with drop-heights of 250, 500, 750 and 1000 mm. The authors reported an increase in the peak load and toughness, which represents energy absorbed, for CRC with increasing drop-height, in contrast to results from tests on companion specimens made of steel fibre reinforced concrete (SFRC). In particular the authors noted that while SFRC became brittle with increase in drop-height, CRC showed brittle response only at very large drop-height (1000 mm in this study), indicating that brittle behaviour, a characteristic of high stress-rate response manifests itself only at larger stress rates for CRC (Bindiganavile et al., 2002). Bindiganavile et al.(2002) concluded that while brittleness would be expected for a high-strength concrete matrix under high impact loading, post-peak energy dissipation becomes possible in CRC due to the addition of steel fibres. The stress-rate sensitivity of CRC in flexure was also compared to that of traditional concrete and SFRC by plotting the dynamic increase factor (DIF) as a function of stress-rate, with the results indicating reduced stress-rate sensitivity for CRC.

2.4 Previous Research on Blast Behaviour of UHPFRC slabs

2.4.1 Ductal - An ultra-high performance material for resistance to blasts and impacts (Cavill et al., 2006).

Cavill et al. (2006) performed a series of explosive tests on seven panels built with reactive powder concrete (RPC). RPC is a type of UHPFRC consisting of cement, sand, silica fume, silica flour, superplastizer, water, and high strength steel fibres. The proprietary version of this material is marketed by Lafarge under the *Ductal* brand name. Almost self-placing, this material has a compressive strength of 160-200 MPa and a flexural strength of 30-40 MPa.

The RPC panels in the Cavill et al. (2006) study were tested in May 2004, at Woomera, South Australia. The seven panels were tested under two separate blasts of six tonnes of TNT equivalent explosive at standoff distances of 30m, 40 m, and 50m, resulting in reflective blast pressures of 2000 kPa, 800 kPa, and 400kPa, for these distances respectively. For reference, a conventional reinforced concrete panel was also tested at 40m from the blast. Each panel measured 2m by 1m with varying thicknesses of 50mm, 75mm and 100 mm. While details are confidential, five of the seven panels contained identical arrangement of high strength prestressing strands with a tensile strength of 1840 MPa. The two other panels were unreinforced. Figure 2.4 shows the panels prior to the blast test.



Figure 2.4 Panels ready for blast

[Adapted from Cavil et al. (2006)]

Post-blast observations of the panels indicated that the panel built using conventional reinforced concrete had sustained severe damage with extensive fragmentation and spalling. In comparison,

fragmentation was eliminated in the RPC panels, and only the RPC panels tested at the closest standoff fractured. The lack of fragmentation, even at fracture, shows the great benefit of using RPC over conventional concrete since concrete fragments can pose great threats to people and the infrastructure itself. Further tests were performed on 100mm panels. These panels successfully resisted explosions from close range, projectile impacts from ballistic tests, and impacts caused by blast produced fragments using fragments simulated projectile tests.

2.4.2 Blast Testing of Ultra-High Performance Fibre and FRP-Retrofitted Concrete Slabs (Wu et al., 2009).

Wu et al. (2009) performed a series of tests to investigate the blast resistance of ultra-high performance fiber reinforced concrete (UHPFRC) slabs with and without reinforcement, and slabs retrofitted with fibre reinforced polymer (FRP) plates (Wu et al., 2009). In total, six 2000mm x 1000mm x 100mm slabs were tested including two control slabs built using normal reinforced concrete (NRC). These control slabs were reinforced with 12mm diameter mesh on both compression and tension face, spaced at 100mm on-center in the major bending plane and 200mm on-center in the minor plane. Two other normal reinforced concrete slabs were built and retrofitted with externally bonded FRP on the tension face, with two 1.4 mm thick CFRP plates used to achieve a total CFRP thickness of 2.8mm. The last two slabs were built using UHPFRC, one without reinforcement and one with steel reinforcement. The reinforced UHPFRC slab was detailed with identical reinforcement found in the NRC slab. The specimens were mounted on a steel frame which provided partial fixity, resulting in a slab effective length of 1800mm, and the panels were exposed to 1-20 kg of equivalent TNT at standoffs of 1-3 m (Wu et al., 2009). The results indicated that the UHPFRC slab suffered less damage over the NRC slabs when tested under comparable blast loads. Moreover, the reinforced UHPFRC slab out-performed every other slab in this research study, showing an ability to sustain larger blast loads when compared to companion panels. Figure 2.5 shows the slab setup prior to the blast test.



Figure 2.5 Slab with test fixture

[Adapted from Wu et al. (2009)]

2.4.3 Blast Tests of Fibre-Reinforced Concrete Panels (Barnett et al., 2009)

Barnett et al. (2009) tested a series of four ultra-high-performance fibre reinforced concrete panels measuring 3500mm x 1300mm x 100mm subjected to 100kg of TNT-equivalent explosive loading. The primary variables investigated in this research program include the type and quantity of fibre reinforcement, the use of conventional steel reinforcement, and the stand-off distance of the explosive charge. The first two panels were replicates and contained conventional steel reinforcement and 2% of 13 mm straight steel fibres by volume of concrete. These panels were tested at stand-offs of 9m and 7m respectively. The panels sustained large maximum displacements of 110 mm and 210 mm, but showed an ability to recover with residual displacements of 20 mm and 50 mm, respectively. The remaining two panels contained no conventional steel reinforcement steel but were reinforced with steel fibres at contents of 2% and 4%, respectively. These specimens were tested at the same stand-off distance of 12m and suffered large permanent displacements of 180 mm and 90 mm, respectively, with horizontal cracking across their full width. Nonetheless, the specimens showed no major fragmentation and remained standing after testing. Barnett et al. (2009) concluded that UHPFRC was shown to have properties suitable for resisting blast loading and could be utilized to protect occupants and buildings against explosions. The authors also noted that detailing of the panels had an important effect on performance under blast loads and should be adjusted based on the specific application. Figure 2.6 shows the panel after the blast test.



Figure 2.6 Panel after testing

[Adapted from Barnett et al. (2009)]

2.4.4 Experimental Investigation and Multiscale Modeling of Ultra-High-Performance Concrete Panels Subject to Blast Loading (Ellis et al., 2013).

Ellis et al. (2013) conducted an experimental program on UHPFRC panels to validate a multiscale model that accounts for structure and phenomena at two length scales: "structural" length scale and "multiple fibre" length scale. The experimental program consisted of testing four 1626 mm x 864 mm x 51mm UHPFRC panels without steel reinforcement under blast loading. The panels contained 2% volume fraction of straight 14mm long fibres. These panels were tested in a Blast Load Simulator (BLS) located in Vicksburg, MS. The specimens were subjected to shockwaves with specific impulses which varied between 0.77 and 2.05 MPa-ms. It was estimated that the panels failed at impulse loads which varied between 0.97 and 1.47 MPa-ms. During failure, all four specimens produced fragmentation and left protruding fibres along the fractures surface. Ellis et al. (2013) used the evidence of this physical experiment to create a base for a multiscale model using multiple fiber and structural length scales. Both of these scales were linked to four quantities: mean tensile strength, standard deviation of the tensile strength, mean dissipated energy density, and standard deviation of the dissipated energy density. The experimental results validated the multiscale model by using critical specific impulse, fracture patterns and mid-span displacements. With the help of the model, Ellis et al. (2013) concluded that fibre geometry, packing, and volume fractions are all factors that critically influence the

resistance of UHPFRC panels under blast loading. Figure 2.7 shows the panel throughout the blast test.

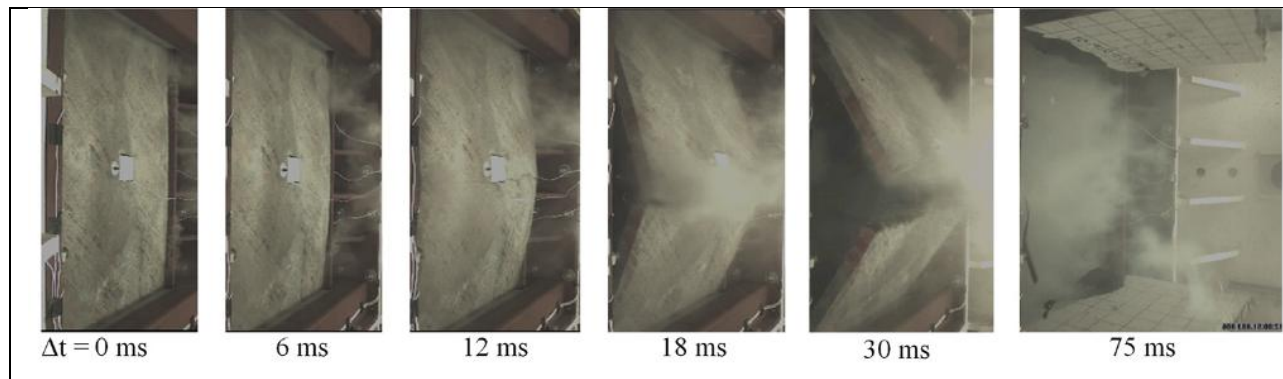


Figure 2.7 Panel at different time intervals during blast

[Adapted from Ellis et al. (2013)]

2.4.5 Blast-Resistant Characteristics of Ultra-High Strength Concrete and Reactive Powder Concrete (Yi et al., 2012)

Yi et al. (2012) carried out some close-in live explosive tests on three two-way panels constructed with normal strength concrete (NSC), high strength concrete (HSC) and a proprietary UHPFRC (RPC). Even though HSC has shown to have compressive strength over 100 MPa, its everyday use is not always favourable due to effectiveness to cost ratio. HSC concrete was also questioned due to safety hazard related to its possible ultra-brittle failure behaviour. The three 1 m x 1m x 150 mm panels were clamped on all four sides and tested under charge weights of 4 - 16 kg of ANFO at a standoff of 1.5m. Yi et al. (2012) conclude during their experimental research that the compressive strength, split-tensile strength, elastic modulus and Poisson's ratio values were all higher for HSC and RPC over the NSC. When compared to the companion specimens made of HSC and NSC, the UHPFRC panel showed improved blast performance, reduced maximum and residual displacements, and controlled cracking with reduced spalling. Finally, the rebar and short steel fibers used during this experimental research was sufficient to negate the brittle material characteristics and provide sufficient ductility, energy absorption and crack controlling capacity.

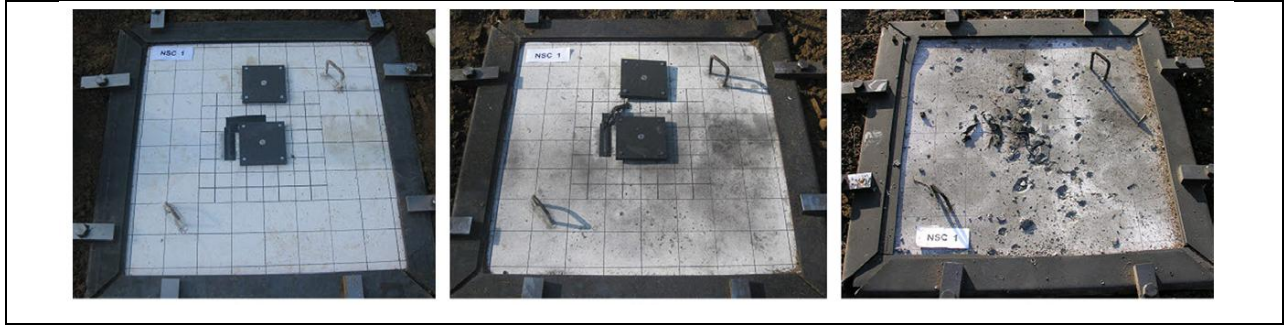


Figure 2.8 From left to right, slab before test, after 4.08 kg TNT blast, after 15.88 kg TNT blast

[Adapted from Yi et al. (2012)]

2.5 Previous Research on the Blast Behaviour of UHPFRC Columns

2.5.1 Response of Normal-Strength and Ultra-High-Performance Fiber-Reinforced Concrete Columns to Idealized Blast Loads (Astarioglu and Krauthammer, 2014).

The potential of using UHPFRC to improve the blast resistance of columns has been studied numerically by Astarioglu and Krauthammer (2014). Using SDOF analysis, the effects of concrete type, boundary conditions, and axial loads were investigated. The response of the UHPFRC columns was compared to that of companion columns designed with normal-strength concrete (NSC). As part of the study the column performance was studied numerically against a series of four different blast scenarios. Astarioglu and Krauthammer analyzed a total of 16 different cases per concrete material and varied parameters such as boundary conditions and axial load levels. In comparison to NSC, the UHPFRC columns showed reductions in peak displacements in the order of 27% to 30% under equivalent blast loads for the simple and fixed boundary condition cases, respectively. The true advantage of UHPFRC was shown by comparing the threshold curves for both concretes using pressure-impulse diagrams. This analysis showed that the UHPFRC columns could sustain over four times the impulse that would generate failure in companion NSC columns. In terms of the effect of axial loads, the authors of this study concluded that higher compressive loads result in smaller deformations and increased resistance in NSC columns subjected to the same transverse load. This behaviour was expected due to P-I diagrams showing moment capacity increase up to the axial load corresponding to the balanced condition. However, this strength increase resulted in lower ductility, making the columns more susceptible to transverse loads at high axial load levels. UHPFRC columns showed very similar behaviour under the effect of varying axial loads.

2.5.2 Performance of Steel Fibre Reinforced Concrete Columns Under Shock Tube Induced Shock Wave Loading (Burrell, 2012).

Burrell (2012) tested a total of 13 half-scaled steel fibre reinforced concrete columns under blast loading in the shock tube at the University of Ottawa. Eight of the specimens were built using normal strength steel fibre reinforced concrete (SFRC) while the other five specimens were built using compact reinforced composite (CRC), a proprietary ultra high performance fibre reinforced concrete (UHPFRC). The columns were designed according to CSA A23.3 standard requirements for both seismic and non-seismic regions and were constructed using different fibre types and fibre contents. Each column was tested under gradually increasing blast pressures to examine the behaviour of the columns at yield and failure. Parameters considered in the research program included the effect of seismic detailing, fibre content, fibre type, and type of concrete. The following conclusions were drawn from Burrell's study:

- The addition of steel fibres was found to enhance the blast resistance of concrete columns by reducing maximum and residual displacements. Results were directly related to the amount of fibres added to the mix;
- The use of seismic detailing was shown to improve column blast performance. Reduced transverse reinforcement was shown to lower maximum and residual displacements while also preventing compression reinforcement bar buckling that occurred in non-seismically designed columns;
- The use of steel fibres in non-seismic columns was found to greatly improve blast performance when comparing to the control specimen without fibers. However, the fibres were not able to prevent compression bar buckling. It was recommended that the combined use of seismic detailing and fibres could prevent bar buckling failures;
- The use of fibres was found to greatly reduce the amount of secondary blast fragments that can be harmful to both people and the structure. The results showed that the amount of fragmentation is directly proportioned to the amount of fibres added to the mix.
- The use of UHPFRC in columns was found to greatly reduce maximum and residual displacements when compared to companion columns subjected to similar loads. Burrell noted that results due to the high compressive and high tensile strength of the material. Despite the improvements in performance the failure mode of the UHPFRC columns was

brittle and resulted from the rupture of tension steel reinforcement which was attributed to the high compressive strength of UHPFRC. Burrell (2012) recommended further research on UHPFRC columns containing larger reinforcement ratio or higher reinforcement grade.

The current research program builds on the results obtained in the Burrell (2012) study and further examines the effect of various design parameters on the blast performance of UHPFRC columns, namely the effect of: fiber properties, longitudinal reinforcement ratio and longitudinal reinforcement strength.

2.6 Previous Research on Blast Behaviour of Concrete Elements Reinforced with High-Strength Steel

2.6.1 Experimental and Finite Element Analysis of Doubly Reinforced Concrete Slabs Subjected to Blast Loads (Thiagarajan et al., 2014).

Thiagarajan et al. (2014) examined the effect of concrete strength and steel reinforcement strength on the blast response of reinforced concrete slabs. The slabs were constructed with either normal-strength or high-strength concrete and were doubly reinforced with either high strength low alloy vanadium (HSLA-V) reinforcement (VR) or conventional steel rebar (NR). The slabs were tested experimentally using a shock tube and then analyzed numerically using Finite element analysis (FEM). In total, four slab combinations were considered; High Strength Concrete (HSC) with HSLA-V steel reinforcement bars (HSC-VR), High Strength Concrete with Conventional Steel Rebar (HSC-NR), Normal Strength Concrete with HSLA-V steel reinforcement bars (NSC-VR), and Normal Strength Concrete with Conventional Steel Rebar (NSC-NR). Numerical analysis of the tested slabs was conducted with LS-DYNA using two models; Winfrith Concrete Model (WCM) and Concrete Model Damage Release 3 (CMD3). Thiagarajan et al. (2014) concluded that:

- The combination of high strength materials significantly improved the blast performance of the slabs. The combined use of high-strength steel and high-strength concrete resulted in reductions in maximum deflections when compared to companion slabs constructed with normal-strength concrete and steel reinforcement;

- High strength steel was more effective when combined with normal strength concrete over high strength concrete. Given the limited number of specimens in this study the authors noted that this conclusion was more of an observation rather than a conclusion; The use of FEM analysis was capable of predicting the response of the tested slabs with reasonable accuracy. Both analytical models used during this research program showed an ability to predict peak deflections and deflection time-histories for all simulations. The WCM model was found more suitable to simulate the propagation of concrete cracks on the slab surface while not being sensitive to mesh sizes.

3 Experimental Program

3.1 General

This research program was undertaken to study the blast load behaviour of ultra-high performance fibre reinforced concrete (UHPFRC) columns. A total of twelve column specimens are tested under simulated blast loading, and parameters considered in this study include the effect of concrete type, fiber content, fiber properties, seismic detailing, longitudinal reinforcement ratio and longitudinal reinforcement strength. This section summarizes the details of the experimental program and provides information regarding the test matrix, material properties, instrumentation, test setup and test procedure.

3.2 Description of Test Specimens

As part of this study a total of twelve UHPFRC columns were tested under simulated blast loads using the University of Ottawa shock tube. The specimens were constructed using compact reinforced composite (CRC), a proprietary UHPFRC produced by CRC Technology, Denmark. The specimens are subdivided into two series; Series 1 included six UHPFRC columns reinforced with regular grade steel, while Series 2 included a further six UHPFRC columns constructed with high-strength steel. Table 3.1 summarizes the details of the specimens and provides information regarding fiber content (which ranged from 2-3% by volume of concrete), fiber type (B, C or D - see Section 3.3.2), transverse steel reinforcement spacing (75 mm or 38 mm, reflecting non-seismic and seismic detailing, respectively), longitudinal steel reinforcement (4 - 10M, 4 - 15M or 4 - #3 bars) and longitudinal steel reinforcement strength (regular grade or high-strength). The nomenclature reflects the test variables as follows:

Concrete Type - Fiber % and Type - Tie Spacing- Rebar Size

For example, specimen CRC-2%B-75-15M uses CRC concrete with 2% of fibre type B, has tie spacing of 75 mm, and is reinforced with 4 - 15M bars. For series 1, the lack of *REBAR SIZE* in the nomenclature indicates the use 10M rebar in these specimens (e.g. in CRC-2%B-75), while the "MMFX" which appears in the nomenclature of the Series 2 columns indicates the use of 4-#3 high-strength steel bars in these columns.

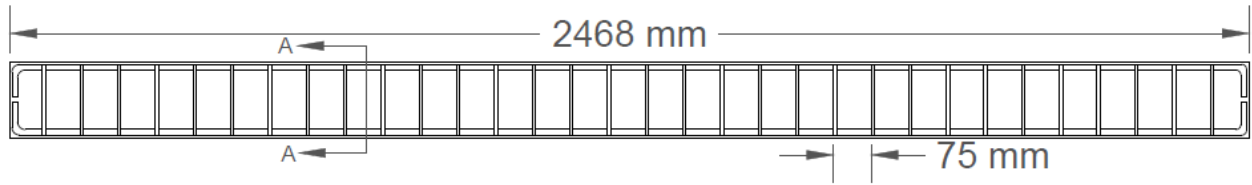
As illustrated in Figure 3.1 Design of non-seismic, seismic and cross-section detailing, all columns had cross-sectional dimensions of 152 mm x 152 mm (6 in. x 6 in.) and a total height of 2468 mm (8 ft). As noted above the longitudinal reinforcement in the specimens varied. The first four columns in Series 1 had 4 - 10M bars ($d_b = 11.3$ mm and $A_s = 100$ mm²), with the two remaining specimens were reinforced with 4 - 15M bars ($d_b = 16$ mm and $A_s = 200$ mm²) to examine the effect of reinforcement ratio on blast performance. All Series 2 columns were reinforced with 4 - #3 American size ($d_b = 9.5$ mm and $A_s = 71$ mm²) high-strength MMFX bars. The rebar in all specimens had 90° hooks extending 75 mm at each extremity to ensure development of the reinforcement. The transverse reinforcement 6.3 mm diameter closed ties made of non-deformed steel wire. The ties had 135° hook extensions, and all columns had a 5 mm clear cover. The non-seismic columns had tie spacing of 75 mm (s), corresponding to moderately ductile detailing as specified in the CSA-A23.3-04 design standard, while the "seismic" columns had 38 mm tie spacing, meeting the ductile requirements of the same standard (CSA A23.3-14, 2014).

The results from this study will be compared to a companion series of column tests conducted by Burrell (2012). This companion set includes two control columns constructed with normal-strength (50 MPa) self-consolidating concrete (SCC), and two UHPFRC columns constructed with a different fiber type.

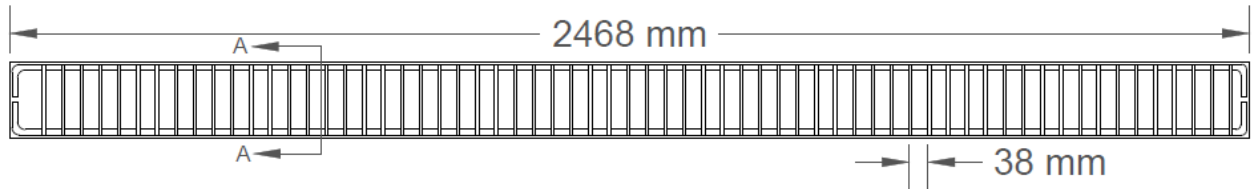
Table 3.1 Properties of test specimen

Series	Columns	Concrete Mix	Cross-Section	Transverse Reinforcements Spacing (s)	Transverse Reinforcing Steel Ties	Fibre Type	Fibre Content (%)
Series1	CRC-2%B-75	CRC	152.4 mm X 152.4 mm	75 mm	4-10M bars	B	2%
	CRC-2%C-75					C	
	CRC-2%B-38			38 mm		B	
	CRC-2%C-38					C	
	CRC-2%B-75-15M			4-15M bars	B		
	CRC-2%C-75-15M				C		
Series 2	CRC-2%B-75-MMFX			75 mm	4-MMFX #3 bars	B	
	CRC-2%C-75-MMFX					C	
	CRC-2%D-75-MMFX					D	
	CRC-2%B-38-MMFX			38 mm		B	
	CRC-2%C-38-MMFX					C	

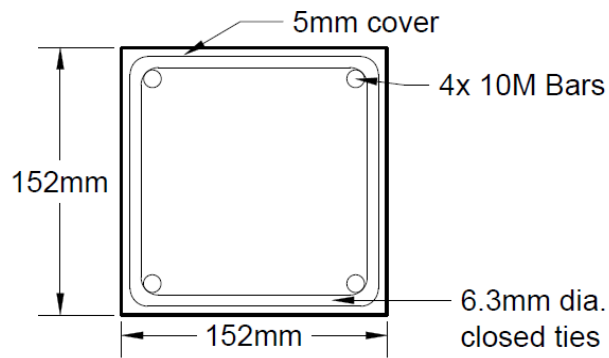
	CRC-3%C-75-MMFX			75 mm			3%
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(a) Non-seismic detailing

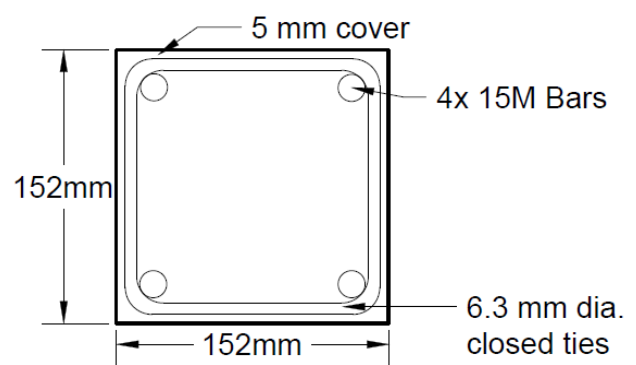


(b) Seismic detailing



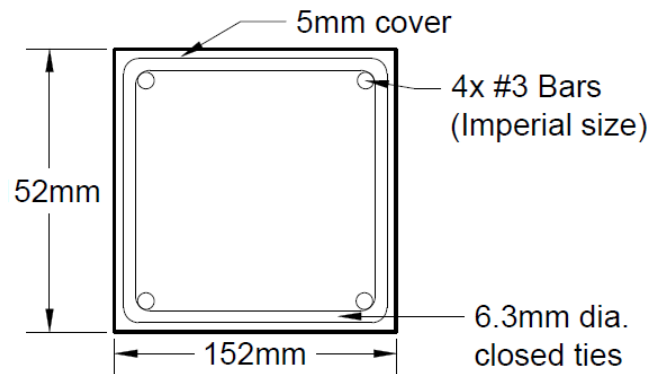
(c) Section A-A

10M longitudinal Reinforcements



(d) Section A-A

10M longitudinal Reinforcements



(e) Section A-A

#3 longitudinal reinforcements

Figure 3.1 Design of non-seismic, seismic and cross-section detailing

3.3 Materials

3.3.1 Reinforcing Steel (Rebar)

This experimental test program used various types of deformed steel rebar. Regular grade 10M and 15M rebar were used in the first series of tests, while the second series of tests used high-strength American size #3 MMFX steel reinforcement. Every column in this test program used 6.3 mm diameter non-deformed steel wire for the transverse reinforcement. Coupons for each type of steel were collected to perform standard tensile tests in a 600 kN Galdabini testing machine located in the structures lab at the University of Ottawa (see Figure 3.3 MTS testing machine). Typical stress-strain relationships for each steel type are shown in Figure 3.2 Stress-Strain relationship for 10M, 15M, MMFX and 6.3 mm wire while Table 3.2 Average steel properties for the steel reinforcement used in this study summarizes average strength and strain properties obtained from the stress-strain curves.

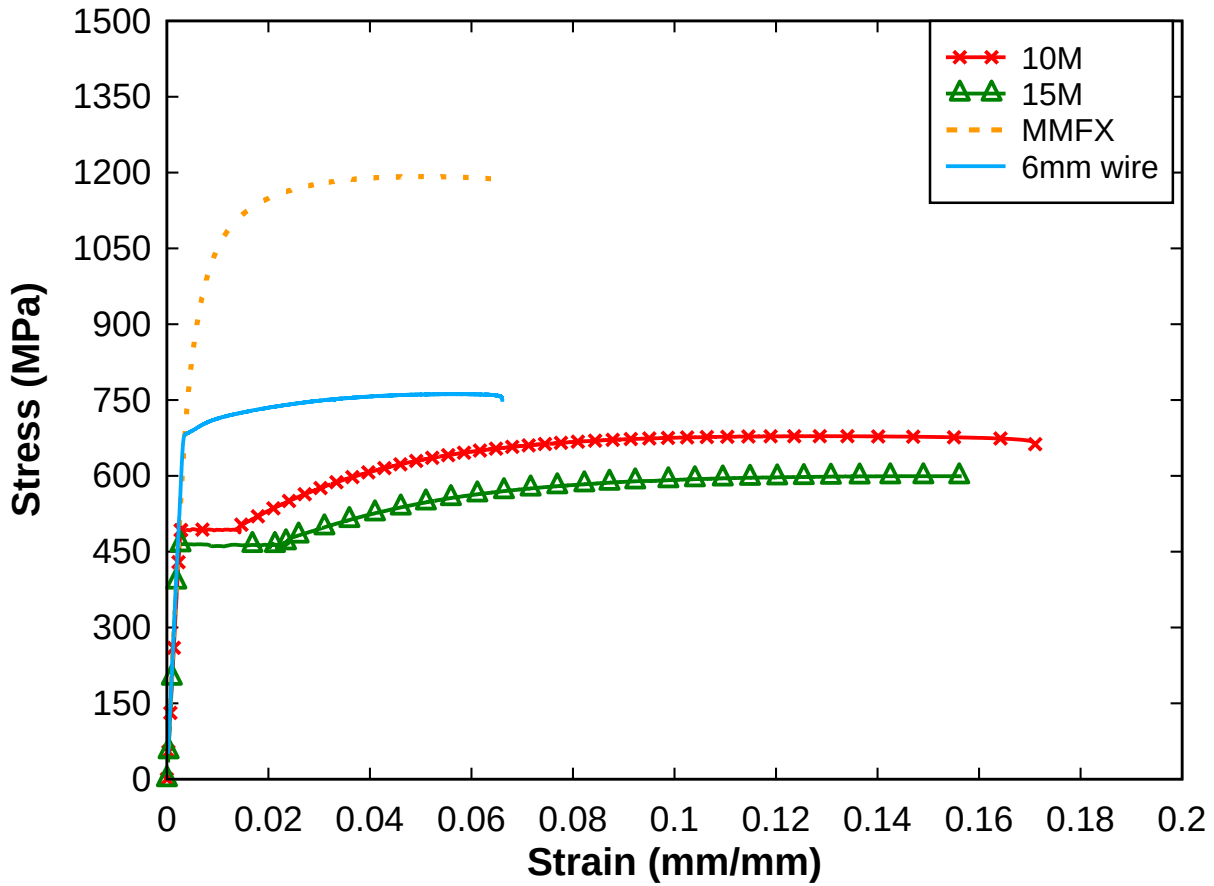


Figure 3.2 Stress-Strain relationship for 10M, 15M, MMFX and 6.3 mm wire

Table 3.2 Average steel properties for the steel reinforcement used in this study

Steel ID	Yield		Strain-hardening	Ultimate		Rupture	
	f_y MPa (ksi)	ϵ_y mm/mm		f_u MPa (ksi)	ϵ_u mm/mm	f_r MPa (ksi)	ϵ_r mm/mm
10M	486 (71)	0.0027	0.0127	678 (98)	0.1369	670 (97)	0.1690
15M	457 (68)	0.0022	0.0219	599 (87)	0.1565	502 (73)	0.1819
MMFX	956 (138)*	0.0070*	--	1192 (173)	0.0500	1187 (172)	0.0640
6 mm wire	604 (88)*	0.0033*	--	694 (101)	0.0607	689 (100)	0.0706

* no distinct yield plateau: yield values obtained using 0.2% offset method

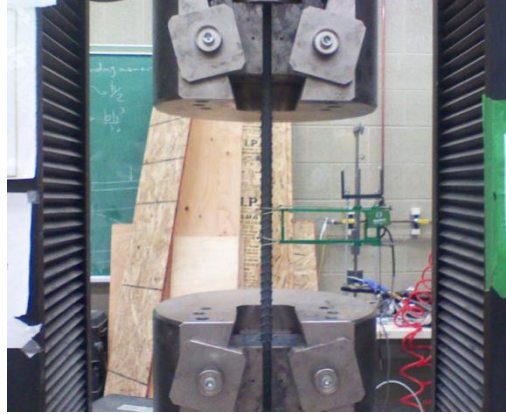


Figure 3.3 MTS testing machine

3.3.2 Steel Fibres

Three different types of fibres were considered in the research program to examine the effect of fibre aspect ratio, length and tensile strength on column blast performance. The results were also compared to columns constructed with a forth fibre type studied by Burrell (2012). The fibres are referred to as fibre types A, B, C, and D in the specimen nomenclature. The properties of the steel fibre used in this study are summarized in Table 3.3. The Stratec, OL, and BEL fibres are manufactured by Stratec GMBH (Germany), Bekaert Cooperation (Belgium), and OJSC (Belarus) respectively.

The CRC used in five of the twelve specimens contained OL 12/0.2 fibres (Fibre B). These fibres had a length (l_f) of 12 mm, a diameter of 0.2 mm (d_f), and an aspect ratio (l_f/d_f) of 65, and had a smooth/straight profile with a tensile strength of 2000 MPa. Six of the twelve columns were built using BEL M-13.0.3 fibres (Fibre C). These fibres had a length of 13 mm, a diameter of 0.3 mm, and an aspect ratio (l_f/d_f) of 43. This brass-coated fibre also had a straight/smooth profile but had a higher tensile strength of 3150 MPa. The Stratec 20/0.2 fibres (Fibre D) were used in only one specimen and had a length of 20 mm, a diameter of 0.2 mm, resulting in an aspect ratio (l_f/d_f) of 100. These fibres also had a straight/smooth, but had a much lower tensile Strength (1350 MPa) in comparison to the two other fibres. The forth fibre that was studied in Burrell's (2012) research program was also manufactured by Stratec but had different properties. Referred to as fibre type A, the Stratec 13/0.4 fibre had a length of 13 mm, a diameter of 0.2 mm, and an aspect ratio (l_f/d_f) of 30, with a tensile strength of 1350 MPa. All fibres came in bags and had to be added manually to the CRC during mixing. The majority of the columns in the research

program had fibres added at a ratio of 2% by volume of concrete (156 kg/m^3). One specimen was built using 3% fibres (234 kg/m^3) to study the effect of fibre content on blast resistance.

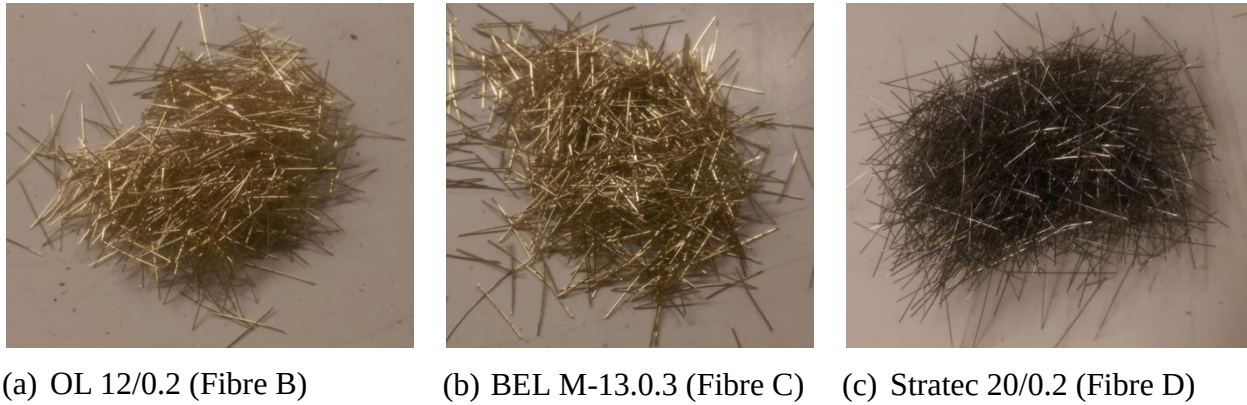


Figure 3.4 Fibres used in this study

Table 3.3 Fibre properties

Fiber	Length Mm (in.) (l_f)	Diameter Mm (in.) (d_f)	Aspect Ratio (l_f/d_f)	Tensile Strength MPa (ksi)
(A) Stratec 13/0.4*	13 (0.51)	0.4 (0.016)	30	1350 (196)
(B) O/L 12/0.2	12 (0.47)	0.2 (0.080)	65	2000 (291)
(C) BELt M 0.3/13	13 (0.51)	0.3 (0.012)	43	3150 (457)
(D) Stratec 20/0.2	20 (0.79)	0.2 (0.080)	100	1350 (196)

*This fibre was used in the Burrell (2012) research program

3.3.3 Concrete

All concrete used during this experimental program was mixed and casted at the University of Ottawa. A pre-packaged CRC mix was used to cast the twelve UHPFRC columns.

3.3.3.1 ***UHPFRC (CRC Joint Cast)***

The twelve UHPFRC columns in the study were built using CRC Joint Cast. The name CRC is a proprietary brand name for an UHPFRC produced by CRC technology, Denmark. The pre-packaged CRC mix consisted of *Densit Binder* which contains a mixture of microsilica and Portland cement, quartz sand and a superplasticizer in dry powder form (Nielsen 1995). It is noted that the mix contains no coarse aggregates and has a maximum aggregate size of 4 mm. The properties and components of the CRC mix are listed in Table 3.4. To improve tensile resistance, toughness and ductility fibres are added to the mix at volumetric ratios which can

range from 2-6%. In the current study the fibre contents ranged from of 2% to 3% using various fibre types as discussed in the previous section.

Table 3.4 Components of CRC Joint Cast mix with 2% to 3% steel fibres by volume

Component*	Fraction by Weight (%)	Density (kg/m ³)	Content (kg/m ³)
Densit Binder	34	Cement: 3150 Microsilica: 2250	1146
Quartz Sand			
0 – 0.25 mm	7	2640	236
0.25 – 1 mm	12	2640	404
1 – 4 mm	24	2740	809
Water	6	1000	202
Water-binder ratio			0.18

*Details of manufacturer's additives are proprietary

3.4 Construction of Test Specimens

The construction of the specimens was completed in the structures lab at the University of Ottawa. The construction of the columns can be separated into 5 stages. The first stage was the preparation of the 6.3 mm ties, where the 6.3 mm wire was cut to a length of 70 cm. Three 90° angles and two 135° angles in specific order were then used to bend the bar using a jig to form a square tie with hook extensions (see Figure 3.5). A total of 544 hoops were bent for these specimens. The second stage was the assembly of the cages. Longitudinal reinforcement bars were bent 90° extending 75 mm at the end to provide full development of the rebar. Spacing on the longitudinal reinforcement was marked so that the transverse reinforcement could be placed and fixed using quick tie wire. Figure 3.6 shows cages corresponding to columns with and without seismic detailing. Once the cages were completed, the third stage, involved addition of strain gauges at specified locations on the longitudinal and transverse steel reinforcement. A belt sander was used to create a smooth surface on the bars before attaching the strain gauges. Strain gauges were added to one compressive and one tensile longitudinal bar, and to a 6.3 mm wire all at mid-span. The fourth stage of the construction was to build horizontal form work using 19mm

plywood. As shown in Figure 3.7, a total of six specimens could be casted per formwork. The last stage of the construction was to cast the concrete.

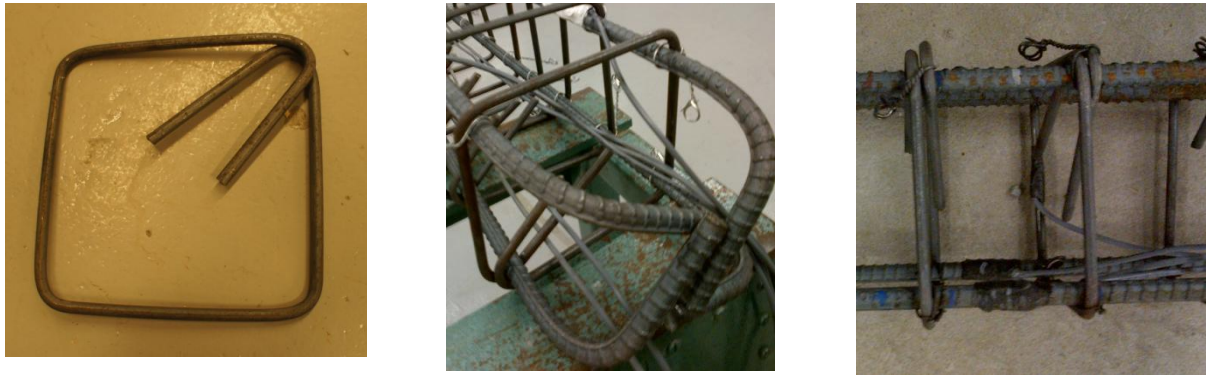


Figure 3.5 Typical hoop (left), rebar end hook (middle) and strain gauges at mid-span (right)



Figure 3.6 Non-seismic (top) vs. Seismic (bottom) detailing



Figure 3.7 Formwork containing six complete specimen cages

3.4.1 Casting

3.4.1.1 Mixer

All specimens were cast with the same mixer located in the structures lab at the University of Ottawa. This mixer was a 420-volt electric multi-flow pan mixer built with three stationary paddle blades on a shaft that creates force mixing action once the pan starts revolving at high velocity. This mixer also contained other side paddle blades that act as scrappers so no concrete can build up on the side of the pan. The mixer can handle a maximum capacity of two columns batched at once (0.108 m^3), around 12-13 bags of CRC mix; adding any more would result in overflow around the sides when the pan started rotating. The high velocity that the mixer was providing was ideal for the proper distribution of the fibres with the CRC mix. Figure 3.8 shows the multi-flow pan mixer in the structures lab at the University of Ottawa.



Figure 3.8 Pan Mixer used in the experimental program

3.4.1.2 Mixing Procedure CRC

Prior to the mixing, the pan mixer was wetted and any residual water in the pan was removed before adding any mix. Six bags of pre-packaged CRC mix were added to the pan. Sometimes two columns with identical fibre content were mixed together. A total of 13 bags were added for a double batch. The extra bag was added to allow for the construction of control compression cylinders. The CRC was dry mixed for 30 seconds before water was added slowly without stopping the mixer. The superplasticizer took about 10 minutes before being sufficiently activated. Once the superplasticizer was active, the fibres were evenly distributed in the mix over a time period of about 1 minute. Everything together was then mixed for another 5 minutes.

3.4.1.3 Casting and Curing

After successful mixing, the concrete was transferred to a wheel barrow. From the wheel barrow, the concrete was transferred to the formwork with plastic buckets. Figure 3.9 shows the placing procedure used during this experimental program. Once the concrete was in place, a 25 mm (1 in.) vibrator was used to ensure proper placing due to confined areas in between the formwork and the cages. After the pouring and vibrating were completed, the concrete was levelled with

trowels and covered with wet burlap and plastic sheeting. The columns were left to cure under these conditions for 5 days, and removed from their formwork once the curing was complete.

3.4.1.4 Workability

Some difficulties were encountered during the placement procedure of some specimens. In the case of the seismic specimens, the combination of 5 mm cover, transverse spacing of 38 mm and 13 mm steel fibre length made the placement procedure challenging. Increasing the fibre length to 20 mm had a significant impact on the workability of the CRC. As noted in Table 3.5, the slump for specimen CRC-2%D-75-MMFX was not measured due to the complete loss of CRC mix flowability with this longer fibre type.

3.4.2 Concrete Properties

3.4.2.1 Fresh State Properties

In order to measure the workability of the CRC, all batches of concrete were subjected to an ASTM C1611 Slump Flow test. Segregation and uniformity was examined visually after the slump test and during the casting. The slump flow test procedure involves filling a standard slump cone with concrete and lifting to cone to measure the horizontal slump (or spread) of the mix. The slump flow is obtained by averaging the spread measured in two perpendicular directions. Table 3.5 lists the measurements of slump flow for all batches mixed during this experimental program. In general all mixtures with 2% fibres were workable without signs of segregation. Increasing the fibre content, fibre aspect ratio and fibre length affected the workability of the CRC mixtures. For instance, the use of 3% fibres resulted in a slight reduction in slump flow, with a more noticeable effect during concrete placement. Similarly the use of the OL fibres (fibre B) resulted in reduced workability when compared to fibre type C due to the increase in aspect-ratio. Test specimens of this experimental program contained fibres with lengths varying from 12 mm to 13 mm with the exception of one column containing fibres with 20 mm length (fibre type D). The mix containing 20 mm long fibres had significantly lower slump flow spread compared to other mixtures and very noticeable reduction in workability. With the exception of the containing 20 mm fibres, all remaining mixes were sufficiently

workable for typical construction. None of the mixes were fully self-consolidating and needed to be vibrated to avoid concrete honeycombing in order to ensure proper concrete placement.

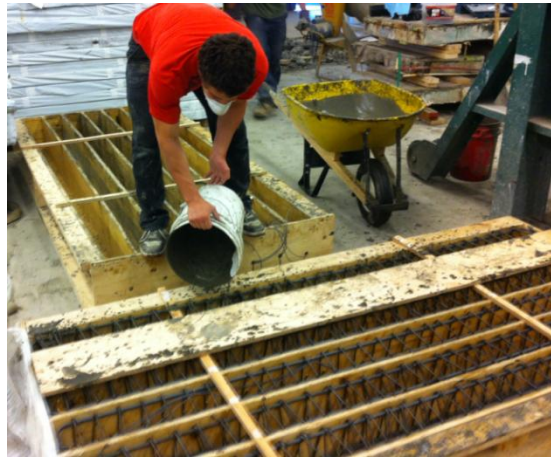


Figure 3.9 Placing concrete in formwork

Table 3.5 Slump flow average diameter of specimens

Series	Columns	Average Slump Flow (mm)
Series1	CRC-2%B-75	455
	CRC-2%C-75	555
	CRC-2%B-38	440
	CRC-2%C-38	555
	CRC-2%B-75-15M	455
	CRC-2%C-75-15M	455
Series 2	CRC-2%B-75-MMFX	460
	CRC-2%C-75-MMFX	455
	CRC-2%D-75-MMFX	n/a*
	CRC-2%B-38-MMFX	460
	CRC-2%C-38-MMFX	455
	CRC-3%C-75-MMFX	465

*Slump not measured for this batch due to loss of CRC mix flowability with this fibre type



Fibre B – 2% (455 mm)

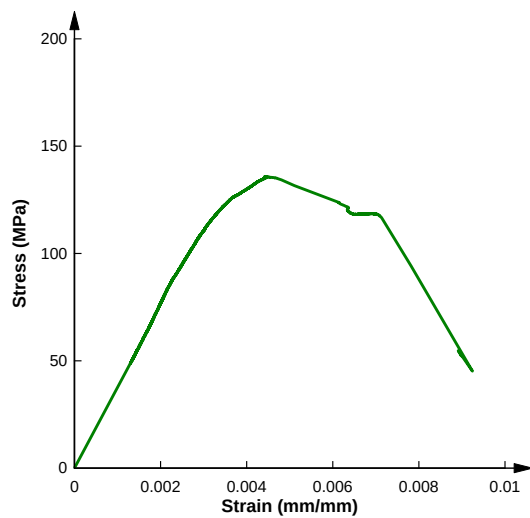


Fibre C – 2% (555 mm)

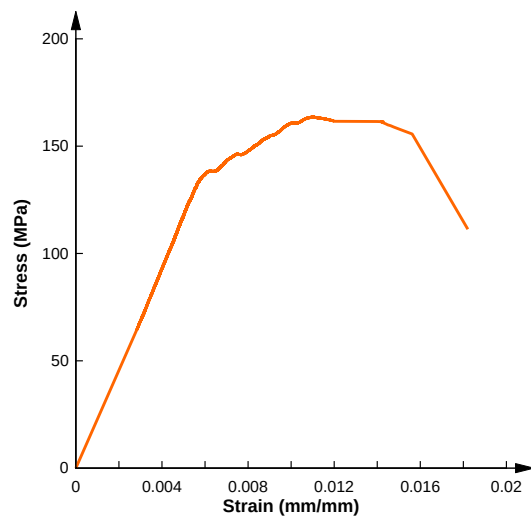


Fibre C – 3% (465 mm)

Figure 3.10 Photos of slump flow for different fibre types and fibre contents



(a) CRC-2%C-75-15M



(b) CRC-2%B-75 and CRC-2%B-75-15M

Figure 3.11 Typical concrete cylinders stress vs. strain curves for CRC

3.4.3 Hardened State Concrete Properties

During the casting process, a series of concrete cylinders were cast and cured under similar conditions to those used during the construction of the test specimens. For each batch, standard 100 mm by 200 mm cylinders were cast to determine the compressive strength of the UHPFRC mixtures. Table 3.6 lists the average peak compressive strength for the cylinders obtained for each column. A limited number of cylinders were tested with two axial LVDTs to obtain full stress-strain curve relationships. Figure 3.12 shows the setup used to determine the peak compressive strength and stress-strain curves at the University of Ottawa structures laboratory.

Due to very limited amounts of spare concrete during the mixing process, only one cylinder could be made for some batches.



(a) Cylinder testing machine
(University of Ottawa)



(b) Stress-strain setup
(University of Ottawa)

Figure 3.12 Compression test setup

Table 3.6 Concrete properties

Columns	Concrete Type	Fibre Type	Fibre Content V_f (%)	Compressive Strength (Std Dev) (MPa)
CRC-2%B-75	CRC Joint Cast	OL 12/0.2	2.00	155(14)
CRC-2%C-75		BEL M-13.0.3	2.00	154(5)
CRC-2%B-38		OL 12/0.2	2.00	155*
CRC-2%C-38		BEL M-13.0.3	2.00	154(5)
CRC-2%B-75-15M		OL 12/0.2	2.00	155(14)
CRC-2%C-75-15M		BEL M-13.0.3	2.00	130(8)
CRC-2%B-75-MMFX		OL 12/0.2	2.00	108*
CRC-2%C-75-MMFX		BEL M-13.0.3	2.00	125(23)
CRC-2%D-75-MMFX		Stratec 20/0.2	2.00	142(0)
CRC-2%B-38-MMFX		OL 12/0.2	2.00	108*
CRC-2%C-38-MMFX		BEL M-13.0.3	2.00	125(23)
CRC-3%C-75-MMFX		BEL M-13.0.3	3.00	166(1)

*Only one cylinder made from extra concrete

3.5 Test Procedure and Instrumentation

3.5.1 Test Setup

The testing setup used during this experimental program is shown in Figure 3.13. Every specimen in this study was tested under combined axial loading and simulated transverse blast loads. The axial load was applied on the columns at the start of testing using a hydraulic jack, while the transverse loading was applied using the Shock Tube at the University of Ottawa.

3.5.2 Axial Loading Mechanism

The axial load was applied to the columns using a manual pump and a 6 inch² (3871 mm²) hydraulic jack which was placed at the base of the columns. A total of 300 kN axial load was applied to the columns at the start of testing, corresponding to 30% of nominal axial load capacity of the conventional concrete (SCC) columns tested by Burrell (2012). It is noted that this load was near the capacity of the hydraulic jack, which was placed directly below the column. The column was then pinched between the jack and the 940 mm strong-floor slab at the top of the columns. Two bevelled steel plates with a wedged steel roller were placed between the column-jack and column-slab interfaces at the column ends to allow rotation of the column ends under transverse loading.

3.5.3 Lateral Load Transferring Mechanism

When working with non-planar elements in the shock tube, a load transferring device (LTD) must be used to transfer the shockwave pressure generated at the shock tube opening to the structural components. In this case of columns, the LTD collects the air pressure created by the shock tube and distributes it uniformly as a series of point loads along the column length. The LTD used during this experimental program was built with 23 gauge sheet metal connected to a series of steel beams. A total of eight 76.2 mm X 76.2 mm X 2438 mm hollow steel beams were attached to the sheet metal (6.53 mm thickness). These hollow steel beams spanned the width of the shock tube opening while being perpendicular to the specimen. Two similar six inch long sections were welded above and below the beams at mid-span to improve the distributed load along the column. The 23 gauge sheet metal used had a thickness of 0.683 mm and covered the entire shock tube opening (2438 mm x 2438 mm). The LTD weighed roughly 209 kg. The LTD was connected to the shock tube by a pair of half inch bolts at the top, allowing full rotation. All

other locations on the LTD were free to move up to the point of maximum deflection of the column. Figure 3.16 shows the LTD and the setup used during this experimental program.

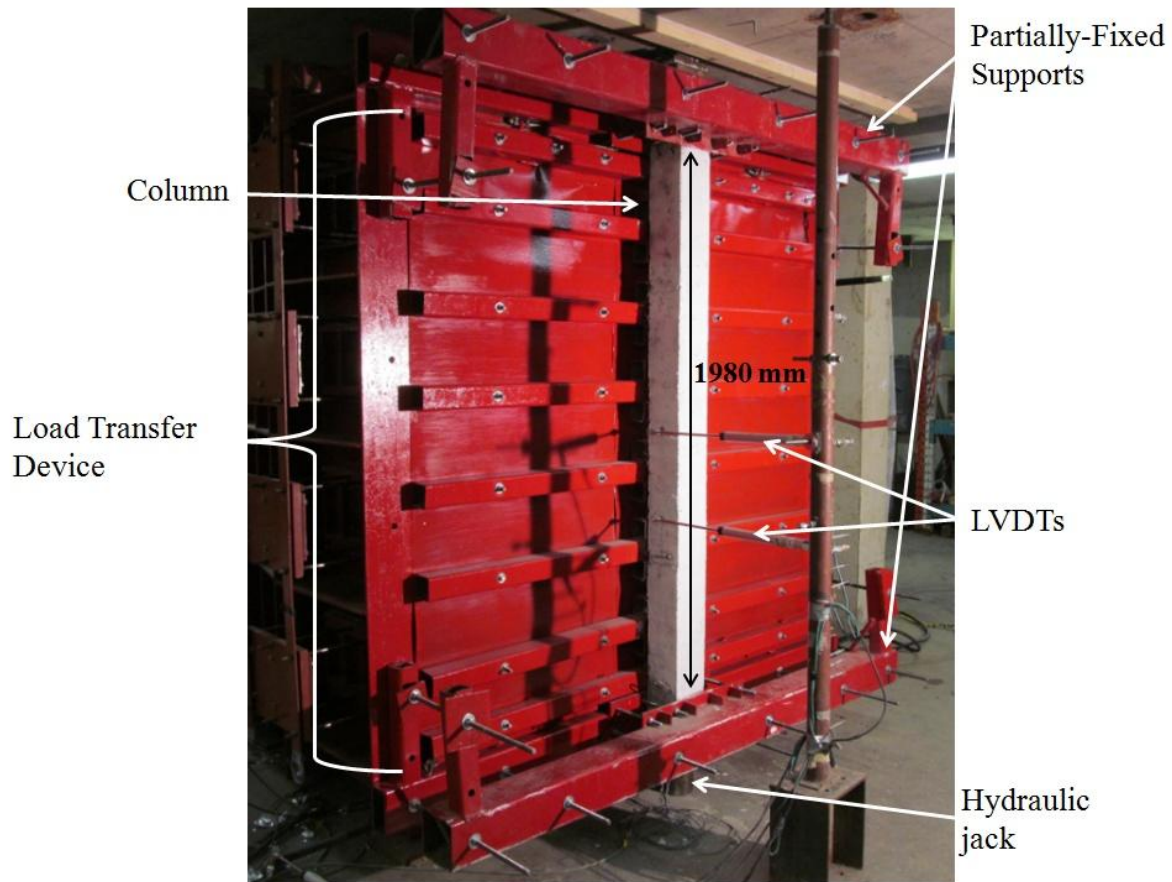


Figure 3.13 Typical test setup

3.5.4 Supports

The supports used during this experimental program were considered partially-fixed. Being identical, both top and bottom supports restrained the column laterally while preventing it from rotating. Figure 3.14 shows the details of the top support, which was identical to the bottom support used for all tests. Previous research work at the University of Ottawa using the same setup determined that the supports have an approximate rotational stiffness of $903000 \text{ N}\cdot\text{m}/\text{rad}$ (Jacques et al. 2012). The supports were built with a steel plate welded on six inch hollow structural steel sections. Twelve 15.9 mm bolts were used to connect each support to the shock tube's end frame. The column's clear span between the supports measured 1980 mm.



Figure 3.14 Angle (top) and close up (bottom) view of the top support used in this study

3.5.5 Instrumentation

All columns tested in this experimental program were instrumented with electric resistance strain gauges placed on the longitudinal and transverse steel reinforcement (350 ohm linear strain gauges, 6 mm in length). Four strain gauges were used per column, all placed at the mid span region. A strain gauge was placed on both tensile and compressive longitudinal reinforcing bars to capture the onset of yielding and buckling, while two other gauges were placed on a transverse tie to assess the effect of confinement. Figure 3.15 shows the location of the strain gauges in a typical column cross-section at mid-span.

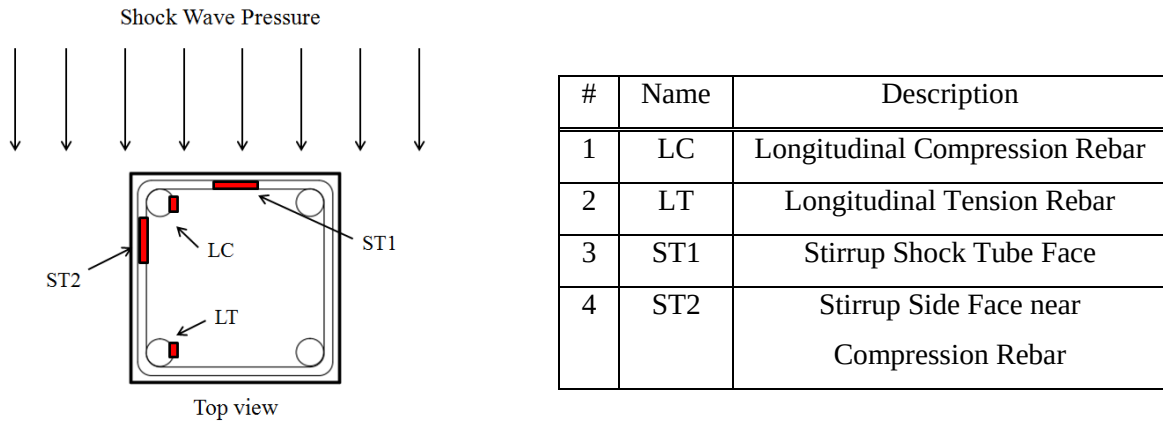


Figure 3.15 Strain gauge locations and descriptions

Full displacement-time histories were recorded using two linear variable differential transducers (LVDT) with a 300 mm stroke. The LVDTs were placed at 1/2 (mid-height) and at 1/3 of the clear span on the column. The column had an unsupported length of 1980 mm, which resulted in having one LVDT at 990 mm and the other at 600 mm measured from the bottom support. Linear extrapolation could be done with the LVDT at third span to find the maximum displacement in case of failure of the LVDT at mid-span. The same extrapolation could be done to find the maximum displacement if the peak displacement was to happen other than at mid-span. The pressure was measured using voltage differential high resolution pressure probes with 14.5 mV/kPa sensitivity capable of reading a maximum pressure of 345 kPa (5000mV) (Lloyd, 2010). Figure 3.16 shows the LVDTs attached to a typical column.



Figure 3.16 LVDT connection to column (left) and support post (right)

3.5.6 Data Acquisition

The data from the tests was recorded using two oscilloscopes having a 100,000 Hz sampling rate (samples per second). For the current research program, four channels recorded the strain on the steel, two channels recorded the pressure and another two channels recorded the displacements. The data acquisition system (DAC) recorded the data on a continuous loop until a trigger signal was reached among one of the pressure sensors. Upon engaging this trigger mechanism the system starts recording the data, along with a high speed video which captures the response of the test specimens under loading (Lloyd, 2010). The pressure gauges placed alongside the load transfer device were utilized to record and plot full reflected pressure time histories for each test. Detailed observations of each test could be done with the help of the high-speed camera which recorded the response of the columns at 500 frames per second. This camera was placed to the side of the columns to observe deflections at high speeds.

3.5.7 Test Procedure and Loading Program

The columns in this research program were tested under gradually increasing shock tube-induced shock waves until failure. The failure of CRC columns was defined by tensile rebar rupture at mid-span and in some cases at the supports. The shock wave pressures were chosen based on previous research conducted by Burrell (2012) on concrete columns having similar properties. Each column was subject to three blasts with the exception of two specimens that had to be tested under a forth blast. The driver length for all blasts was kept at 9 ft (2743 mm). The driver pressures corresponding to Blast 35, Blast 80 and Blast 100 were 35 psi (207 kPa), 80 psi (552 kPa), and 100 psi (689 kPa), respectively. Blast 35 was chosen to bring the tension steel reinforcement in the columns to yield, while the 80 psi blast was used to induce damage in the columns (it is noted that most displacement comparisons were taken from this blast). Blast 100 was used to generate failures in the specimens. The forth blast, Blast 100(2), involved a second application of Blast 100, and was used to induce failures in a limited number of specimens. Figure 3.17 shows typical shockwaves corresponding to Blasts 35-100, while Table 3.7 summarizes the average shockwave properties for each Blast.

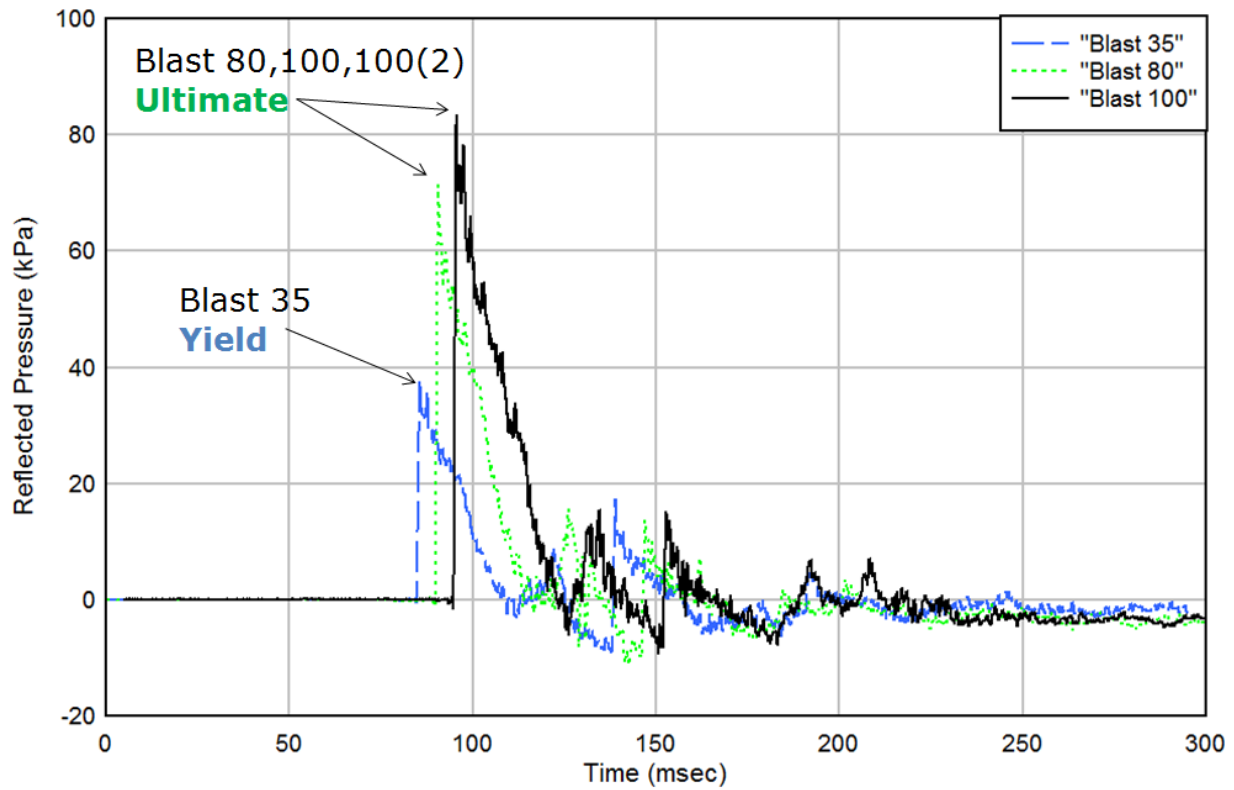


Figure 3.17 Reflected pressure vs. time history of blast shots

Table 3.7 Blast load properties

Name	Driver Pressure Psi (kPa)	Driver Length Ft (mm)	Average Reflected Pressure (kPa)	Average Reflected Impulse (kPa*msec)	Average Positive Phase Duration (Msec)
Blast 35	35 (207)	9 (2743)	42.8	388	22.95
Blast 80	80 (552)	9 (2743)	78.9	748	26.21
Blast 100	100 (689)	9 (2743)	96.3	893	25.09

4 Experimental Results

4.1 Introduction

This section of the thesis presents the results of the twelve specimens tested in this experimental program.. Section 4.2 provides a summary of the test results while section 4.3 summarizes the response for each column, including pressure, impulse and displacement histories, maximum and residual displacements, as well as a summary of damage progression and failure mode.

4.2 Summary of Results

The individual results for the columns tested in the experimental program are summarized in Table 4.1 for Series 1 and in Table 4.2 for Series 2 (MMFX series). The tables include shockwave property data such as recorded maximum reflected pressure (P_r), total reflected impulse over the positive phase duration (I_p) and the positive phase duration (t_d). The tables also list the maximum and residual mid-height displacements recorded for each blast (D_{max} and D_{res}).

A total of three blasts were applied to each column during this experimental program. The first blast was applied to test the column at yield (Blast 35). The second blast was used to bring the column past yielding and induce damage in the columns. This blast was also used to compare the effect of the various design parameters (Blast 80). The third blast tested the columns at ultimate conditions (Blast 100). In some cases, a forth blast was needed to cause column failure; this extra blast was identical to Blast 100 and is designated as Blast 100(2).

Pressure data presented in Table 4.1 was captured by the pressure transducers near the load transfer device during the tests. The pressure transducers were located on the interior of the shock tube expansion section. The impulse data presented in Table 4.1 is the integral of the recorded pressure-time histories over the positive phase duration. These impulse values represent the area under the pressure-time curve calculated using the trapezoidal method in the graphical software *Dplot*.

Maximum and residual mid-span displacement values noted in Table 4.1 are not cumulative since the LVDTs were zeroed prior to each blast test. Cumulative displacement data can be found in section 4.3. Strain gauge data can be found in the appendix section.

Table 4.1 Summary of Series 1 Test results (Regular Grade Steel)

		Shockwave Properties			Maximum Mid-Span Displacement	Residual Mid-Span Displacement	
		P _r	T _d	I _p	D _{max}	D _{res}	
		(kPa)	(msec)	(kPa*msec)	(mm)	(mm)	
CRC-2%B-75	Blast35	42.90	21.65	396.21	17.63	4.56	**
	Blast80	81.29	22.64	830.02	60.11	23.90	
	Blast100	86.14	21.70	997.49	NO DATA		
CRC-2%C-75	Blast35	41.64	17.91	326.39	15.68	1.13	* **
	Blast80	80.10	25.59	752.98	55.75	12.4	
	Blast100	95.20	28.54	885.70	NO DATA		
CRC-2%B-38	Blast35	41.56	19.60	402.46	21.80	1.23	*/†
	Blast80	75.12	19.68	802.80	61.03	16.70	
	Blast100	87.98	22.57	1010.67	277.70	259.80	
CRC-2%C-38	Blast35	41.54	18.60	396.02	19.64	1.77	*
	Blast80	80.30	26.57	745.94	41.36	9.23	
	Blast100	85.20	24.61	896.35	111.80	23.80	
CRC-2%B-75-15M	Blast35	41.67	20.67	393.64	19.93	1.99	*
	Blast80	76.77	22.64	789.91	51.97	16.83	
	Blast100	85.63	20.83	959.07	87.78	41.50	
	Blast100(2)	85.63	18.70	834.01	88.10	62.10	
CRC-2%C-75-15M	Blast35	40.73	17.92	381.84	20.20	1.45	*
	Blast80	74.61	27.56	800.09	50.60	13.01	
	Blast100	85.78	26.57	869.43	78.81	43.30	
	Blast100(2)	82.37	22.04	722.14	82.80	61.20	

* Maximum and residual displacements determined using high-speed video

** No data available due to failure of video recording

† No data available due to column leaving bottom support after severe failure.

‡ Specimen broke in half (see notes in section 4.3)

Table 4.2 Summary of Series 2 Test results (MMFX Steel)

		Shockwave Properties			Maximum Mid-Span Displacement	Residual Mid-Span Displacement	
		P _r	T _d	I _p	D _{max}	D _{res}	
		(Kpa)	(msec)	(Kpa*msec)	(mm)	(mm)	
CRC-2%B-75-MMFX	Blast35	39.47	22.64	445.41	20.39	1.45	
	Blast80	NO DATA			NO DATA		**
	Blast100	86.66	27.78	926.6	NO DATA		†
CRC-2%C-75-MMFX	Blast35	36.97	20.67	399.04	23.89	12.88	
	Blast80	72.02	24.6	779.15	87.94	32.44	
	Blast100	82.87	24.61	915.58	NO DATA		†
CRC-2%D-75-MMFX	Blast35	39.2	21.69	383.73	19.34	1.86	
	Blast80	77.78	27.56	760.91	66.44	28.98	
	Blast100	87.07	24.8	937.35	375.5	352.7	*/‡
CRC-2%B-38-MMFX	Blast35	39.54	22.64	497.65	16.86	1.85	
	Blast80	71.96	19.68	764.04	46.31	14.79	
	Blast100	80.81	23.44	864.06	94.32	8.87	*
CRC-2%C-38-MMFX	Blast35	38.42	20.67	405.81	20.48	6.8	
	Blast80	71.88	22.64	787.24	65.05	21.68	
	Blast100	83.01	31.5	926.42	310.55	292.08	*/‡
CRC-3%C-75-MMFX	Blast35	40.25	18.6	420.6	21.15	0.94	
	Blast80	72.97	30.51	821.05	61.32	15.66	
	Blast100	84.42	31.5	986.42	360.12	324.66	*/‡

* Maximum and residual displacements determined using high-speed video

** No data available due to failure of video recording

† No data available due to column leaving bottom support after severe failure.

‡ Specimen broke in half (see notes in section 4.3)

4.3 Description of Experimental Results – Series 1 (Regular Grade Steel series)

4.3.1 CRC-2%B-75

This column was the first of the series 1 specimens, which were constructed with CRC and regular grade longitudinal steel reinforcement. This specimen was built with four 10M steel bars. The transverse reinforcement design for this column had non-seismic detailing ($s = 75$ mm). The shock wave data that was recorded during the different blasts can be found in Table 4.1. Complete pressure and mid-span displacements time histories, and pressure and impulse time histories for each blast can be found in Figure 4.1. Post-blast photographs of the column after each blast can be found in Figure 4.2 and Figure 4.3.

Blast 35 was intended to test the column just beyond the elastic range up to yielding. The maximum and residual displacements after this shot were 17.63 mm and 4.56 mm respectively. Very little amounts of chipping can be observed along the compression face and the load transfer device interface which can be seen in Figure 4.2. Cracks are visible on the full length of this column. These cracks are believed to be caused during the initial application of axial load rather than being caused by the blast pressure itself.

Blast 80 was intended to bring the column within the plastic region without destroying it. The mid-span and residual displacements following this shot were 60.11 mm and 23.90 mm (28.46 mm cumulative) respectively. High speed video data shows a significant crack forming around mid-span. Already existing cracks located around the stirrups seemed to be more pronounced around mid-span after the blast. No spalling can be observed with the high speed video.

Blast 100 was intended to bring the column to failure. Based on previous research by Burrell (2012) it was expected that this blast could cause brittle column failure. In order to avoid failure of the LDVTs, no displacement-time histories were recorded during this blast. For later column high speed video camera frames were used to determine maximum and residual mid-span displacements, however, the high speed camera failed to record data during this blast. A three point failure mechanism occurred during this test. The failure of this column was due to the rupture of the tension steel at mid-span and at both supports. The column was still standing but only by the two compression 10M bars.

4.3.2 CRC-2%C-75

This column was the second specimen built with regular grade steel reinforcement. This column had identical details to column CRC-2%B-75 except that the CRC mix contained fibre type C. Different fibre properties were integrated in this research to study the effect of fibre properties on CRC column blast performance. This type of fibre was considered to be of high performance due to its comparatively high tensile strength (3150 MPa). Details about the fibres can be found in Table 3.3. The shock wave data that was recorded during the different blasts can be found in Table 4.1. Complete pressure and mid-span displacements time histories, and pressure and impulse time histories for each blast performed can be found in Figure 4.4. Photographs taken after each blast can be found in Figure 4.5 and Figure 4.6.

Blast 35 resulted in mid-span maximum and residual displacements of 15.68 mm and 1.13 mm respectively. Few cracks could be noticed near mid-span post-blast. Some concrete chipping occurred on the face of the column interacting with the load transfer device. Overall limited damage can be seen in Figure 4.6.

Blast 80 resulted in mid-span maximum and residual displacements of 55.75 mm and 12.40 mm (13.70 mm cumulative) respectively. Cracks were intensified near the middle of the column after this blast. The horizontal cracks occurred at a spacing of 75 mm, corresponding to the location of the transverse ties along the column height. The cracks are showed in Figure 4.6.

Blast 100 was conducted to bring the column to failure. No displacement measuring devices were used during this blast. Unfortunately, computer errors resulting in no high-speed video recorded during this blast. The tensile steel was ruptured at mid-span but not at the supports after this blast, however no spalling or concrete crushing was observed, even at failure. The column performed better than the identical column using fibre type B both in terms of post-blast damage and reduction in residual displacements after Blast 100. Figure 4.5 shows the overall minimal damage that occurred to the column during this blast.

4.3.3 CRC-2%B-38

This column was the second specimen built using fibre type B. The column had identical properties as CRC-2%B-75 but had transverse reinforcement designed for seismic regions ($s = 38$ mm) to study the effect of seismic detailing on column blast performance. The shock wave data

that was recorded during the different blasts can be found in Table 4.1. Complete pressure and mid-span displacements, pressure and impulse time histories for each blast can be found in Figure 4.7. Photographs of each blast's aftermath can be found in Figure 4.8 and Figure 4.9.

Blast 35 produced maximum and residual displacements of 21.80 mm and 1.23 mm respectively. No noticeable damage could be observed on the column post-blast. Small cracks along the length of the column can be noticed, as discussed previously, this was believed to be caused by the initial application of axial load. Pictures showing these cracks can be seen in Figure 4.9.

Blast 80 resulted in maximum and residual mid-span displacements of 61.03 mm and 16.70 mm (17.93 mm cumulative) respectively. The cracks located near the transverse ties showed to be more pronounced after this blast, as shown in Figure 4.8. From the high speed video recorded during the experiment, the plastic hinge seemed to have formed just below mid-span. The results from this column suggest that seismic detailing seems to help the column's performance in terms of reducing residual displacements.

Blast 100 was conducted to study the failure mechanism of the column. No displacement measuring devices were used during this blast. Using high-speed video camera frames, maximum and residual mid-span displacements were estimated to be 277.70 mm and 259.80 mm (277.73 mm cumulative) respectively. A three point hinge failure mechanism was observed, shown in Figure 4.8. The column failed with rupture of tension steel just below mid-span at the location of the most pronounced crack observed during the previous shot. This is believed to be caused due to having a slightly stiffer support at the top during this experiment. Stiffness can vary slightly depending on the way and the order the bolts on the setup are tightened.

4.3.4 CRC-2%C-38

This column was identical to the previous specimen but was built using fibre type C. The effect of both seismic design and fibre properties could be studied with this column. The shock wave data that was recorded during the different blasts with the shock tube inducers can be found in Table 4.1. Complete pressure, impulse and displacements time histories for each blast can be found in Figure 4.10. Post-blast photographs of the column can be found in Figure 4.11 and Figure 4.12.

Blast 35 resulted in mid-span maximum and residual displacement of 19.64 mm and 1.77 mm respectively. Minor cracks developed near mid-span on the tension face of the column while slight concrete crushing occurred along the compression face. Figure 4.11 shows the minimal damage sustained by the column during the blast.

Blast 80 resulted in mid-span maximum and residual displacements of 41.36 mm and 9.23 mm (11.00 mm cumulative) respectively. The pre-existing cracks were amplified while the appearance of new ones developed along the tension face. The biggest crack occurred at mid-span and can be seen in Figure 4.11. Both 10M specimens built using fibre type C outperformed the companion 10M columns built with fibre type B in terms of control of maximum and residual displacements.

As with previous specimens, Blast 100 resulted in column failure. Despite this, the failure of this column was much more controlled compared to the companion column having fibre type B. No LVDTs were used during this blast, however some displacement data could be estimated using high-speed video camera frames. Using the frames, maximum and residual mid-span displacements of 111.80 mm and 23.8 mm (34.80 mm cumulative) respectively were estimated, significant reductions when compared to previous columns in this test series. Rupture of the tension steel occurred solely at mid span with very little spalling and concrete crushing observed during this blast. The amount of cracking observed during this test was identical to the amount observed during Blast 80.

4.3.5 CRC-2%B-75-15M

This specimen had identical properties as column CRC-2%B-75 but was constructed using 15M longitudinal reinforcement instead of 10M bars. The purpose of using a different size of longitudinal reinforcement was to study the effect of reinforcement ratio on column blast performance. The shock wave data recorded during the different blasts can be found in Table 4.1. Complete pressure and mid-span displacements time histories, and pressure and impulse time histories for each blast can be found in Figure 4.13. Photographs of column damage after each blast can be found in Figure 4.14 and Figure 4.15.

Blast 35 resulted in a maximum and residual displacement of 19.93 mm and 1.99 mm respectively. Similar to the previous columns, this specimen had hairline transverse cracks along

its length due to the application of axial load and limited concrete cover near the ties. No damage can be observed following this test. Figure 4.14 shows the crack pattern which correlates with the tie spacing.

Blast 80 caused the column to have maximum and residual mid-span displacements of 51.97 mm and 16.83 mm (18.82 mm cumulative) respectively. Having similar displacement to the 10M specimen, the 15M suffered no additional damage after this blast. As shown in Figure 4.14, the column had unnoticeable difference in damage when comparing to the previous shot.

Blast 100 was performed to fail the 10M specimens. It was predicted that the use of 15M bars in this column would allow it to survive this blast, thus it was decided to keep an LVDT for data recording during this test. This blast resulted in the column having mid-span maximum and residual displacements of 87.78 mm and 41.50 mm (59.32 mm cumulative) respectively. At this stage, the damage dealt to the column was similar to the damage done on the companion 10M column at Blast 80. One larger crack around mid-span can be seen in Figure 4.14.

Having survived the previous blast, column CRC-2%B-75-15M was subjected to a second application of Blast 100. Expecting failure of the column, no LVDTs were used during this test. Using high-speed video camera frames, maximum and residual mid-span displacements of 88.10 mm and 62.10 mm (121.42 mm cumulative) were estimated for this test. It is noted that while both blasts had a reflected pressure of 85.63 kPa, the second application of Blast 100 resulted in a reduction of 125.06 kPa*msec in reflected impulse. Following Blast 100(2), the column looked slightly more damaged (see Figure 4.14), however no spalling or crushing was observed. The high speed video showed the tensile steel to have potentially ruptured. Upon closer inspection with a chisel, the tensile steel was confirmed ruptured. The result from this test confirms that increasing the reinforcement ratio can delay rupture of tension steel and improve the blast performance of UHPFRC columns.

4.3.6 CRC-2%C-75-15M

This last column in Series 1 was identical to specimen CRC-2%B-75-15M but used fibres with improved properties (Fibre type C). This column could be used to study the combined effect of improving fibre properties and increasing reinforcement steel area. The shock wave data that was recorded during the different blasts can be found in Table 4.1. Complete pressure, mid-span and

impulse time histories for each blast performed can be found in Figure 4.16. Photographs of each blast's aftermath can be found in Figure 4.17 and Figure 4.18.

Blast 35 resulted in a mid-span maximum and residual displacement of 20.20 mm and 1.45 mm respectively. No cracking occurred during this blast. Minor amounts of crushing occurred on the face interacting with the load transfer device.

Blast 80 resulted in mid-span maximum and residual displacements of 50.60 mm and 13.01 mm (14.46 mm cumulative) respectively. Minor tensile cracks occurred on the tensile face of the column during this blast. Cracking occurred at intervals of 75mm, corresponding to the locations of the tie reinforcement. Overall the column had very good damage tolerance (see Figure 4.17).

Blast 100 was intended to bring the column to failure. This column survived the blast with maximum and residual displacements of 78.81 mm and 43.30 mm (57.76 mm cumulative). No spalling or concrete crushing could be observed. The sizes of already existing cracks were magnified during this blast, and new cracks were visible near mid-span (see Figure 4.17). Since this column survived this blast, a second shot of Blast 100 was applied to the specimen.

Blast 100(2) resulted in maximum and residual mid-span displacements of 82.80 mm and 61.20 mm (118.96 mm cumulative) respectively. These were identified using high-speed video camera frames since no LVDTs were used during this blast. After further inspection after the test, no rebar rupture could be identified. This column showed to have the highest performance of all specimens during in this test series. This column was the only specimen to resist tensile bar rupture after four different blasts.

4.4 Description of Experimental Results – Series 2 (MMFX Steel series)

4.4.1 CRC-2%B-75-MMFX

This was the first column in the second series of CRC columns built using high-strength MMFX steel reinforcement. This column was built identical to CRC-2%B-75 but used 4 - #3 high-strength longitudinal bars. The properties of the MMFX bars can be found in Table 3.2. Despite having the same fibre type, fibre content, and stirrup spacing, the results from the MMFX columns cannot be compared directly with the companion 10M specimens since the area of the

American #3 rebar is reduced when compared to the Canadian 10M bars ($A_s = 71 \text{ mm}^2$ vs. 100 mm^2 , therefore total tension area of 142 mm^2 vs. 200 mm^2). The shock wave data that was recorded during the different blasts can be found in Table 4.2. Complete pressure and mid-span displacements time histories, and pressure and impulse time histories for each blast can be found in Figure 4.19. Photographs of the column after each blast can be found in Figure 4.20 and Figure 4.21. It should be noted that the compressive strength of the cylinders for this specimen was found to be 108 MPa, while the compressive strength of the remaining specimens varied between 140-160 MPa.

Blast 35 resulted in maximum and residual mid-span displacements of 20.39 mm and 1.45 mm respectively. Compared to previous specimens, this column had no cracks along the length where the ties were located. No visible cracks near mid-span could be identified and no concrete crushing occurred during this blast.

Unfortunately, no shockwave or displacement data could be recorded during Blast 80 for this column due to hardware malfunction. Blast 80 resulted in multiple mid-span cracks and concrete crushing on the face interacting with the load transfer device. The column condition at this stage looked more damaged when compared to the companion Series 1 columns having 10M rebar.

Blast 100 resulted in failure of the column. High speed videos were recorded but no displacement measuring tools were used. This blast overwhelmed the column's capacity and resulted in a very brittle failure. The three point hinge mechanism occurred along with ejection of the column from the lower support. As shown in Figure 4.20, the column was held up only by a 6 mm tie as both tension and compression rebar were ruptured. The MMFX column using non-seismic ties sustained the most amount of damage from Blast 100 among all fibre B columns.

4.4.2 CRC-2%C-75-MMFX

This next column was identical to column CRC-2%B-75-MMFX but used fibre type C instead of the previous fibre, type B. This column was built to study the effect of fibres in UHPFRC columns constructed using high-strength steel reinforcement. It is important to note that the compressive strength of the cylinders were of 125 MPa, slightly below average (140 – 160 MPa). The shock wave data that was recorded during the different blasts can be found in Table 4.2. Complete pressure and mid-span displacement time histories, and pressure and impulse time

histories for each blast can be found in Figure 4.22. Photographs after each blast are shown in Figure 4.23 and Figure 4.24.

Blast 35 resulted in mid-span maximum and residual displacements of 23.89 mm and 12.88 mm respectively. Following the blast, a few cracks formed on the tensile face of the column. These can be seen in Figure 4.23.

Blast 80 resulted in mid-span maximum and residual displacements of 87.94 mm and 32.44 mm (45.32 mm cumulative) respectively. These measures were slightly higher than the average displacements for other columns. Post blast examination of the column shows the tensile cracks where more pronounced, while one crack, just below mid-span, appeared to be going all the way through the column (see Figure 4.23). Tensile steel rupture was predicted to happen at this crack.

Blast 100 resulted in column failure with the three point hinge mechanism observed in previous tests. Tensile steel bar rupture occurred at both supports and mid-span. Just like specimen CRC-2%B-75-MMFX, the bottom of the column was expelled from the lower support during this test. The damage tolerance of the column was particularly low. The columns mid-section was fully blown off, and both tensile and compressive rebar were ruptured, with the column only held up by one of the 6 mm stirrups.

4.4.3 CRC-2%D-75-MMFX

This column was identical to the previous two specimens aside from being built using a third type of fibre, D. This column was the only specimen built using this type of fibre. The fibre properties can be found in Table 3.3; it is noted that these fibres measured 20mm in length making them 40% longer than other fibres using in this study. The shock wave data that was recorded during the different blasts can be found in Table 4.2. Complete pressure, impulse and mid-span displacements time histories for each blast can be found in Figure 4.25. Photographs of each the column after each blast can be found in Figure 4.26 and Figure4.27.

Blast 35 resulted in mid-span maximum and residual displacements of 19.34 mm and 1.86 mm respectively. A few cracks formed on the tensile face of the column after application of this blast pressure. These cracks can be seen in Figure 4.26.

Blast 80 resulted in mid-span maximum and residual displacements of 66.44 mm and 28.98 mm (30.84 mm cumulative) respectively. The previously formed tensile cracks were more pronounced after this blast. Figure 4.26 shows one crack, just below mid-span, going all the way through the column. Tensile steel rupture was predicted to happen at this crack.

Blast 100 resulted in this column's failure. No LVDTs were used during this test. Using high-speed video camera frames, maximum and residual mid-span displacements of 375.50 mm and 352.70 mm (381.68 mm cumulative) were estimated for this shot. These values do not represent much since the column suffered a very brittle failure and fully broke in half during this test. Tensile bar rupture occurred at mid-span and at both supports. The damage tolerance of the specimen was slightly higher than both previous specimens since the bottom of the column stayed within the support.

4.4.4 CRC-2%B-38-MMFX

This column was built exactly like the CRC-2%B-75-MMFX but was designed with seismic detailing ($s = 38$ mm). This was the last column built using fibre type B. The shock wave data that was recorded during the different blasts can be found in Table 4.2. Complete pressure, impulse and mid-span displacements time histories for each blast can be found in Figure 4.28. Photographs of each blast's aftermath can be found in Figure 4.29 and Figure 4.30. The concrete used for this column was of the same batch as specimen CRC-2%B-75-MMFX, thus resulting in a compressive strength of 108 MPa, which is comparatively lower than the average.

Blast 35 resulted in mid-span maximum and residual displacements of 16.86 mm and 1.85 mm respectively. Minimal amounts of concrete crushing can be seen along the face of the column interacting with the load transfer device. No cracks were observed following this blast.

Blast 80 resulted in mid-span maximum and residual displacements of 46.31 mm and 14.79 mm (16.64 mm cumulative) respectively. When considering all columns using fibre type B, the MMFX column designed for seismic regions had the lowest maximum and residual displacements at Blast 80. As shown in Figure 4.29, the column had moderate amounts of cracks on the tensile side near column mid-span, however overall damage was limited after this blast.

Blast 100 was intended to bring the column to failure. No LVDTs were used during this blast due to the brittle failures observed during the testing of previous specimens. Using high-speed video

camera frames, maximum and residual mid-span displacements were estimated to be 94.32 mm and 8.87 mm (25.51 mm cumulative) respectively. It is noted that column had minimal residual displacement, and upon primary inspection, the column seemed to have survived the blast without rebar rupture. No new cracks or additional concrete crushing were noticed. However, one crack at mid-span (see Figure 4.29) seemed much more pronounced when compared to the previous blast. High speed video data showed the column to have considerable maximum mid-span displacement which indicated the possibility of tensile steel rupture. Upon closer inspection of the column using a chisel and hammer, the tensile steel was confirmed ruptured due Blast 100.

4.4.5 CRC-2%C-38-MMFX

This column was built identical to specimen CRC-2%B-38-MMFX but used high strength steel fibres, Type C. This column could be compared to specimen CRC-2%B-38-MMFX to study the effect of style fibre properties since fibre type C had a higher tensile strength and a different aspect ratio. This column could also be compared with the previous column to study the effect of seismic detailing since the transverse reinforcement was designed for seismic regions ($s = 38$ mm). The shock wave data that was recorded during the different blasts can be found in Table 4.2. Complete pressure and mid-span displacements time histories, and pressure and impulse time histories for each blast can be found in Figure 4.31. Post-blast photographs of the column can be found in Figure 4.32 and Figure 4.33.

Blast 35 resulted in mid-span maximum and residual displacements of 20.48 mm and 6.80 mm. Following this blast, no damage could be observed on the column. Figure 4.33 shows the damage control of this column.

Blast 80 resulted in mid-span maximum and residual displacements of 65.05 mm and 21.68 mm (28.48 mm cumulative) respectively. Minimal amounts of cracking occurred during this test, with cracking located near the mid-span on the tensile face of the column. The cracks were spaced at 38mm, matching the location of the tie spacing. Minor amounts of concrete crushing at mid-span can be seen in Figure 4.33.

Blast 100 resulted in the failure of the column. The three point hinge mechanism that occurred can be seen in Figure 4.33. Bar rupturing only occurred at mid-span on the tension side. No LVDTs were used during this test due to previously mentioned reasons. Using high-speed video

camera frames, maximum and residual mid-span displacements of 310.55 mm and 292.08 mm (320.5 mm cumulative) were estimated for this shot. These values were significantly higher than those of the companion specimen constructed using lower strength fibres, CRC-2%B-38-MMFX.

4.4.6 CRC-3%C-75-MMFX

This was the last column tested in this experimental program. This specimen had a higher fibre content of 3%, but was otherwise identical to specimen CRC-2%C-75-MMFX. The results from this column allow for a study of the effect of fibre content. The shock wave data that was recorded during the different blasts can be found in Table 4.2. Complete pressure, impulse and mid-span displacements time histories for each blast can be found in Figure 4.34. Photographs of the column after each blast can be found in Figure 4.35 and Figure 4.36.

Blast 35 resulted in mid-span maximum and residual displacements of 21.15 mm and 0.94 mm respectively. No cracks were observed on the column following the blast. The maximum displacement obtain during this test was similar to the values recorded for other MMFX columns while the residual displacement, was comparatively much lower. The increase in the fibre content could be one reason for the reduction in residual displacements.

Blast 80 resulted in mid-span maximum and residual displacements of 61.32 mm and 15.66 mm (16.60 mm cumulative) respectively. The tensile cracks where more pronounced after this blast and Figure 4.35 shows one crack, near mid-span, going all the way through the column. Tensile steel rupture was predicted to happen at this crack.

Blast 100 was brought the column to failure. No LVDTs were used during this blast. Using high-speed video camera frames, maximum and residual mid-span displacements of 360.12 mm and 324.66 mm (341.26 mm cumulative) were estimated. Tensile bar rupture occurred at both mid-span and at the supports. The failure of the column resembled that of specimens CRC-2%B-75-MMFX and CRC-2%C-75-MMFX, without the bottom of the column leaving the support.

4.5 Pressure-Displacement Time Histories & Photos

4.5.1 CRC-2%B-75

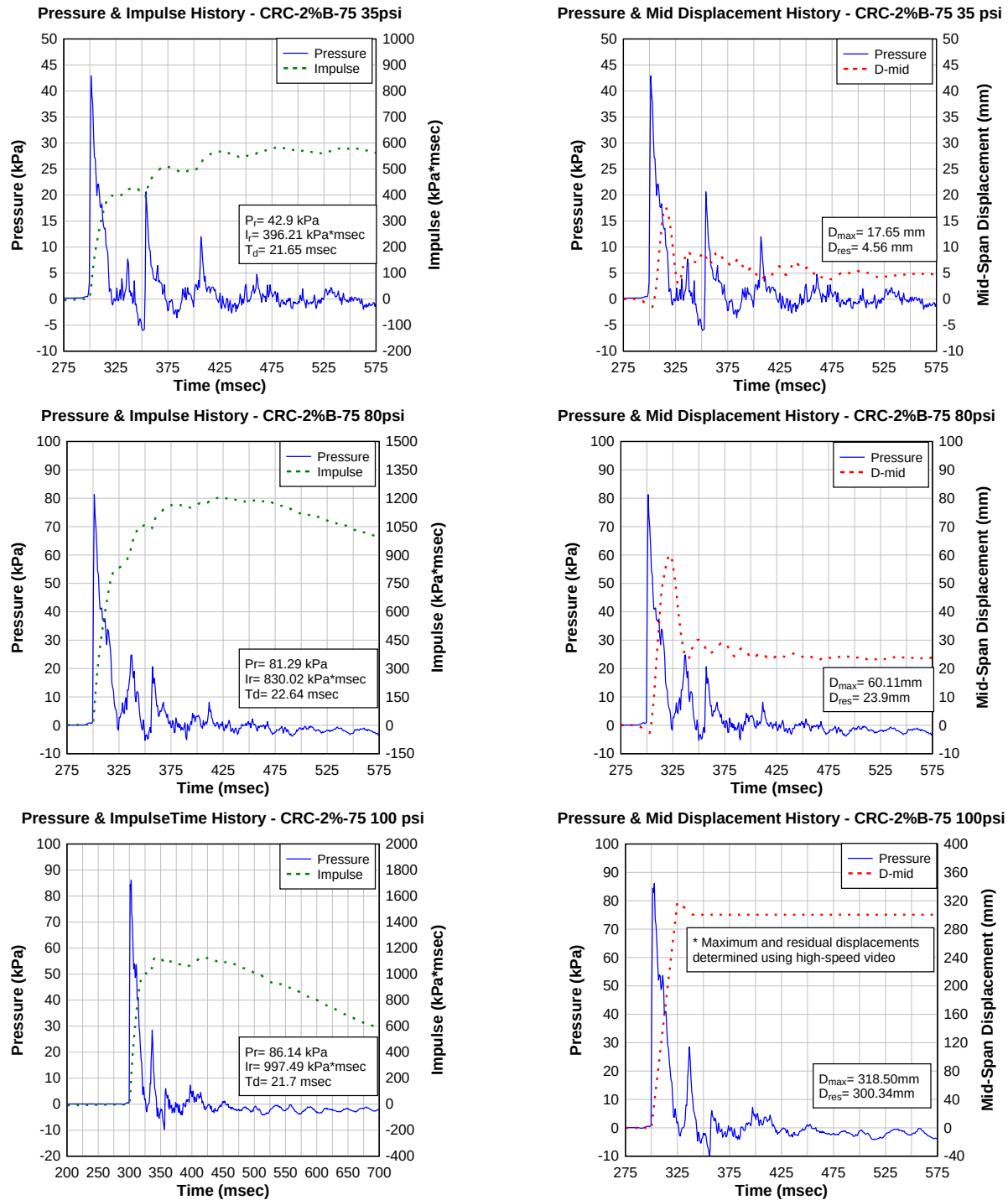
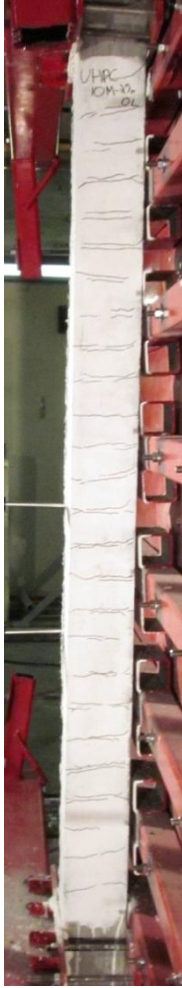
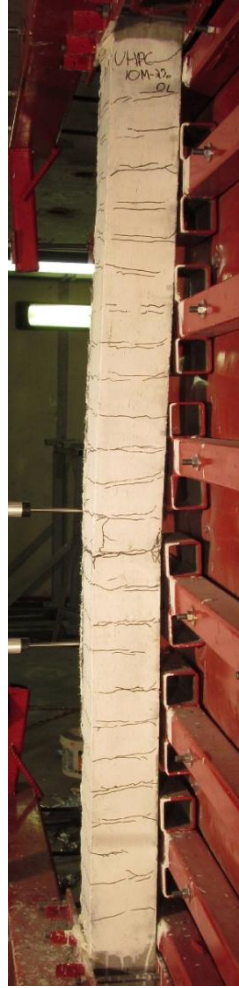


Figure 4.1 (Left) Pressure and impulse time history, and (right) pressure and mid-span displacement time history for column CRC-2%B-75



(a) Blast30



(b) Blast 80

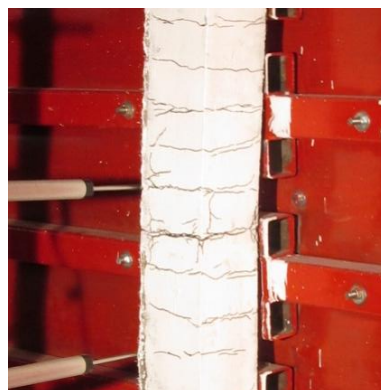


(c) Blast 100

Figure 4.2 Column CRC-2%B-75 at various stages of testing



(a) Blast30



(b) Blast 80



(c) Blast 100

Figure 4.3 Mid-span close up of column CRC-2%B-75

4.5.2 CRC-2%C-75

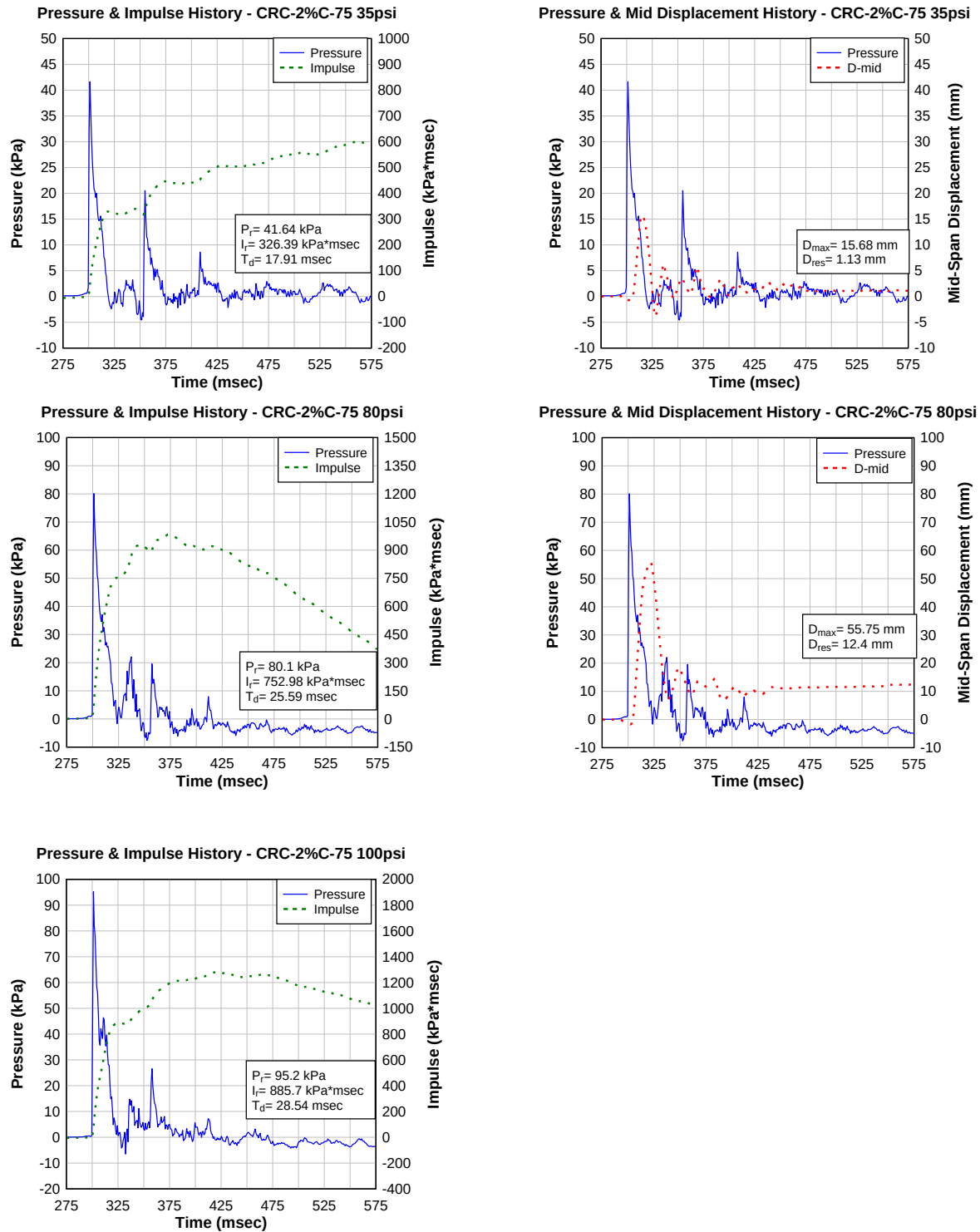


Figure 4.4 (Left) Pressure and impulse time history, and (right) pressure and mid-span displacement time history for column CRC-2%C-75



(a) Blast 35



(b) Blast 80



(c) Blast 100

Figure 4.5 Column CRC-2%C-75 at various stages of testing



(a) Blast 35



(b) Blast 80



(c) Blast 100

Figure 4.6 Mid-span close up of column CRC-2%C-75

4.5.3 CRC-2%B-38

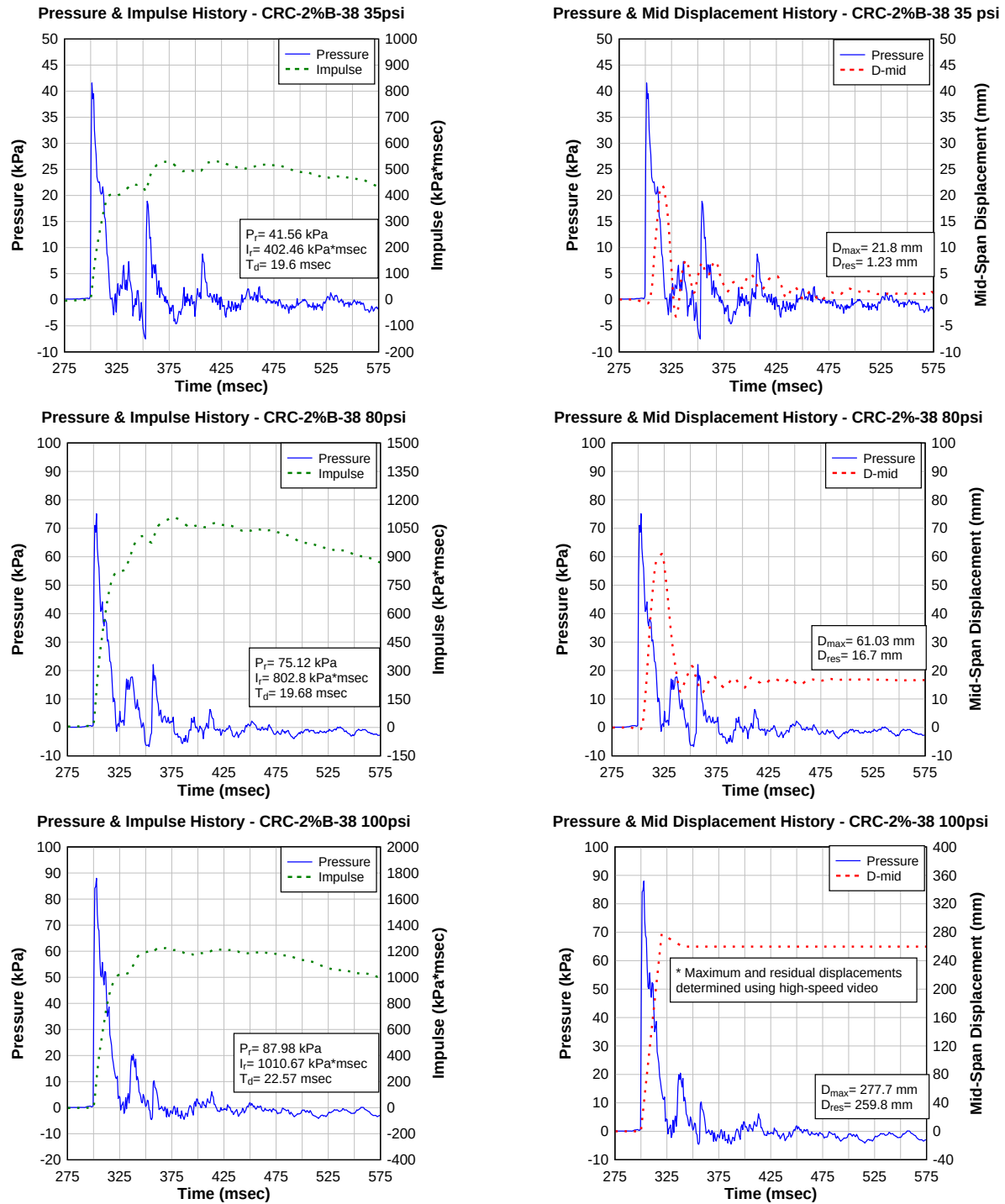


Figure 4.7 (Left) Pressure and impulse time history, and (right) pressure and mid-span displacement time history for column CRC-2%B-38



(a) Blast 35



(b) Blast 80



(c) Blast 100

Figure 4.8 Column CRC-2%B-38 at various stages of testing



(a) Blast 35



(b) Blast 80



(c) Blast 100

Figure 4.9 Mid-span close up of column CRC-2%B-38

4.5.4 CRC-2%C-38

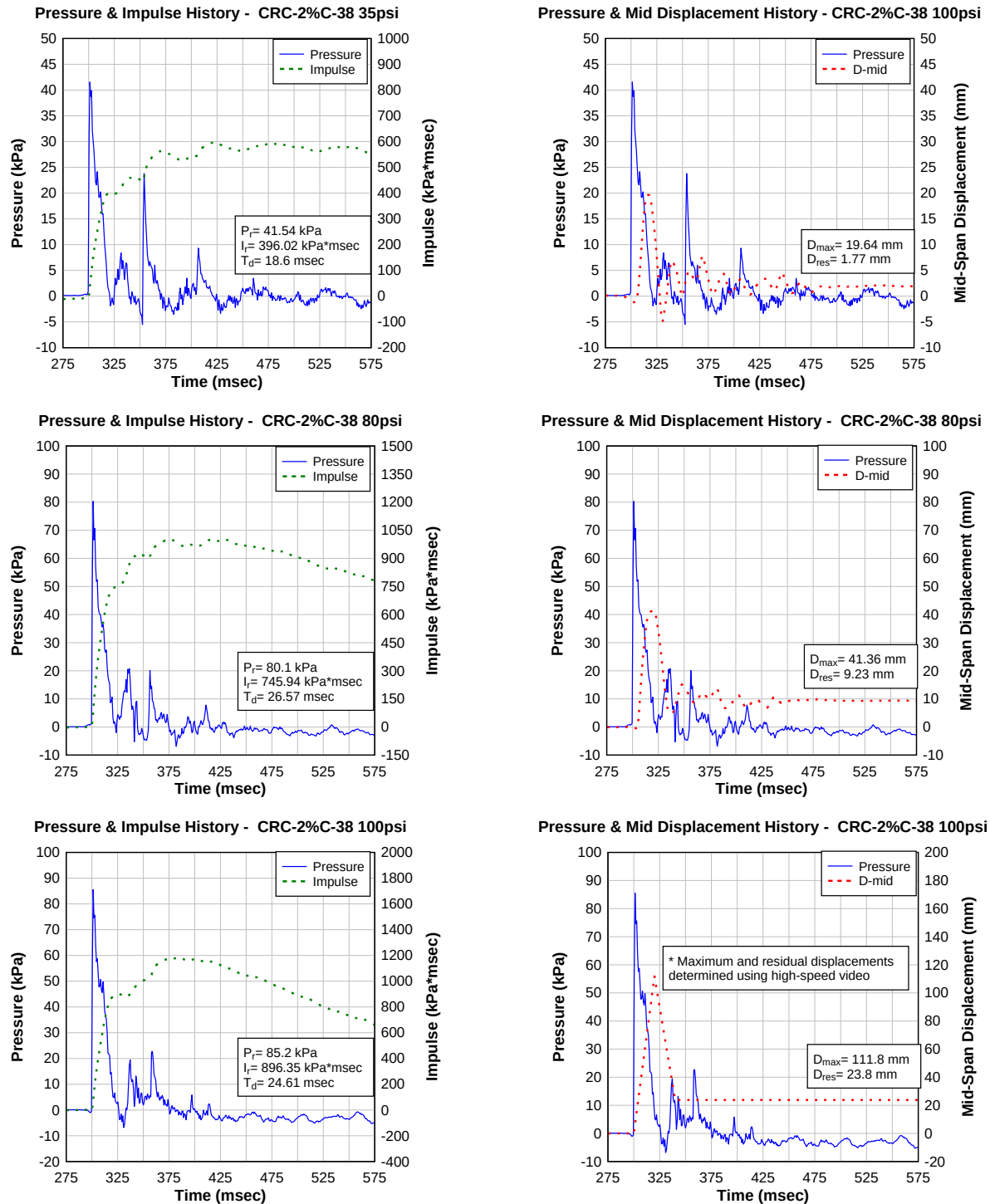
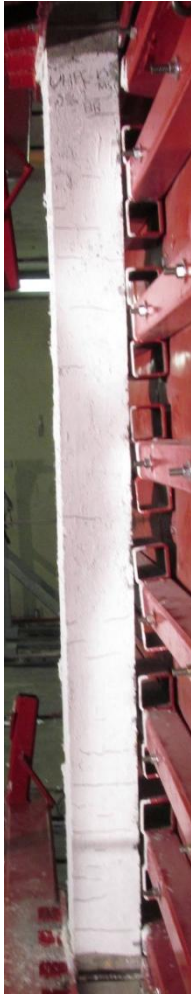


Figure 4.10 (Left) Pressure and impulse time history, and (right) pressure and mid-span displacement time history for column CRC-2%C-38



(a) Blast 35



(b) Blast 80

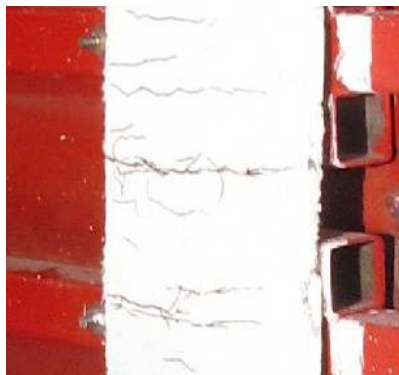


(c) Blast 100

Figure 4.11 Column CRC-2%C-38 at various stages of testing



(a) Blast 35



(b) Blast 80



(c) Blast 100

Figure 4.12 Mid-span close up of column CRC-2%C-38

4.5.5 CRC-2%B-75-15M

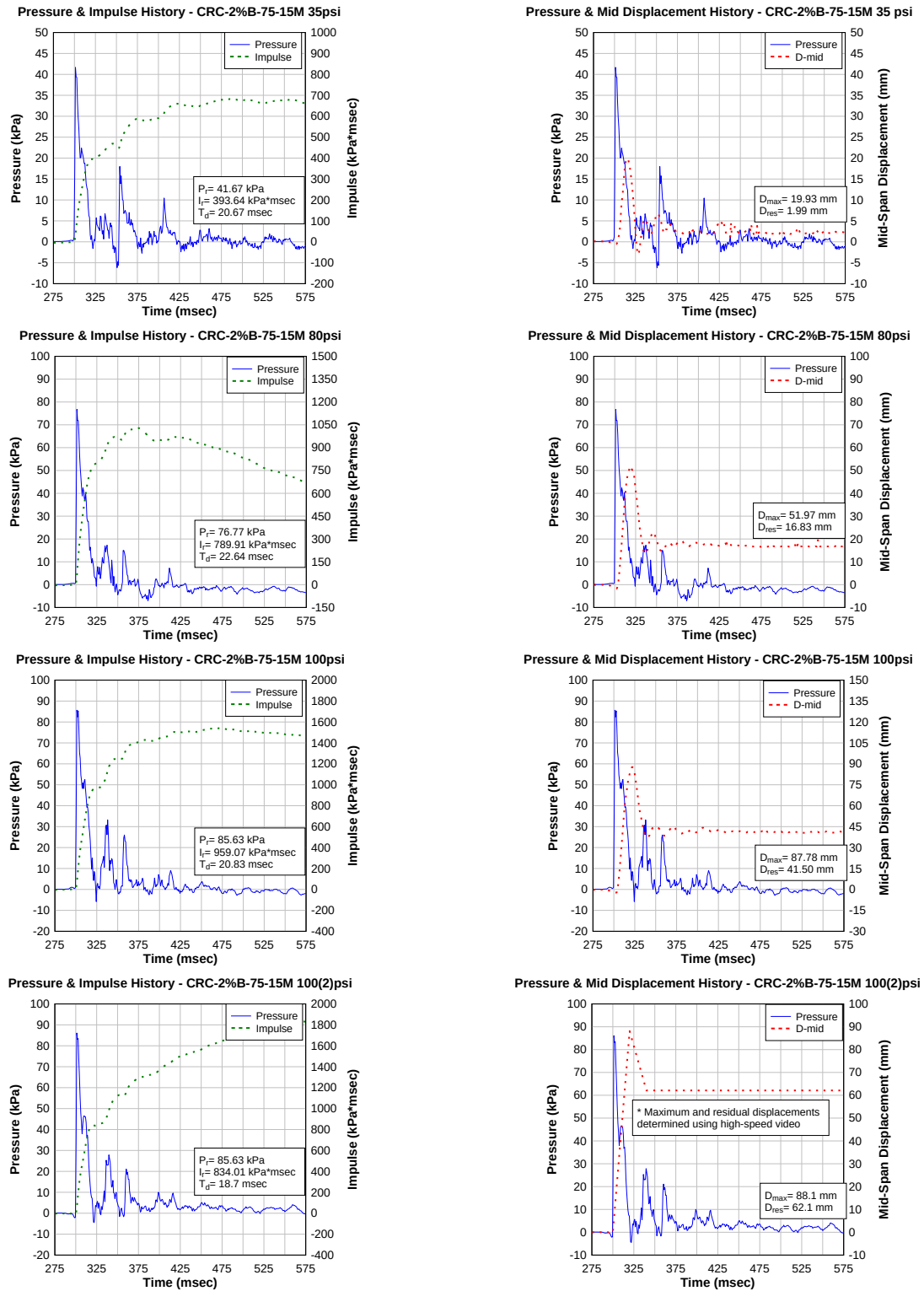
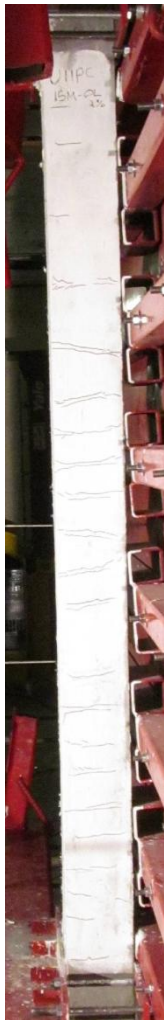


Figure 4.13(Left) Pressure and impulse time history, and (right) pressure and mid-span displacement time history for column CRC-2%B-75-15M



(a) Blast 35



(b) Blast 80

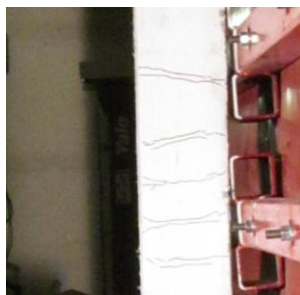


(c) Blast 100

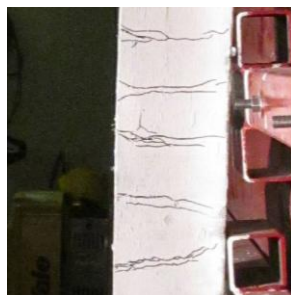


(d) Blast 100(2)

Figure 4.14 Column CRC-2%B-75-15M at various stages of testing



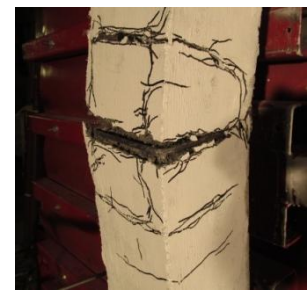
(a) Blast 35



(b) Blast 80



(c) Blast 100



(d) Blast 100(2)

Figure 4.15 Mid-span close up of column CRC-2%B-75-15M

4.5.6 CRC-2%C-75-15M

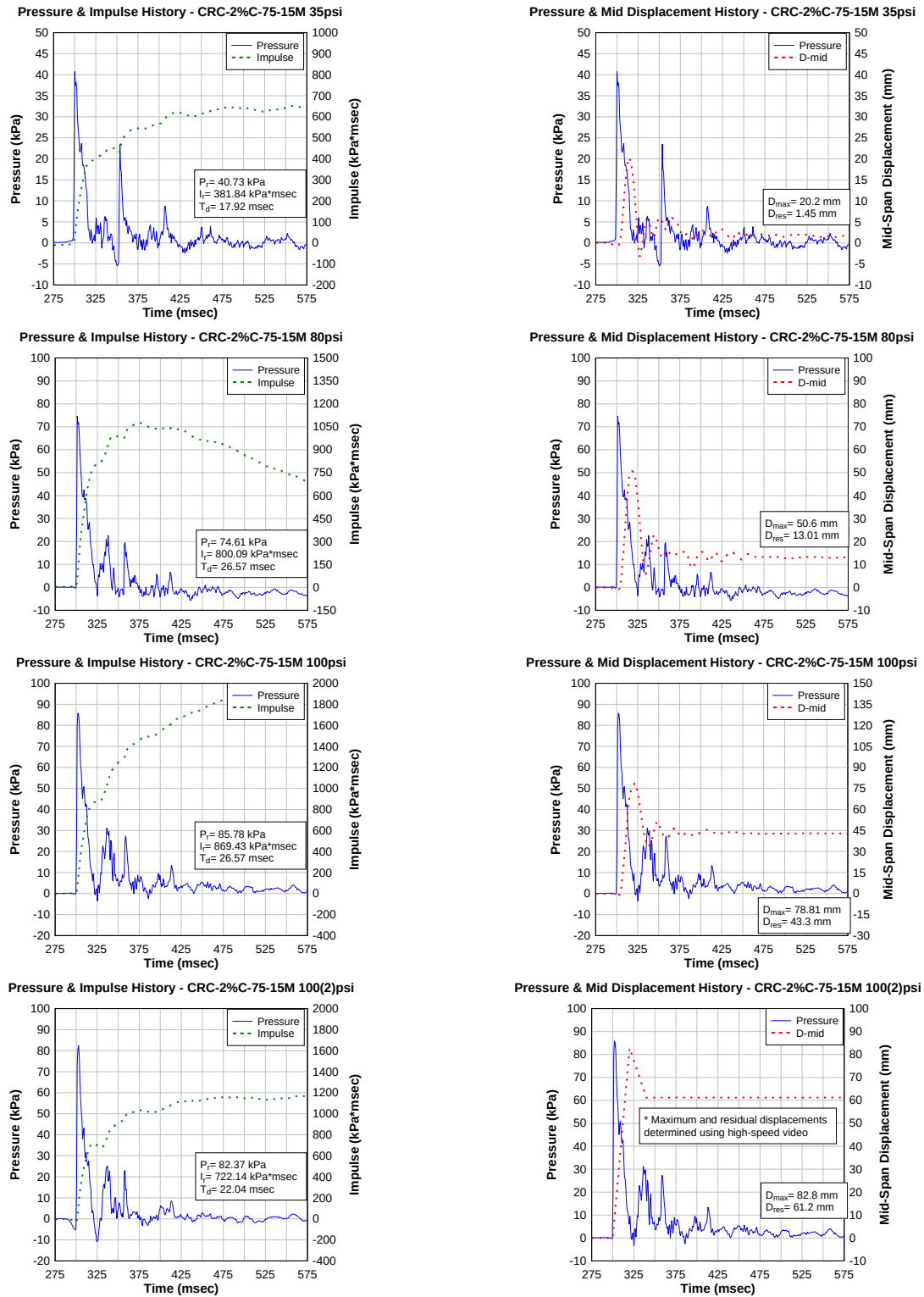
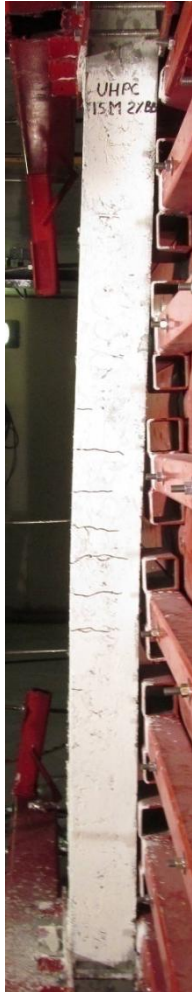


Figure 4.16 (Left) Pressure and impulse time history, and (right) pressure and mid-span displacement time history for column CRC-2%C-75-15M



(a) Blast 35



(b) Blast 80



(c) Blast 100



(d) Blast 100(2)

Figure 4.17 Column CRC-2%C-75-15M at various stages of testing



(a) Blast 35



(b) Blast 80



(c) Blast 100

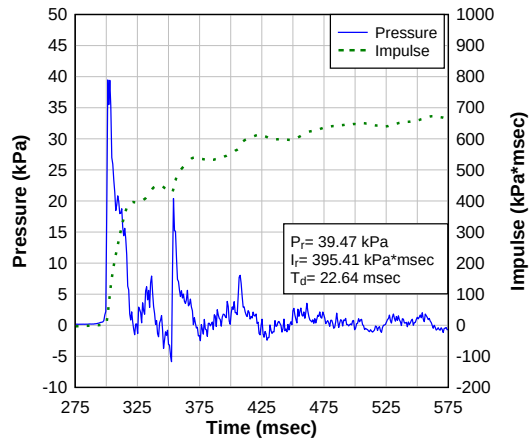


(d) Blast 100(2)

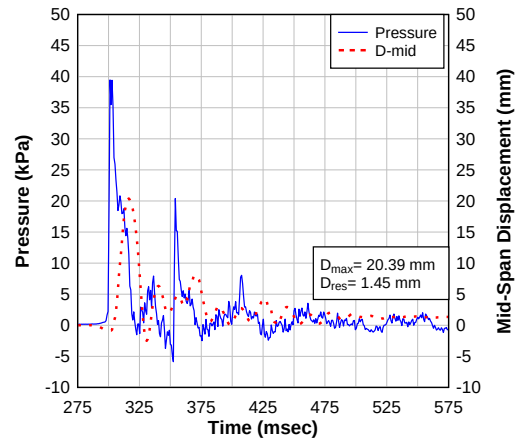
Figure 4.18 Mid-span close up of column CRC-2%C-75-15M

4.5.7 CRC-2%B-75-MMFX

Pressure & Impulse History - CRC-2%B-75-MMFX 35psi



Pressure & Mid Displacement History - CRC-2%B-75-MMFX 35 psi



Pressure & Impulse History -CRC-2%B-75-MMFX 100psi

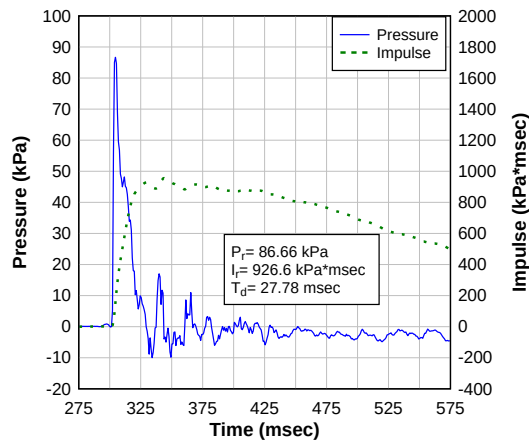


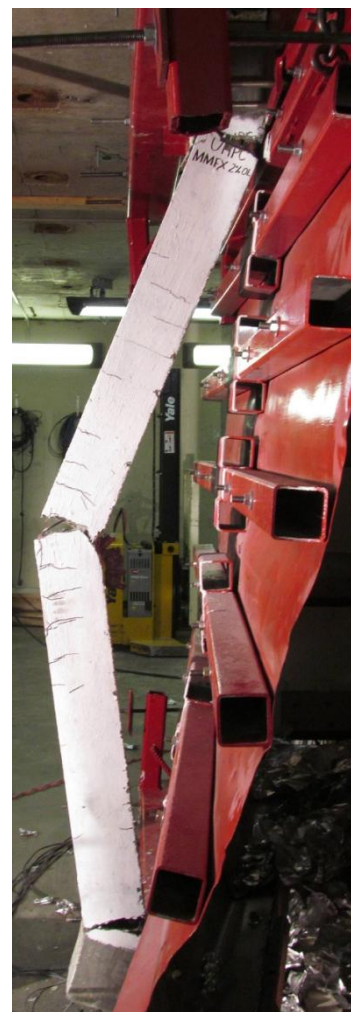
Figure 4.19 (Left) Pressure and impulse time history, and (right) pressure and mid-span displacement time history for column CRC-2%B-75-MMFX



(a) Blast 35



(b) Blast 80



(c) Blast 100

Figure 4.20 Column CRC-2%B-75-MMFX at various stages of testing



(a) Blast 35



(b) Blast 80

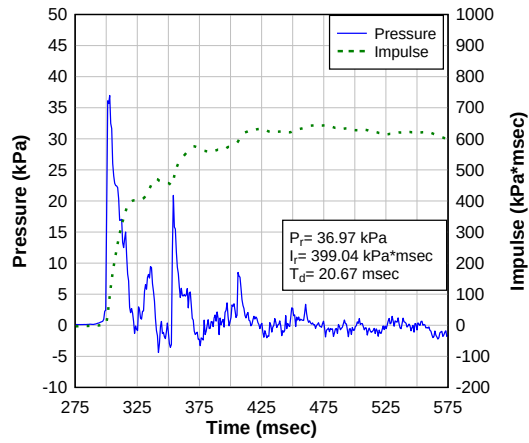


(c) Blast 100

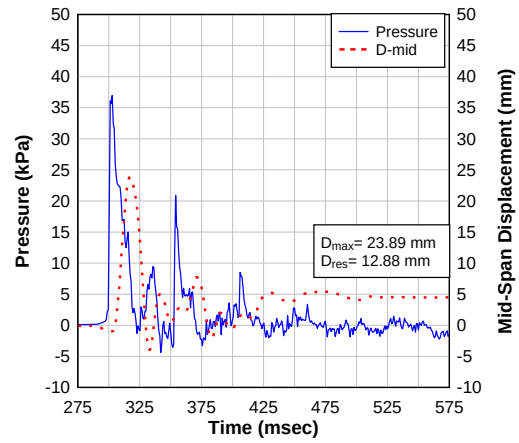
Figure 4.21 Mid-span close up of column CRC-2%B-75-MMFX

4.5.8 CRC-2%C-75-MMFX

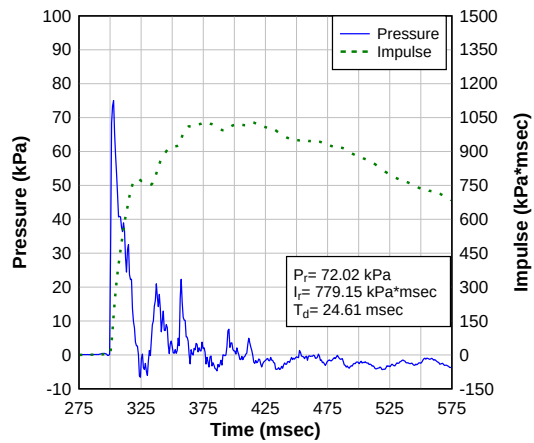
Pressure & Impulse History - CRC-2%C-75-MMFX 35psi



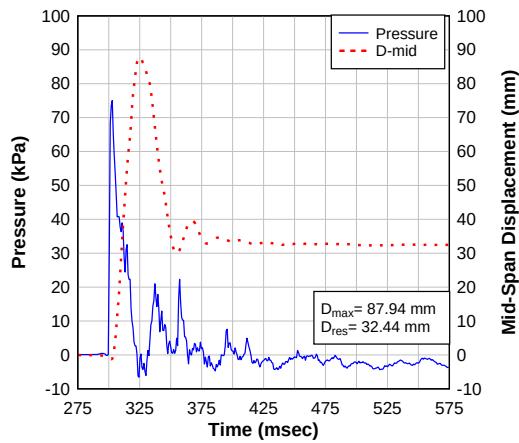
Pressure & Mid Displacement History - CRC-2%C-75-MMFX 35psi



Pressure & Impulse History - CRC-2%C-75-MMFX 80psi



Pressure & Mid Displacement History - CRC-2%C-75-MMFX 80psi



Pressure & Impulse History - CRC-2%C-75-MMFX 100psi

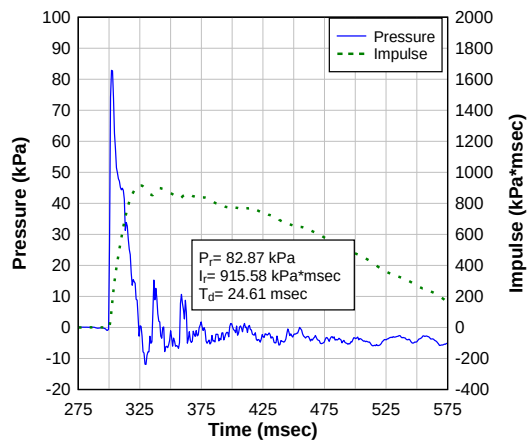


Figure 4.22 (Left) Pressure and impulse time history, and (right) pressure and mid-span displacement time history for column CRC-2%C-75-MMFX



(a) Blast 35



(b) Blast 80

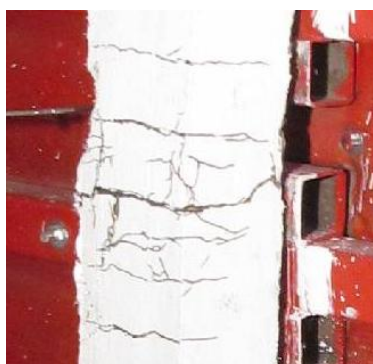


(c) Blast 100

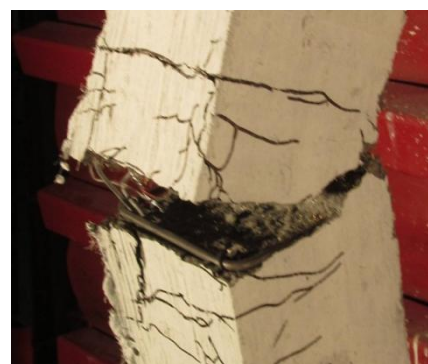
Figure 4.23 Column CRC-2%C-75-MMFX at various stages of testing



(a) Blast 35



(b) Blast 80

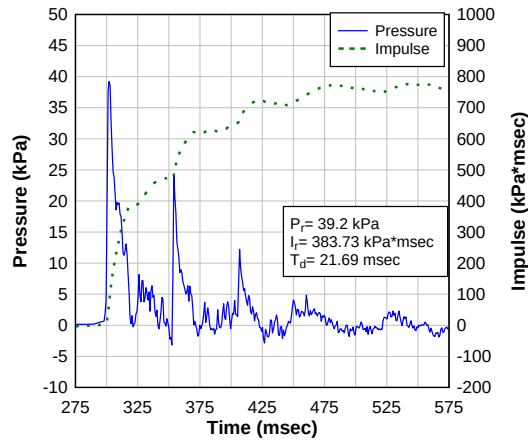


(c) Blast 100

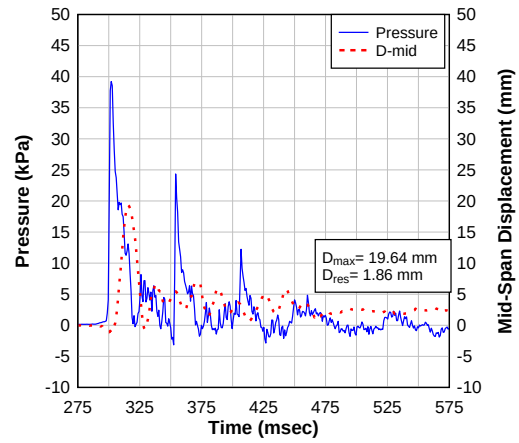
Figure 4.24 Mid-span close up of column CRC-2%C-75-MMFX

4.5.9 CRC-2%D-75-MMFX

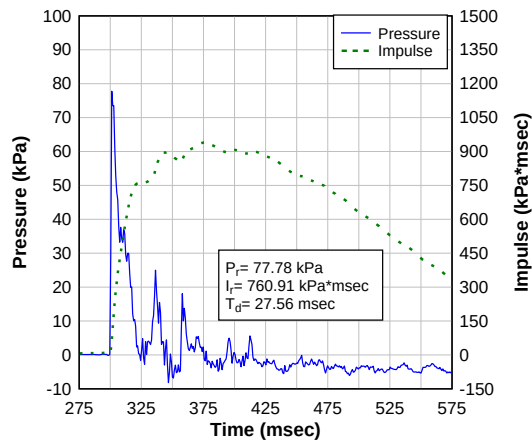
Pressure & Impulse History - CRC-2%D-75-MMFX 35psi



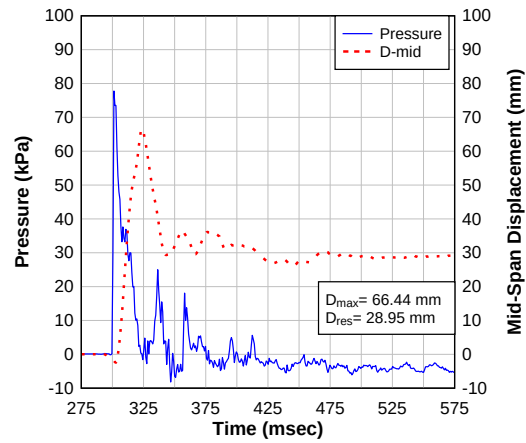
Pressure & Mid Displacement History - CRC-2%D-75-MMFX 35psi



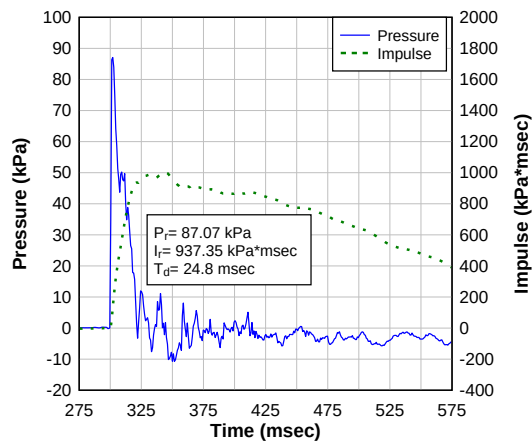
Pressure & Impulse History - CRC-2%D-75-MMFX 80psi



Pressure & Mid Displacement History - CRC-2%D-75-MMFX 80psi



Pressure & Impulse History - CRC-2%D-75-MMFX 100psi



Pressure & Mid Displacement History - CRC-2%D-75-MMFX 100psi

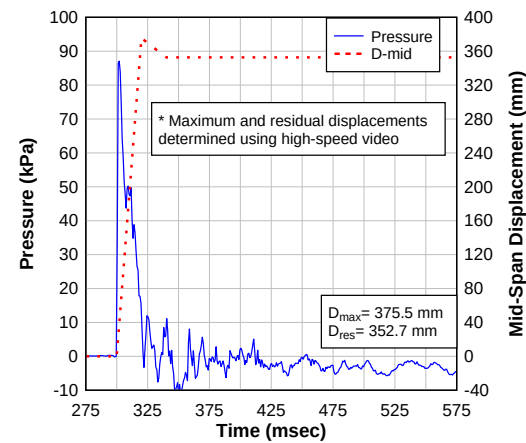


Figure 4.25 (Left) Pressure and impulse time history, and (right) pressure and mid-span displacement time history for column CRC-2%D-75-MMFX



(a) Blast 35

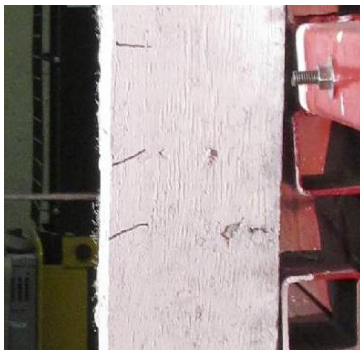


(b) Blast 80

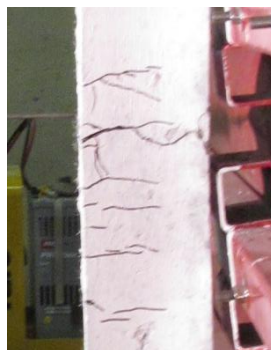


(c) Blast 100

Figure 4.26 Column CRC-2%D-75-MMFX at various stages of testing



(a) Blast 35



(b) Blast 80

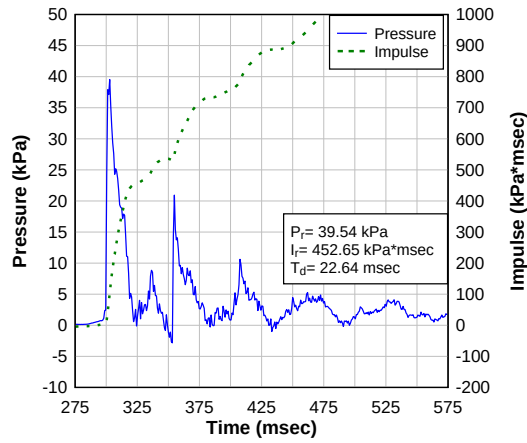


(c) Blast 100

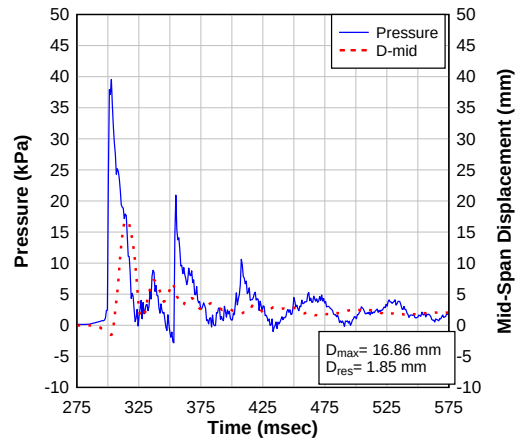
Figure 4.27 Mid-span close up of column CRC-2%D-75-MMFX

4.5.10 CRC-2%B-38-MMFX

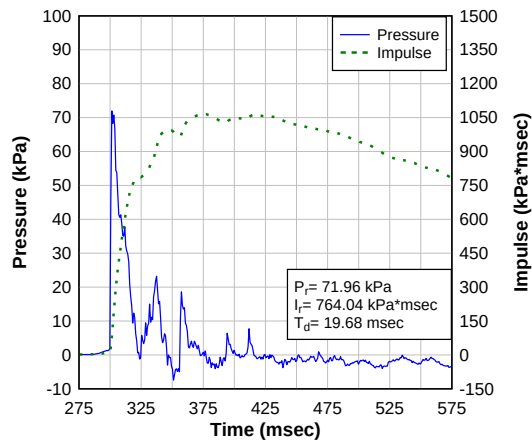
Pressure & Impulse History - CRC-2%B-38-MMFX 35psi



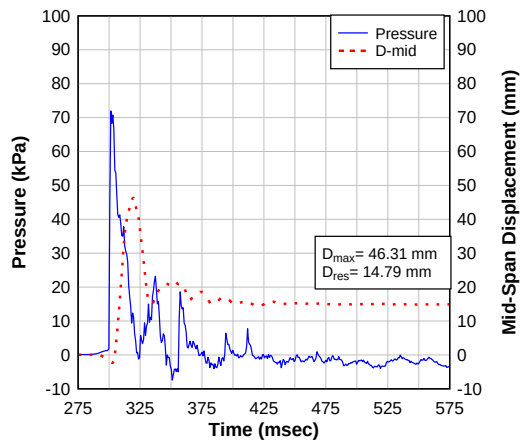
Pressure & Mid Displacement History - CRC-2%B-38-MMFX 35 psi



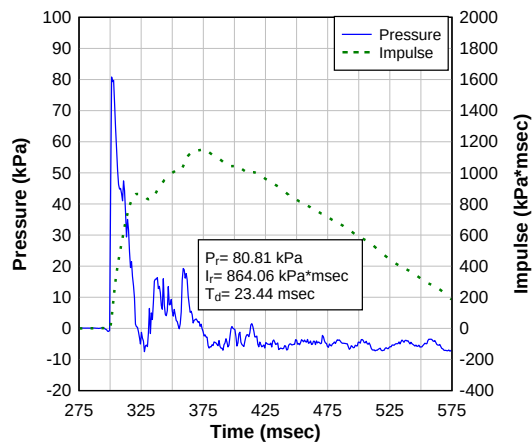
Pressure & Impulse History - CRC-2%B-38-MMFX 80psi



Pressure & Mid Displacement History - CRC-2%B-38-MMFX 80psi



Pressure & Impulse History - CRC-2%B-38-MMFX 100psi



Pressure & Mid Displacement History - CRC-2%B-38-MMFX 100psi

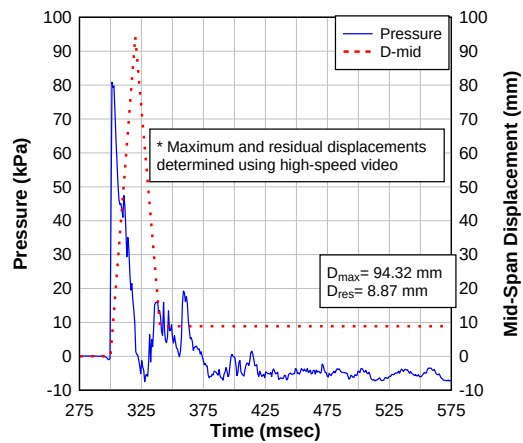


Figure 4.28 (Left) Pressure and impulse time history, and (right) pressure and mid-span displacement time history for column CRC-2%B-38-MMFX



(a) Blast 35



(b) Blast 80



(c) Blast 100

Figure 4.29 Column CRC-2%B-38-MMFX at various stages of testing



(a) Blast 35



(b) Blast 80

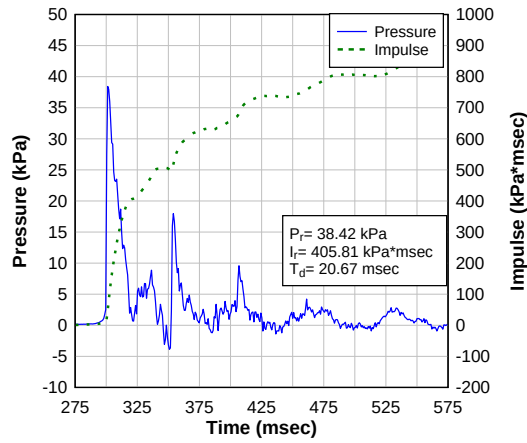


(c) Blast 100

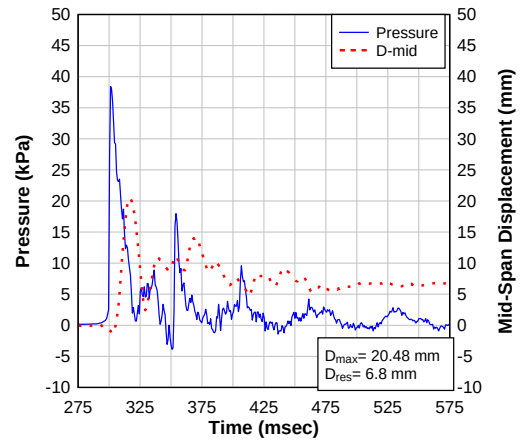
Figure 4.30 Mid-span close up of column CRC-2%B-38-MMFX

4.5.11 CRC-2%C-38-MMFX

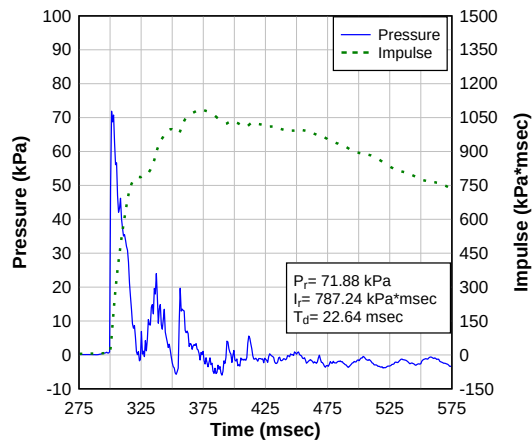
Pressure & Impulse History - CRC-2%C-38-MMFX 35psi



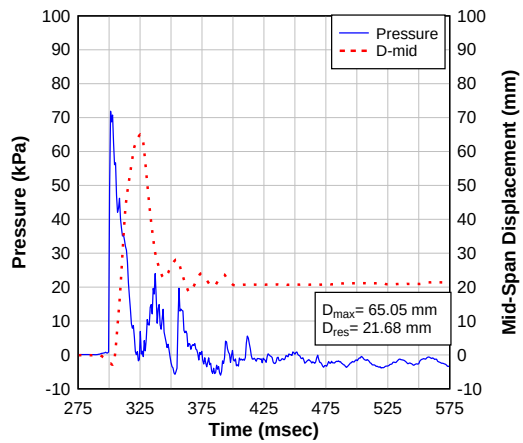
Pressure & Mid Displacement History - CRC-2%C-38-MMFX 35psi



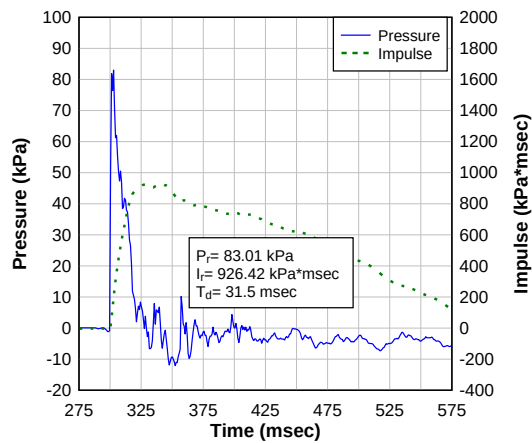
Pressure & Impulse History - CRC-2%C-38-MMFX 80psi



Pressure & Mid Displacement History - CRC-2%C-38-MMFX 80psi



Pressure & Impulse History - CRC-2%C-38-MMFX 100psi



Pressure & Mid Displacement History - CRC-2%C-38-MMFX 100psi

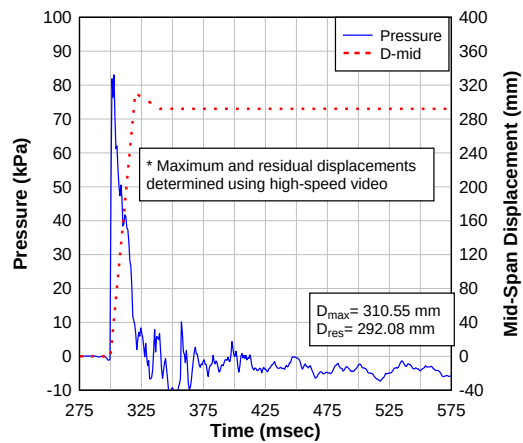
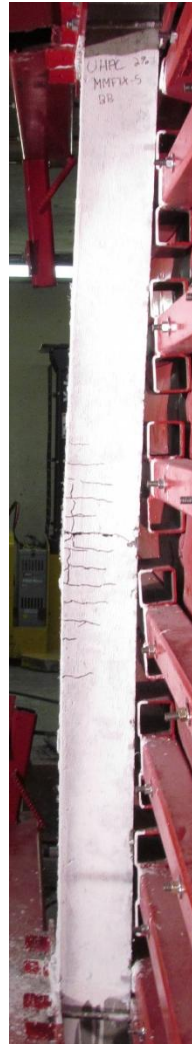


Figure 4.31 (Left) Pressure and impulse time history, and (right) pressure and mid-span displacement time history for column CRC-2%C-38-MMFX



(a) Blast 35



(b) Blast 80

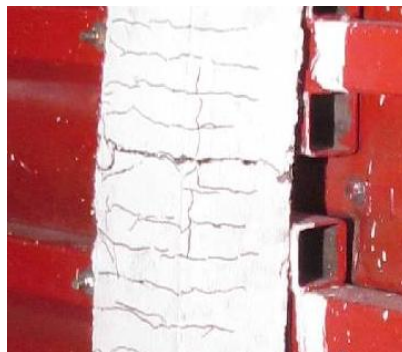


(c) Blast 100

Figure 4.32 Column CRC-2%C-38-MMFX at various stages of testing



(a) Blast 35



(b) Blast 80

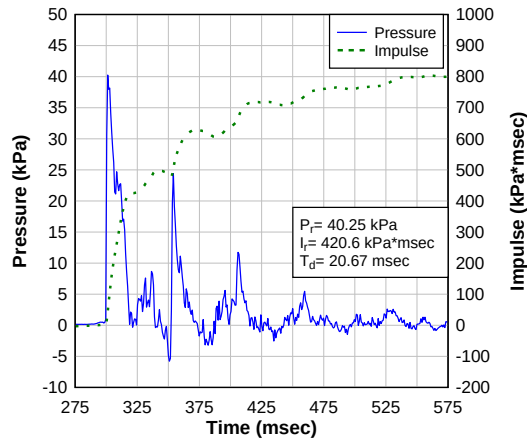


(c) Blast 100

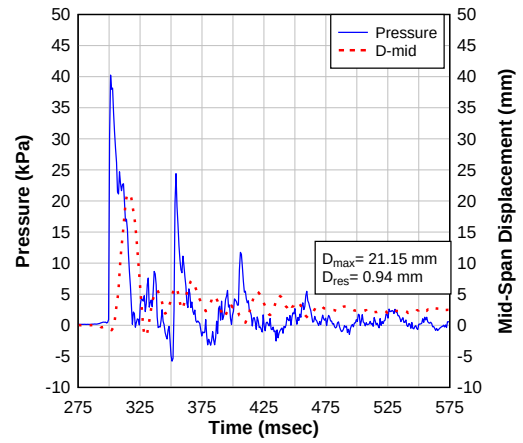
Figure 4.33 Mid-span close up of column CRC-2%C-38-MMFX

4.5.12 CRC-3%C-75-MMFX

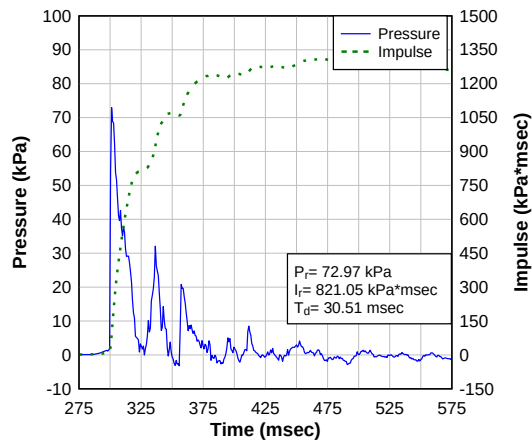
Pressure & Impulse History - CRC-3%C-75-MMFX 35psi



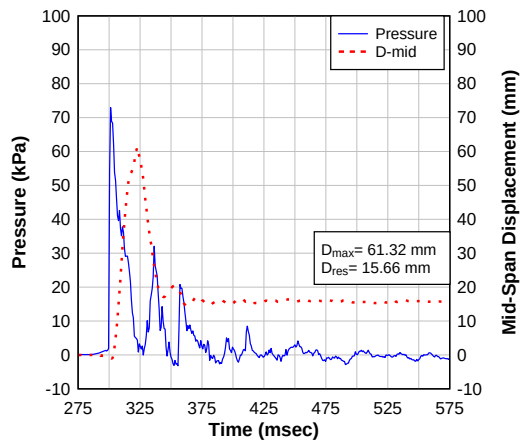
Pressure & Mid Displacement History - CRC-3%C-75-MMFX 35psi



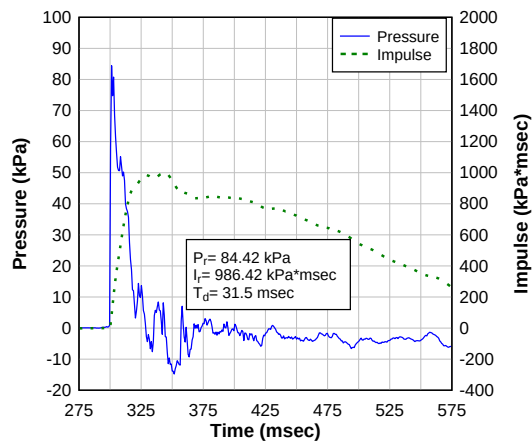
Pressure & Impulse History - CRC-3%C-75-MMFX 80psi



Pressure & Mid Displacement History - CRC-3%C-75-MMFX 80psi



Pressure & Impulse History - CRC-3%C-75-MMFX 100psi



Pressure & Mid Displacement History - CRC-3%C-75-MMFX 100psi

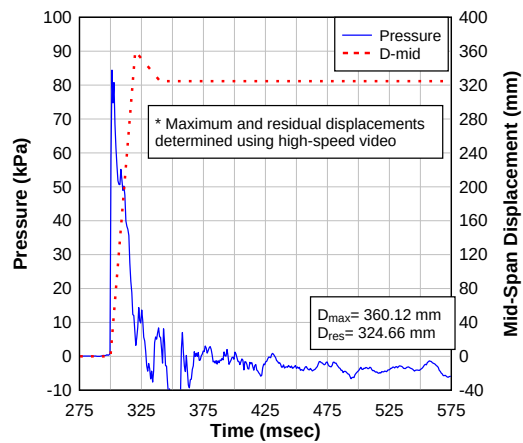
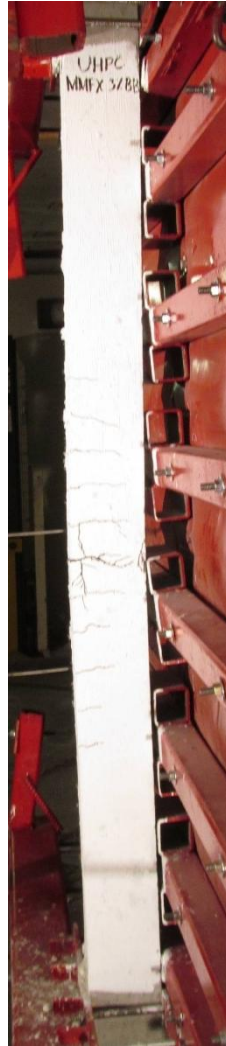


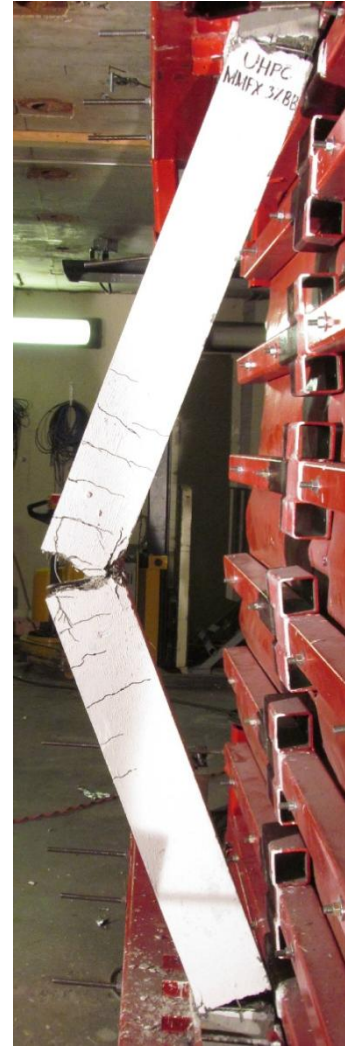
Figure 4.34 (Left) Pressure and impulse time history, and (right) pressure and mid-span displacement time history for column CRC-3%C-38-MMFX



(a) Blast 35

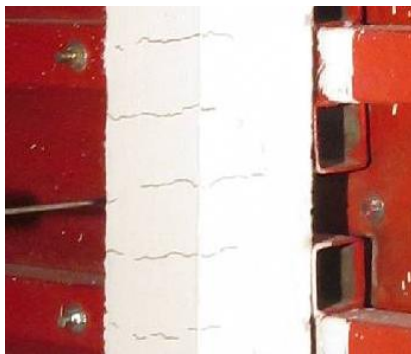


(b) Blast 80



(c) Blast 100

Figure 4.35 Column CRC-3%C-38-MMFX at various stages of testing



(a) Blast 35



(b) Blast 80



(c) Blast 100

Figure 4.36 Mid-span close up of column CRC-3%C-38-MMFX

5 Discussion

5.1 Section Overview

This section will discuss and compare the results obtained during the experimental program. The effects of concrete type, fiber properties, fiber content, seismic detailing, longitudinal reinforcement ratio and longitudinal reinforcement strength on maximum and residual displacements, damage tolerance and failure mode are all considered. Section 5.2 will discuss the results from the Series 1 specimens, while section 5.3 will discuss the results from Series 2.

In order to highlight the effect of the test variables, the results from this research program are compared to results from five other columns tested by other researchers (see Table 5.1):

- SCC-0%-75 had similar properties to the non-seismic 10M columns in Series 1, but was constructed with normal-strength SCC concrete (Burrell, 2012);
- SCC-0%-38 had similar properties to the seismic 10M columns in Series 1, but was constructed with normal-strength SCC concrete (Burrell, 2012);
- CRC-2%A-75 had similar properties to the non-seismic 10M columns in Series 1, but was constructed with fiber type A (Burrell, 2012);
- CRC-2%A-38 had similar properties to the seismic 10M columns in Series 1, but was constructed with fiber type A (Burrell, 2012);
- CRC-2%B-75-#3 was constructed and tested subsequent to this study and allows for an investigation into the effect of reinforcement strength in UHPFRC columns having the same reinforcement ratio (De Carufel, 2015);

No data was recorded for column CRC-2%B-75-MMFX due to computer malfunction, and therefore the results from this specimen are not discussed in this section.

Table 5.1 Specimen tested by other researchers used for comparison in discussion

Series	Columns	Concrete Mix	Cross-Section	Transverse Reinforcements Spacing (s)	Transverse Reinforcing Steel Ties	Fibre Type	Fibre Content (%)
Comparison Specimens	SCC-0%-75*	SCC	152.4 mm X 152.4 mm	75 mm	4-10M bars	N/A	0%
	SCC-0%-38*			38 mm			
	CRC-2%A-75*	CRC		75 mm		A	2%
	CRC-2%A-38*			38 mm			
	CRC-2%B-75-#3**			75 mm	4-#3 American	B	

* Tested by Burrell (2012)

** Tested by De Carufel (2015)

5.2 Series 1 – Regular Grade Steel

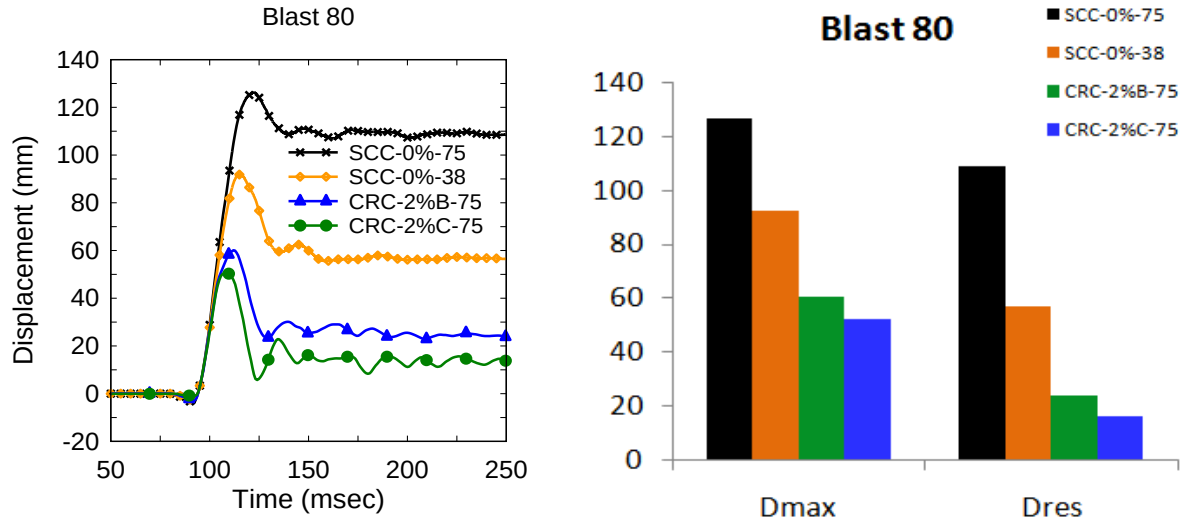
This section discusses the results from Series 1. For comparison, maximum and residual displacements at all Blasts for all columns in this series, along with identification of failure mode, are summarized in Figure 9.1. To investigate the effect of the test parameters, displacement time-histories at Blast 80 and 100 pressures are compared in Figures following each effect of parameters discussed. Photos of the columns after the application of Blasts 80, 100 and 100(2) are shown in Figure 5.12. Sample photos of failure mode and high-speed video stills are presented in Figure 5.13 and Figure 5.14.

5.2.1 Effect of Concrete Type

Columns SCC-0%-75, CRC-2%B-75 and CRC-2%C-75 had identical properties (cross-section, longitudinal reinforcement and hoop spacing), but were constructed with plain SCC and CRC having 2% steel fibers by volume of concrete, respectively. It is noted that column SCC-0%-75 was tested by Burrell (2012). Both of the CRC columns contained fibres with different properties that will be compared later on. Blast 35, was intended to test the columns at yield conditions, resulted in some limited cracking of concrete in column SCC-0%-75, with maximum and residual displacements of $D_{\max} = 29.9$ mm & $D_{\text{res}} = 9.7$ mm (Burrell, 2012). In comparison, no damage was observed in the companion CRC columns, with reduced displacements of $D_{\max} = 17.6$ mm & $D_{\text{res}} = 4.5$ mm for CRC-2%B-75 and $D_{\max} = 15.6$ mm & $D_{\text{res}} = 1.1$ mm for CRC-2%C-75, under the same blast pressures (see Figure 5.1). Failure of column SCC-0%-75 occurred after Blast 80 loading due to compression rebar buckling. This blast caused maximum and residual displacements of $D_{\max} = 126$ mm and $D_{\text{res}} = 108$ mm, and resulted in significant

crushing of concrete on the compression side, the formation of diagonal cracks at mid-span and significant spalling of cover concrete (see Figure 5.12a). In contrast, damage was relatively minor and compression rebar buckling was prevented in the companion CRC columns after Blast 80 (see Figure 5.12d). Figure 5.1 compares the response of specimens SCC-0%-75, CRC-2%B-75 and CRC-2%C-75 under Blast 80 loading and shows that CRC significantly improves blast resistance by reducing maximum and residual displacements. Columns CRC-2%B-75 and CRC-2%C-75 showed displacements of $D_{\max}=60.1\text{mm}$ & $D_{\text{res}}=23.9\text{mm}$, and $D_{\max}=55.75\text{ mm}$ & $D_{\text{res}}=12.4\text{ mm}$, resulting in reductions of 52% & 78%, and 56% & 89% respectively when compared to the control SCC column. The failure mechanism of the CRC columns was tension rebar rupture after Blast 100 (see Figure 5.12d and e). It is also noted that the CRC specimens were able to not only match but better control maximum and residual displacements when compared to column SCC-0%-38, which had seismic detailing and was tested by Burrell (2012); Figure 5.1 also compares the response of SCC-0%-38 with the CRC columns at Blast 80 loading.

The results indicate clear benefits associated with the use of UHPFRC in columns subjected to blast loads. When compared to conventional concrete, CRC has high compressive strength and improved tensile resistance, with the development of significant post-cracking strength due to the provision of steel fibers. Recent research also indicates that provision of fibers in CRC also allows for improved confinement when compared to conventional concrete Hosinie (2014). Thus the increased compressive strength, enhanced tensile resistance, improved confinement and increased damage tolerance in CRC lead to increased stiffness and improved flexural capacity in columns tested under combined axial loads and lateral blast pressures.



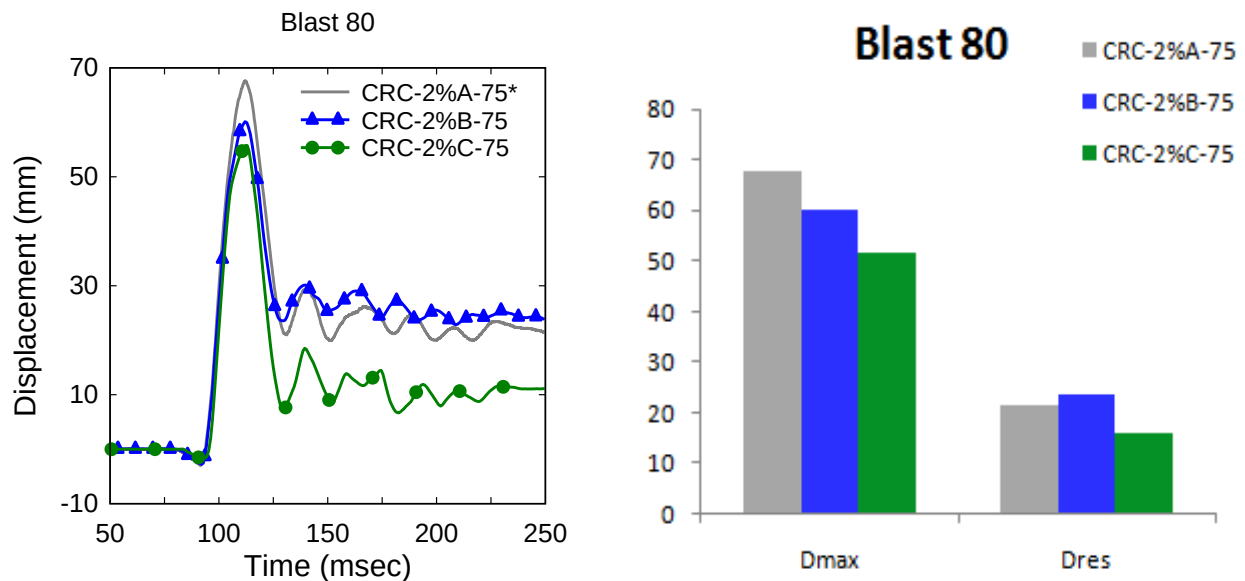
*Columns tested by Burrell, 2012

Figure 5.1 Effect of concrete type: comparison of response at Blast 80

5.2.2 Effect of Fibre Type

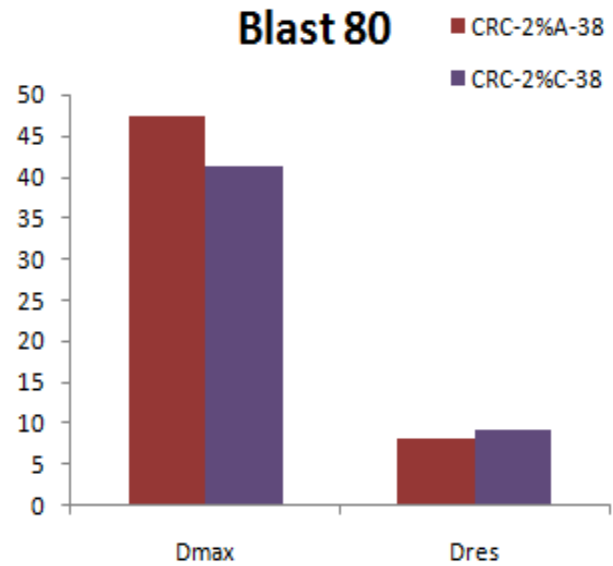
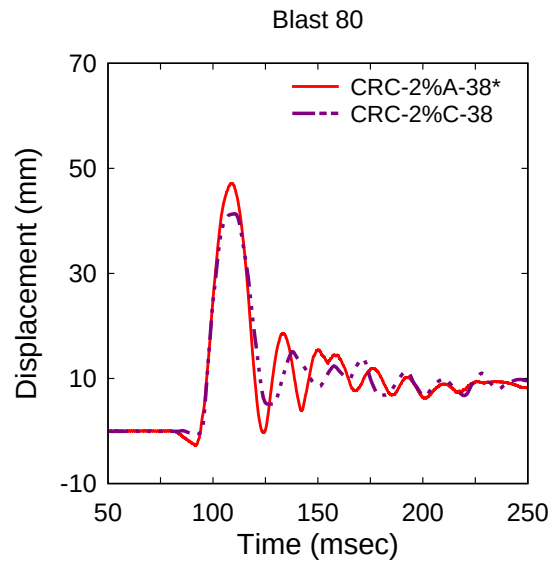
The effect of fiber type on the blast performance of the CRC columns can be studied by comparing the response of columns CRC-2%A-75, CRC-2%B-75 and CRC-2%C-75, which had 2% of fiber types A, B and C, respectively. It is noted that column CRC-2%A-75 was tested by Burrell (2012). The response of the specimens under Blast 80 loading is shown in Figure 5.2. It is seen that the CRC column response improves as the fiber properties are enhanced. Column CRC-2%A-75 shows maximum and residual displacements $D_{\max} = 67.8$ mm and $D_{\text{res}} = 21.7$ mm under Blast 80, while column CRC-2%B-75 shows a lower maximum displacement of $D_{\max} = 60.1$ mm, with a similar residual displacement of $D_{\text{res}} = 23.9$ mm. It is noted that when compared to fiber type A, fiber type B had a twofold increase in aspect-ratio (60 vs. 30), with fiber tensile strength increased by a factor of approximately 50% (2000 MPa vs. 1350 MPa). The use of fiber type C, which had tensile strength of 3150 MPa (an increase of 133% when compared to fiber type A), resulted in maximum and residual displacements of $D_{\max} = 55.75$ mm and $D_{\text{res}} = 12.4$ mm in column CRC-2%C-75, reductions of 18% and 43% when compared to column CRC-2%A-75. While all three columns failed due to rupture of tension steel under application of Blast 100 pressures, the failure mode of column CRC-2%C-75 was noticeably more controlled when compared to the two companion columns constructed with fiber types A and B (compare failure photos in Figure 5.12c, d & e). The effect of fiber type can further be examined by comparing the

response of columns CRC-2%A-38 and CRC-2%C-38, which had seismic detailing, and 2% of fiber types A and C, respectively. It is noted that column CRC-2%A-38 was tested by Burrell (2012). Under Blast 80 loading, column CRC-2%C-38 shows a reduction in maximum displacement by a factor of approximately 12% when compared to column CRC-2%A-38 (see Figure 5.3). Likewise, the effect of fibre can also be observe comparing specimen CRC-2%B-75-15M and CRC-2%C-75-15M. At Blast 80, both of these column showed to have similar results while at Blast 100, the column contain fibre type C showed a maximum mid-span displacement reduction of approximately 10% (see Figure 5.4, Figure 5.5). Finally, the last observation that should be noted is the comparison of specimen CRC-2%B-38 and CRC-2%C-38 at Blast 100. Both of these columns resulted in tensile rebar rupture due to the blast pressures but the specimen containing fibre type C had a much more controlled failure (see Figure 5.6). In summary, the results from the limited number of tests indicate that the use of fibers with enhanced properties results in better control of displacements in columns tested under equivalent blast loads. It is noted that the use of high fiber contents can result in difficulties in concrete placement; the use of fibers with optimized properties may be a more practical and cost-effective method for improving the blast performance of UHPFRC columns - further research is recommended.



*Columns tested by Burrell, 2012

Figure 5.2 Effect of fiber type (1): comparison of response at Blast 80



*Columns tested by Burrell, 2012

Figure 5.3 Effect of fiber type (2): comparison of response at Blast 80

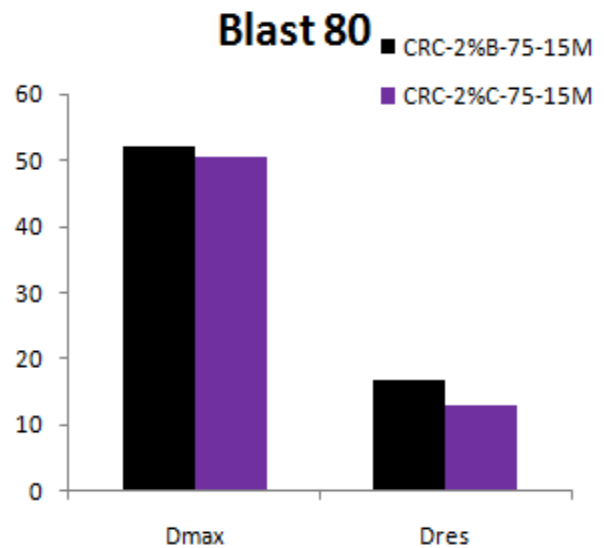
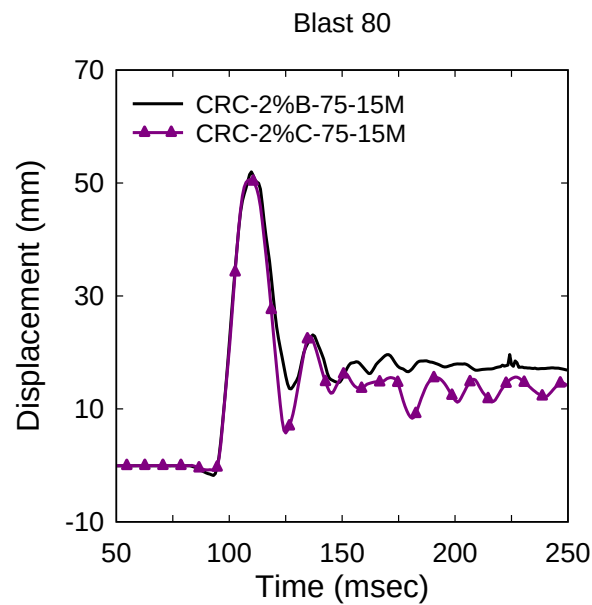


Figure 5.4 Effect of fiber type (3): comparison of response at Blast 80

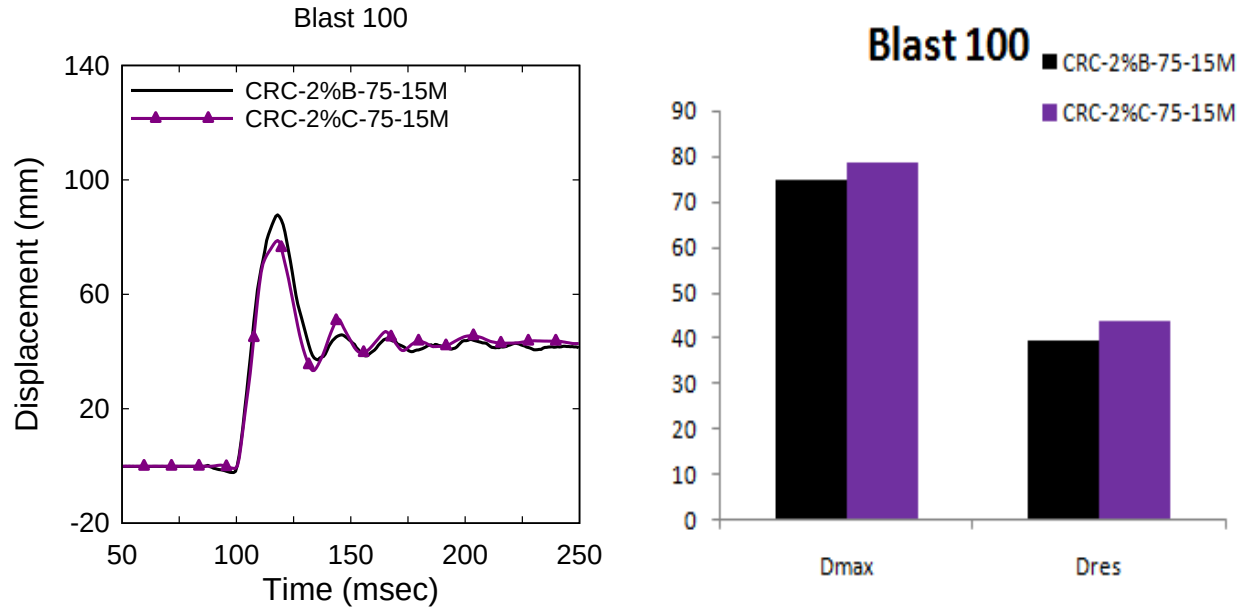
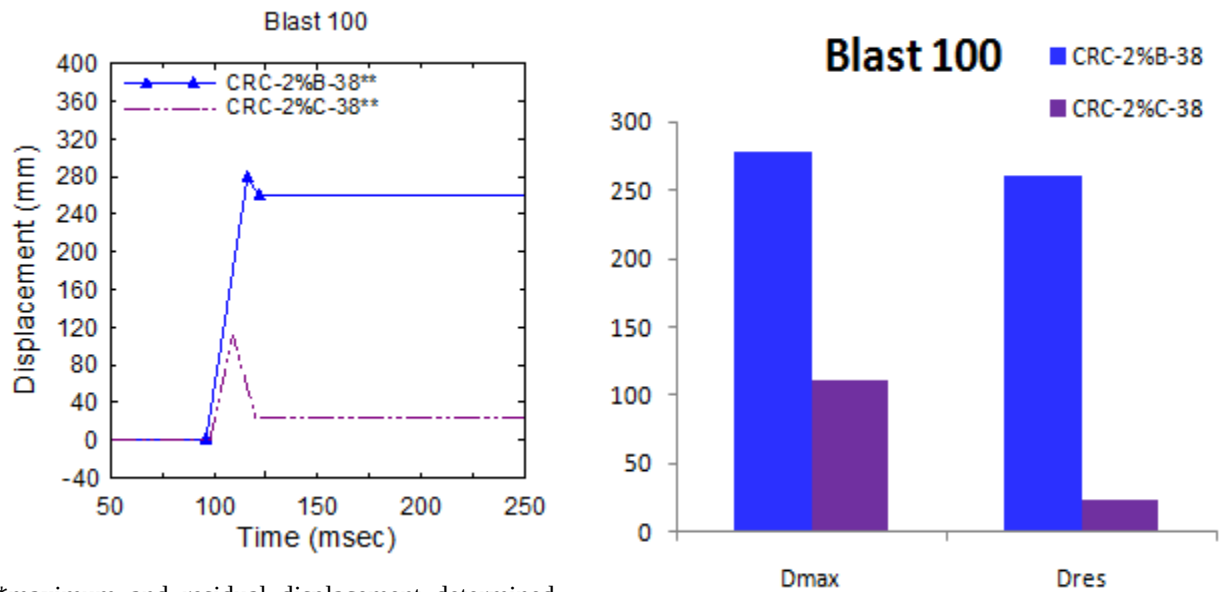


Figure 5.5 Effect of fiber type (4) : comparison of response at Blast 100



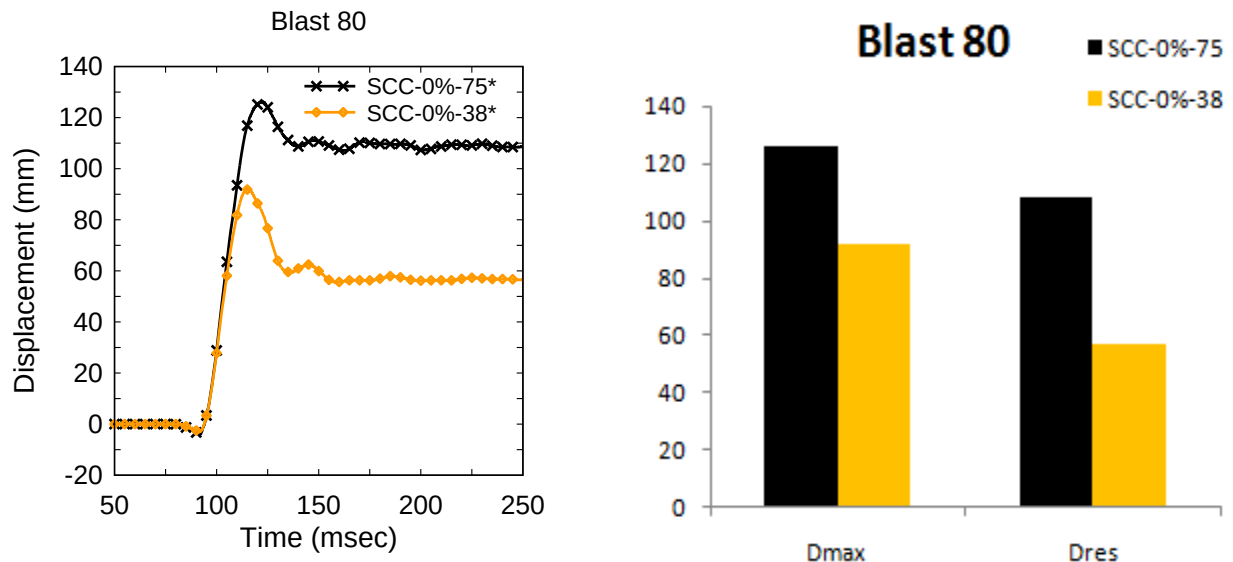
**maximum and residual displacement determined using high-speed video

Figure 5.6 Effect of fiber type (5): comparison of response at Blast 100

5.2.3 Effect of Seismic Detailing

Previous researchers have shown that the reduction of transverse reinforcement spacing in conventional concrete columns can improve performance under blast loading. Figure 5.7 shows the behaviour of two companion specimens tested in Burrell's (2012) research program. The

columns were built with conventional concrete and reinforced with non-seismic and seismic detailing; it can be seen that displacements are better controlled as the transverse reinforcement spacing is reduced from $s = 75$ to 38 mm. The benefit of combined use of CRC and seismic detailing is highlighted in this research program by comparing the responses of columns CRC-2%C-38 and CRC-2%C-75 after Blast 80. When compared, the maximum and residual displacements for the seismically detailed CRC column are decreased by factors of approximately 26% and 26%, with $D_{\max} = 55.75$ mm & 41.36 mm, and $D_{\text{res}} = 12.4$ mm & 9.23 mm for columns CRC-2%C-75 and CRC-2%C-38, respectively (see Figure 5.8). . The results from the limited tests conducted in the investigation indicates that the use of seismic detailing in UHPFRC columns results in better control of displacements, and an ability to sustain larger blast loads. Provision of closely spaced transverse reinforcement improves UHPFRC confinement, enhancing strength, ductility and integrity of core concrete, which can result in enhanced column moment capacity and increased flexural deformability under blast loads. It can be concluded that under blast loads, improvements in UHPFRC column performance from seismic detailing are possible if the behaviour is dominated by combined flexure and axial compression, however further research is required to investigate the effect of such detailing in columns that are susceptible to brittle shear failure modes.



*Columns tested by Burrell, 2012

Figure 5.7 Effect of seismic detailing (1) : comparison of response at Blast 80

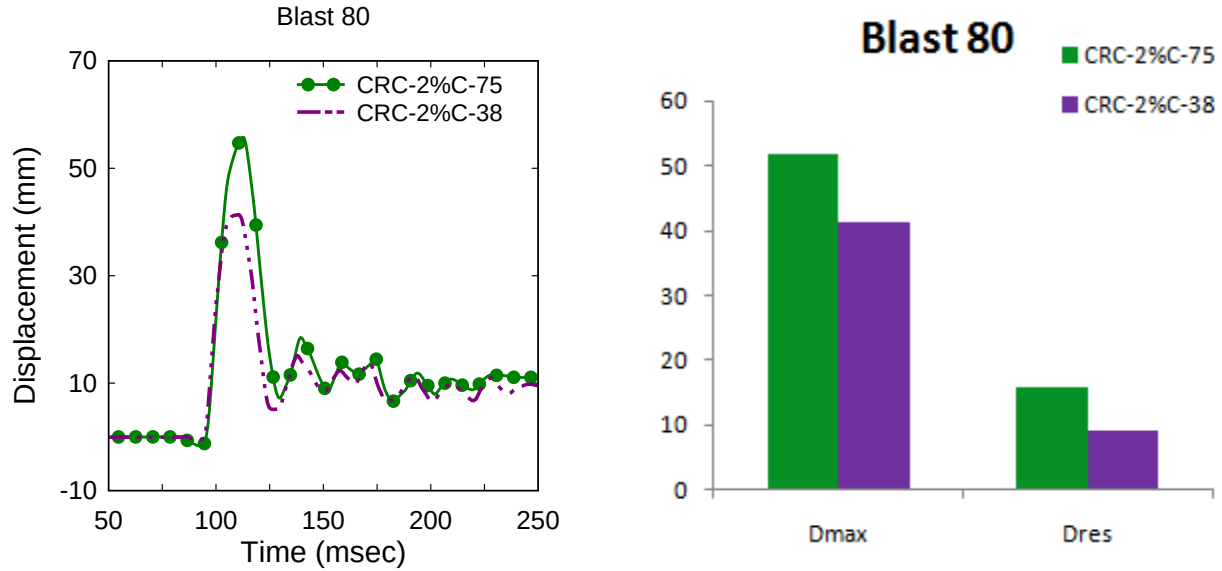


Figure 5.8 Effect of seismic detailing (2) : comparison of response at Blast 80

5.2.4 Effect of Longitudinal Reinforcement ratio

The effect of longitudinal reinforcement ratio was the last parameter investigated in this series. Two specimens containing 4 - 15M bars (CRC-2%B-75-15M and CRC-2%C-75-15M) were tested alongside specimens having identical properties but containing 4 - 10M bars (CRC-2%B-75 and CRC-2%C-75). No clear trend is seen when comparing the response of the companion columns with 15M and 10M reinforcement under Blast 35 loads. Under Blast 80 pressures, the maximum and residual displacements were reduced by factors of approximately 9% and 30% for column CRC-2%B-75-15M ($D_{\max} = 52$ mm and $D_{\text{res}} = 16.8$ mm) when compared to column CRC-2%B-75 ($D_{\max} = 60.1$ mm and $D_{\text{res}} = 23.9$ mm) (see Figure 5.9). The differences in displacements are less important when comparing the response of columns CRC-2%C-75-15M and CRC-2%C-75 which shows similar response at Blast 80 (see Figure 5.10). The benefit of larger reinforcement ratio is more evident when examining the failure mode of the columns. As noted previously failure of the CRC columns having 2% fibers and 10M reinforcement (columns CRC-2%B-75, CRC-2%C-75) occurred under Blast 4 loading due to rupture of tension steel. The provision of the larger 15M bars in columns CRC-2%B-75-15M and CRC-2%C-75-15M prevented rupture of tension steel, and allowed the columns to survive a second application of blast pressures corresponding to Blast 100 (see Figure 5.12i & Figure 5.12j). No complete displacement-time history data is available for columns CRC-2%B-75 and CRC-2%C-75 at Blast 100 due to failure of the LVDTs, however Figure 5.11 compares the response of columns CRC-

2%B-75-15M and CRC-2%B-75 at this loading and illustrates the important improvement in blast resistance in the column containing 15M reinforcement. The limited results from this research program indicate that an increase in longitudinal reinforcement is an effective means of preventing premature rupture of tension steel in UHPFRC columns, which can in turn lead to increased blast capacity - further research is recommended.

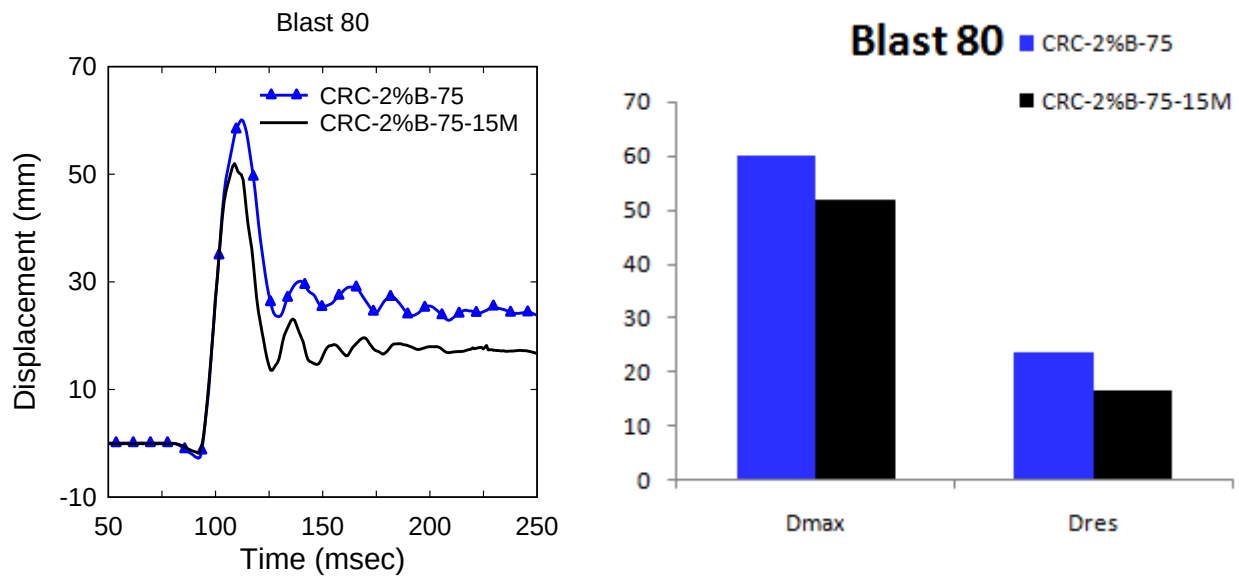


Figure 5.9 Effect of 15M bars (1) : comparison of response at Blast 80

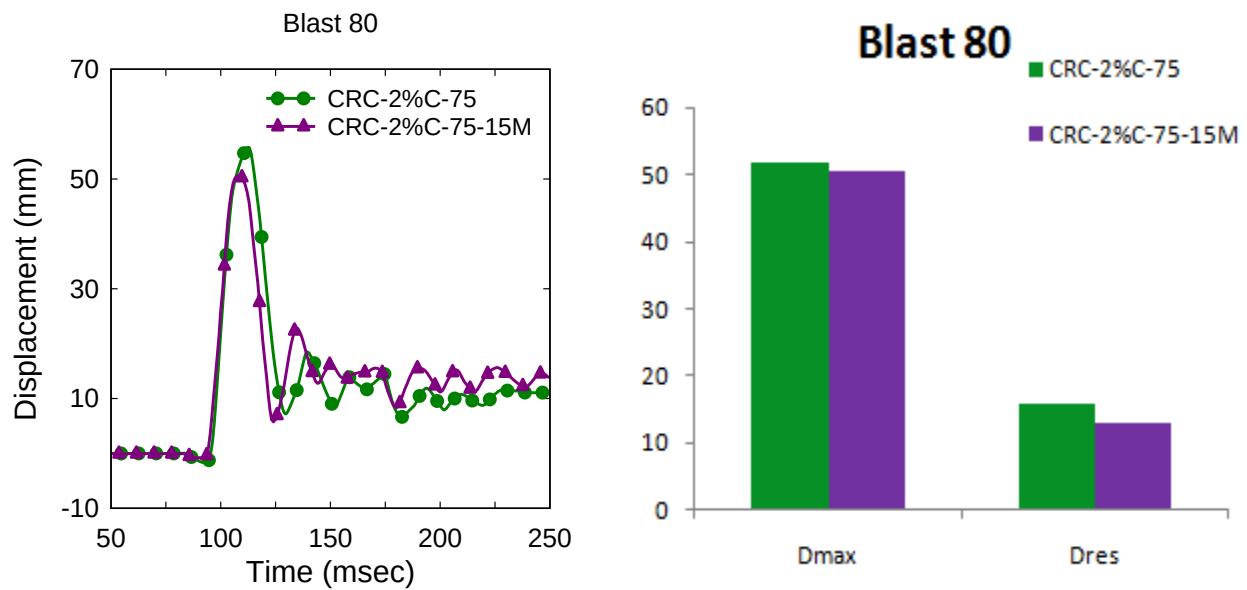
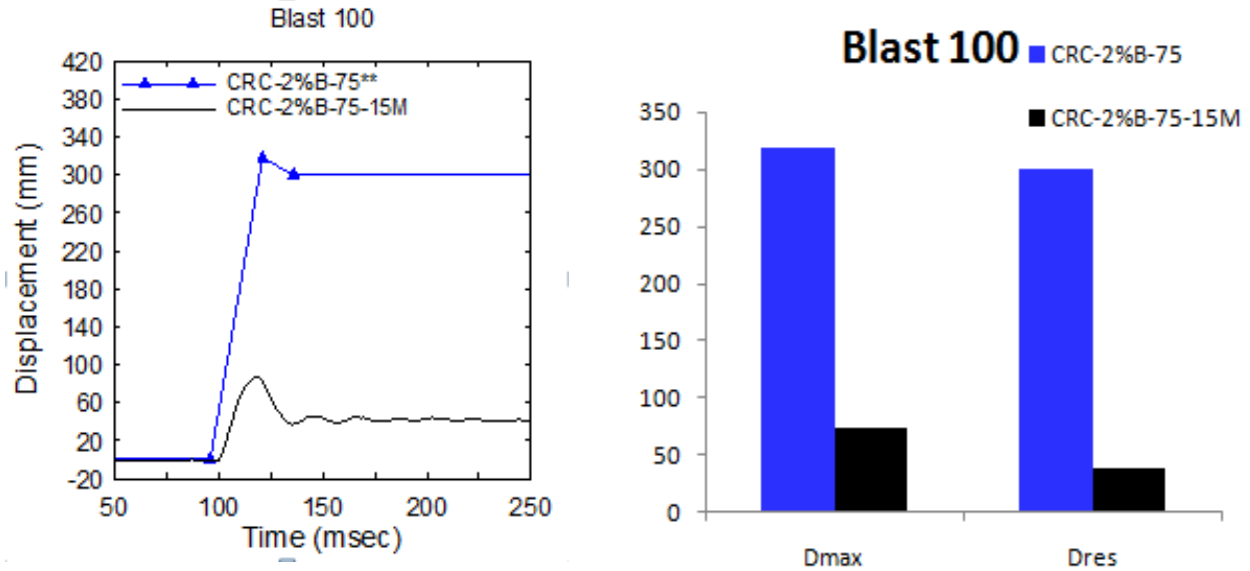


Figure 5.10 Effect of 15M bars (2) : comparison of response at Blast 80



**maximum and residual displacement determined using high-speed video

Figure 5.11 Effect of 15M bars (3) : comparison of response at Blast 100

5.2.5 Effect of UHPFRC on Damage Tolerance and Failure Mode

The use of UHPFRC had an important effect on damage tolerance and secondary blast fragments. When compared to the control concrete specimens in Burrell's (2012) testing program, the UHPFRC columns showed enhanced control of cracking and spalling, and the ability to eliminate flying debris. Figure 5.13a shows crushing of compression concrete, buckling of compression bars, spalling of cover concrete and significant damage in column SCC-0%-75 after the application of Blast 80. In contrast, the companion UHPFRC columns showed essentially no damage at this blast (see Figure 5.12c, d & e). Figure 5.14a and Figure 5.14b compare high-speed video stills of columns SCC-0%-75 and CRC-2%B-75 after Blast 80; it is noted that while the SCC column shows significant fragmentation, the UHPFRC column shows no secondary blast fragments. Although no video is available for columns CRC-2%B-75 or CRC-2%C-75 at Blast 100, Figure 5.14c and Figure 5.14d compare high-speed video stills of columns SCC-0%-38 and CRC-2%A-75 after Blast 100 which were tested by Burrell (2012). It is noted that even at failure, the only noted secondary fragments for the UHPFRC column are in the form of a fine powder, in contrast to the significant fragmentation associated with the failure of the seismically-detailed SCC column (columns CRC-2%B-75 and CRC-2%C-75 showed similar failure response to column CRC-2%A-75). The improved damage tolerance of UHPFRC is further illustrated when comparing the failure photos of columns SCC-0%-38 and CRC-2%B-

75 in Figure 5.13b & Figure 5.13c, where the degree of damage and spalling is significantly reduced for the UHPFRC column. Although the reduction of fragmentation may not be critical in the case of columns, flying debris may pose a hazard in other types of reinforced concrete structural elements (e.g. slabs and walls). The use of UHPFRC with its improved damage tolerance can be used to prevent the effect of flying debris in such elements and could thus eliminate injury to occupants. The use of UHPFRC also had an important effect on failure mode. Contrary to the control SCC specimens in Burrell's (2012) test program, all the UHPFRC specimens in the current study prevented compression rebar buckling at Blast 80, irrespective of presence or absence of seismic detailing. Failure of the CRC specimens containing 10M reinforcement was caused by rupture of tensile steel. The rupture of the tensile reinforcement can be linked to the high compressive strength of CRC which results in the development of high tensile strains, which in turn can lead to rupture of tensile steel reinforcement. In the current research study the use of larger reinforcement in the form of 15M bars was found to improve the blast capacity of the columns and prevent the premature rupture of tension steel (see Figure 5.13d which shows column CRC-2%B-75-15M after two applications of Blast 100 loads).

It is noted that in contrast to the control SCC columns, there was no distinct formation of plastic hinging at mid-span in the case of the UHPFRC columns (compare Figure 5.13b & Figure 5.13c). It is also noted that all the UHPFRC columns generally showed limited damage prior to application of the failure blasts, however at failure major damage concentrated at a single crack, with fiber pullout. This is in accordance with the observations made by Bidiganavile et al. (2010), where CRC showed excellent energy absorption capacity and damage tolerance under impact at moderate drop-heights, but was observed to become more brittle at large drop-heights. Finally, it should be mentioned that the columns in this test program were subjected to repeated blast loads before failure. This is an important consideration given that the application of repeated blast loads can trigger debonding and partial pullout of the straight and smooth fibers prior to application of the blast required to cause failure. As mentioned previously, examination of high-speed video showed wide opening of the critical tensile crack in column CRC-2%B-75-15M during Blast 100. It is noted that this column failed under a second application of this same blast and thus supports this observation. Further research comparing behaviour of UHPFRC columns under single and repeated blast loads is recommended.

					
Blast 80	Blast 80	Blast 100	Blast 80	Blast 100	Blast 100
(a) SCC-0%-75	(b) SCC-0%-38		(c) CRC-2%A-75		(d) CRC-2%B-75
					
Blast 100	Blast 80	Blast 100	Blast 100(2)	Blast 100	Blast 100
(e) CRC-2%C-75	(f) CRC-2%A-38			(g) CRC-2%B-38	(h) CRC-2%C-38
					
Blast 100	Blast 100(2)	Blast 100	Blast 100(2)		
(i) CRC-2%B-75-15M		(j) CRC-2%C-75-15M			

Figure 5.12 Sample photos for Series 1 columns after Blast 35-100



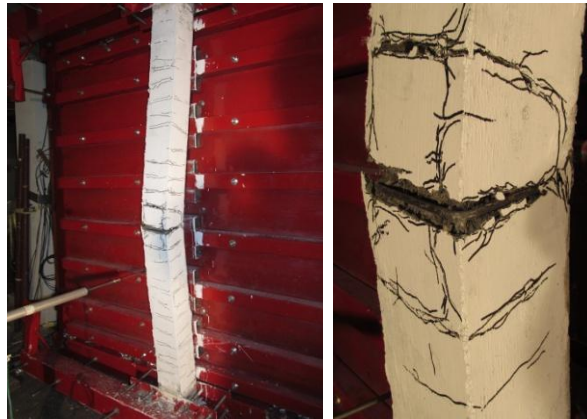
(a) SCC-0%-75 (Blast 80)



(b) SCC-0%-38 (Blast 100)

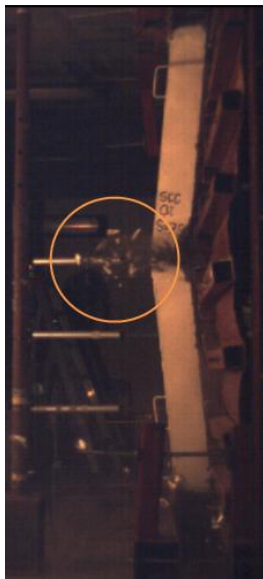


(c) CRC-2%B-75 (Blast 100)



(d) CRC-2%B-75-15M (Blast 100(2))

Figure 5.13 Comparison of damage tolerance and mode of failure in SCC and UHPFRC columns



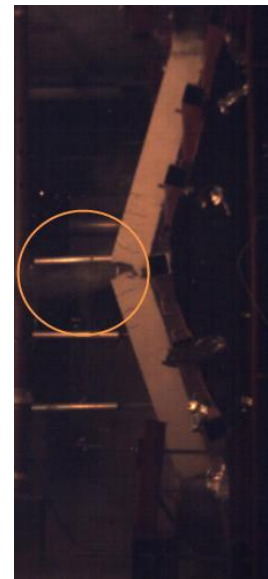
(a) SCC-0%-75 at Blast 80



(b) CRC-2%B-75 at Blast 80



(c) SCC-0%-38 at Blast 100



(d) CRC-2%B-75 at Blast 100

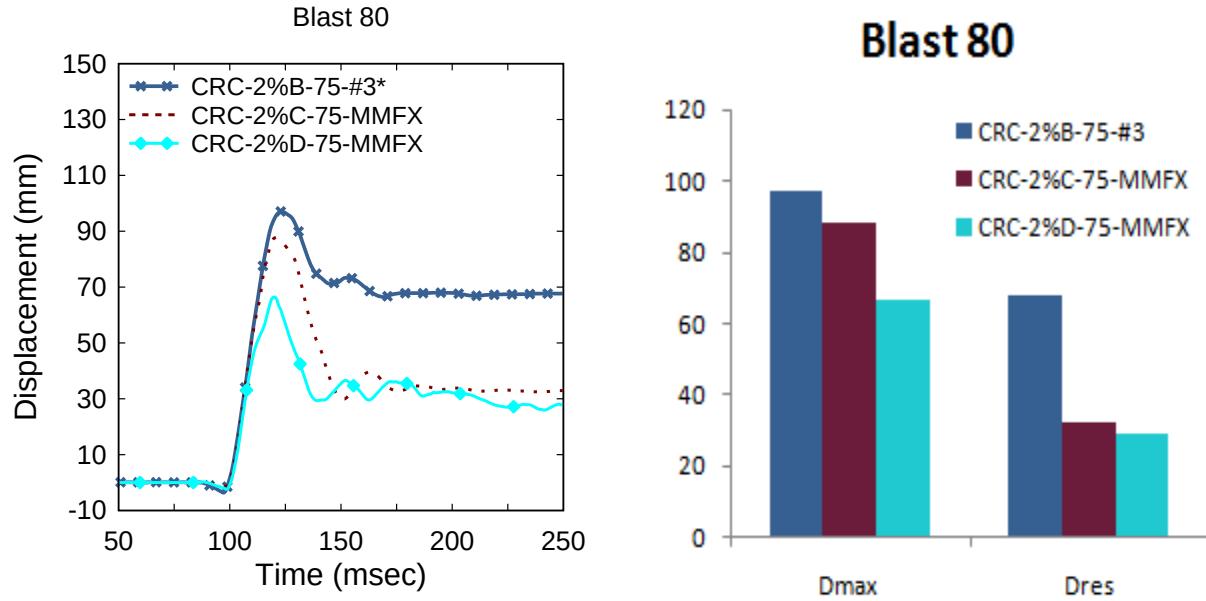
Figure 5.14 High speed video stills showing secondary blast fragments for SCC and UHPFRC columns.

5.3 Series 2- High Strength MMFX Steel

This section discusses the results from the Series 2 columns which were constructed with high-strength MMFX steel. Maximum and residual displacements for all columns in this series, along with identification of failure mode, are summarized in Figure 9.2. Displacement time-histories at Blast 80 and 100 pressures are compared in figures following the effect of parameter discussed, while photos of the columns after the application of Blasts 80 and 100 are shown in Figure 5.20.

5.3.1 Effect of High-Strength Steel

Column CRC-2%B-75-#3 tested by De Carufel (2015) and columns CRC-2%C-75-MMFX and CRC-2%D-75-MMFX had similar properties (cross-section, concrete type and hoop spacing), but were constructed with 4 - #3 regular grade and 4-#3 high-strength (MMFX) longitudinal steel bars, respectively. When compared to regular grade steel, MMFX has higher tensile strength; the increased tensile resistance is expected to lead to increased stiffness and improved flexural capacity in columns tested under lateral blast pressures. Blasts 35, which was intended to test the columns at yield conditions, resulted in maximum and residual displacements of $D_{\max} = 25.05$ mm & $D_{\text{res}} = 6.31$ mm for Column CRC-2%B-75-#3, while Columns CRC-2%C-75-MMFX and CRC-2%D-75-MMFX showed comparative displacements of $D_{\max} = 23.89$ mm & $D_{\text{res}} = 12.88$ mm, and $D_{\max} = 19.34$ mm & $D_{\text{res}} = 1.86$ mm, respectively (see Figure 5.15). At Blast 80, the control CRC-2%B-75-#3 column had maximum and residual displacements of $D_{\max} = 97.19$ mm and $D_{\text{res}} = 67.64$ mm, while the companion columns containing MMFX steel reinforcement had comparative displacements of $D_{\max} = 87.84$ mm & $D_{\text{res}} = 32.44$ mm and $D_{\max} = 66.44$ mm & $D_{\text{res}} = 28.98$ mm for CRC-2%C-75-MMFX and CRC-2%D-75-MMFX, relative reductions of 10% & 53%, and 31% & 57%, respectively (see Figure 5.15). The failure mechanism of all CRC columns was tension rebar rupture after Blast 100 regardless of steel reinforcement type (see Figure 5.20a, c & d). The results at Blast 80 indicate some benefits associated with the use of high-strength steel in columns subjected to blast loads. However, it should be noted that the American #3 bars used in these columns are relatively small in area ($A_s = 71 \text{ mm}^2$) and further research on columns having larger diameter high-strength bars is recommended.



*Specimen tested by De Carufel (2015)

Figure 5.15 Effect of MMFX: comparison of response at Blast 80

5.3.2 Effect of Fibre Length

The effect of fibre length can be studied by comparing the response of columns CRC-2%C-75-MMFX & CRC-2%D-75-MMFX. These columns had identical properties but had different fibre properties. Fibre C had a length of 13 mm while fibre D had a length of 20mm resulting in a length increase of 35% for fibre D. Under Blast 80 loading, columns CRC-2%C-75-MMFX and CRC-2%D-75-MMFX show maximum and residual displacements of $D_{\max} = 87.84 \text{ mm}$ & $D_{\text{res}} = 32.44 \text{ mm}$ and $D_{\max} = 66.44 \text{ mm}$ & $D_{\text{res}} = 28.89 \text{ mm}$, relative reductions of 24% and 10% for the column constructed with fibre type D. The full response comparison of the two columns can be examined in Figure 5.16. Results show that increasing the length of the fibre by 35% improves column performance under equivalent blast loads. It is noted that the two fibres had different strengths; to better examine the effect of fiber length further research on fibres with same tensile strength is recommended. Nonetheless, it should be noted that fibre C had a tensile strength increase of 57% and one would expect the same or better improvement in performance if the two fibres had the same strength. It is important to note that the use of fibre type D resulted in a CRC mix which had almost no slump flow; thus using this fibre on a work site may not be practical. Increased blast pressures at Blast 100 levels were required to cause failure in both columns, with failure occurring due to rupture of the tensile steel (see Figure 5.20c & d). In summary, the results from the tests conducted in the investigation indicate that the use of longer fibres in

UHPFRC columns can result in better control of displacements under equivalent blasts, while noting the loss in mix workability associated with the use of such fibres; further research focussing on developing workable UHPFRC mix designs suitable for longer fibres is recommended.

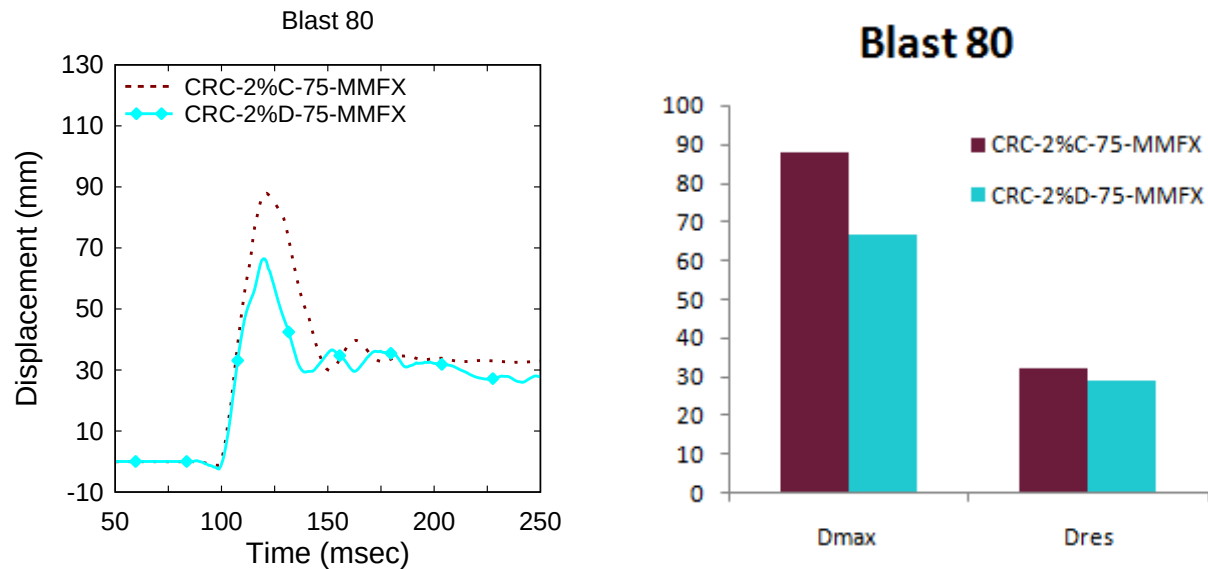


Figure 5.16 Effect of fibre length: comparison of response at Blast 80

5.3.3 Effect of Aspect ratio

The effect of aspect ratio can be investigated by comparing the response of columns CRC-2%B-38-MMFX and CRC-2%C-38-MMFX. Each column had identical properties but had different fibre properties. The aspect ratio of the fibres were 65 (Fibre B) and 43 (Fibre C). Despite having a lower aspect ratio, fibre C had a tensile strength increase of 36% over fibre B. All steel fibre properties can be found in Table 3.3. Under Blast 80 loading, column CRC-2%B-38-MMFX shows maximum and residual displacements of $D_{\max} = 46.31$ mm & $D_{\text{res}} = 14.79$ mm while column CRC-2%C-38-MMFX shows $D_{\max} = 65.05$ mm & $D_{\text{res}} = 21.68$ mm, resulting in modest reductions of 29% and 32% for fibre type B. The full response comparison of the two columns can be examined in Figure 5.17. Increased blast pressures at Blast 100 levels were required to cause failure in both columns, with failure occurring due to rupture of the tensile steel (see Figure 5.20f & g). However, the failure of the seismically detailed column with B fibres was more controlled. Given the limited testing conducted further research examining the effect of fibre aspect ratio vs. fibre tensile strength is recommended.

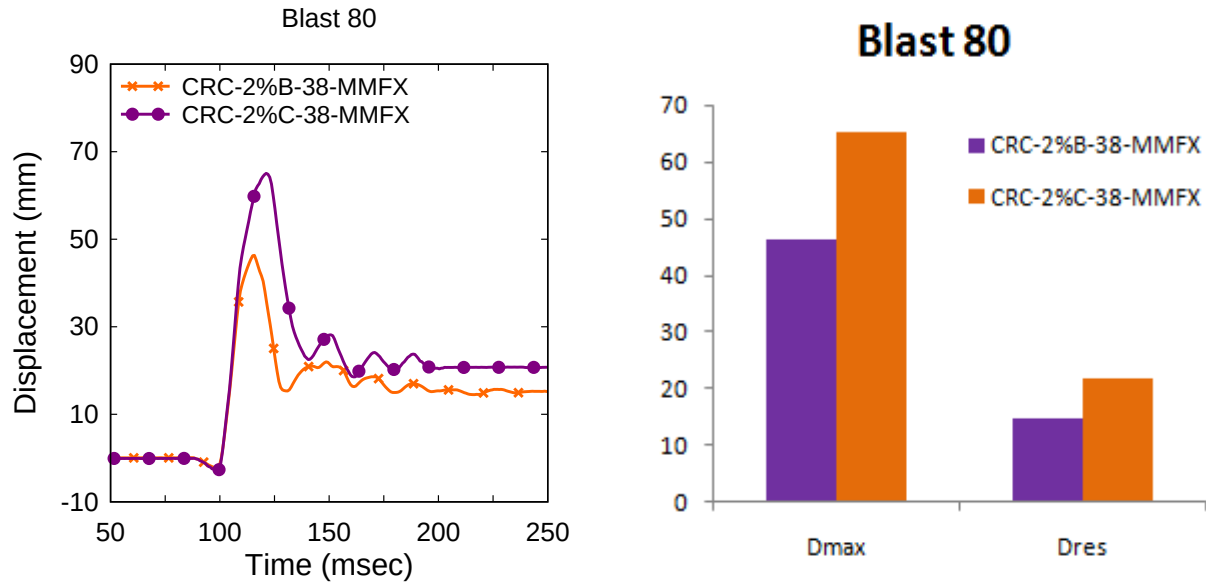


Figure 5.17 Effect of aspect ratio: comparison of response at Blast 80

5.3.4 Effect of Fibre content

The effect of fibre content can be investigated by comparing the response of columns CRC-2%C-75-MMFX & CRC-3%C-75-MMFX. These columns had identical properties but had different fibre content. As their titles suggest, one column contained 2% of steel fibres by volume of concrete while the other contained 3%. Under Blast 80 loading, column CRC-2%C-75-MMFX shows maximum and residual displacements of $D_{max} = 87.84$ mm & $D_{res} = 32.44$ mm while column CRC-3%C-75-MMFX shows displacements of $D_{max} = 61.32$ mm & $D_{res} = 15.66$ mm. The full response comparison of the two columns can be examined in Figure 5.18. The results show that increasing the fibre content by 1% reduced maximum mid-span displacements by 30% and reduced residual displacements by half under Blast 80 pressures. Increased blast pressures at Blast 100 levels were required to cause failure in both columns, with failure occurring due to rupture of the tensile steel (see Figure 5.20c & e). Both specimen were subjected to brittle failure under Blast 100. The results from the tests conducted in the investigation indicate that increasing fibre content in UHPFRC columns results in better control of displacements under equivalent blast loads. This same effect was observed by Burrell (2012) in his research program. The increase in fibre content did not significantly affect the slump-flow of the concrete, however the effect was more noticeable during concrete placement, where workability was reduced. Thus, for practical applications, the effect of fibre content on mix workability should be considered to ensure that UHPFRC can be placed during construction.

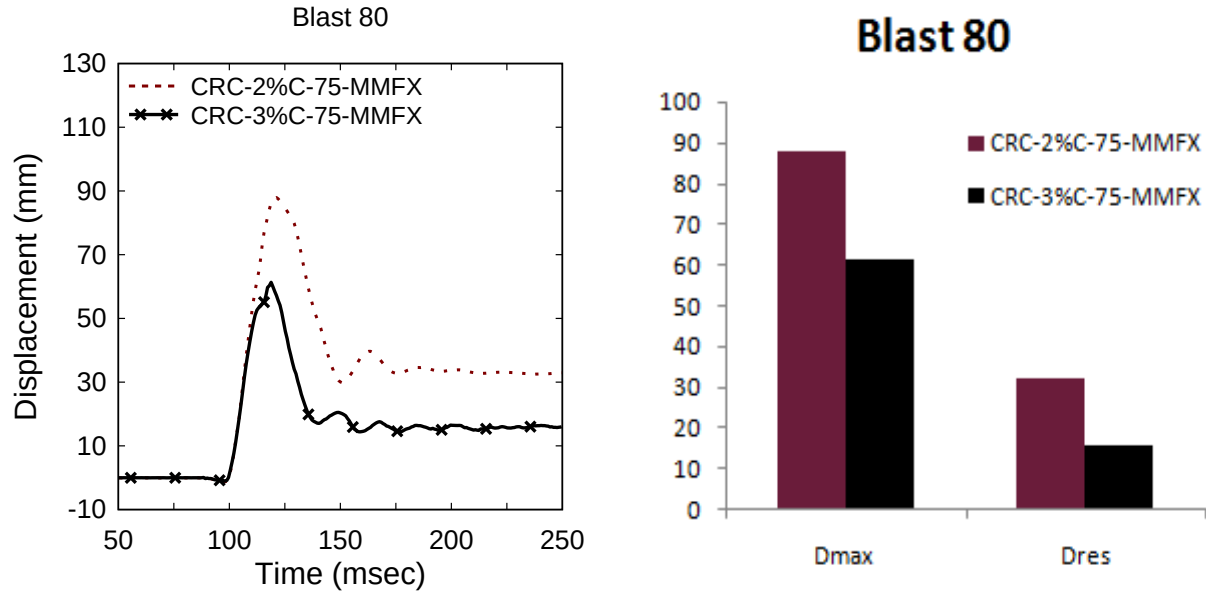


Figure 5.18 Effect of fibre content: comparison of response at Blast 80

5.3.5 Effect of Seismic Detailing

The effect of seismic detailing was the last parameter investigated in this series. The benefit of combined use of CRC, high strength steel and seismic detailing is highlighted by comparing the responses of columns CRC-2%C-75-MMFX & CRC-2%C-38-MMFX after Blast 80. When compared, the maximum and residual displacements were reduced by factors of approximately 26% and 33% for column CRC-2%C-38-MMFX ($D_{\max} = 65.1$ mm and $D_{\text{res}} = 21.7$ mm) when compared to column CRC-2%C-75-MMFX ($D_{\max} = 87.9$ mm and $D_{\text{res}} = 32.4$ mm) (see Figure 5.19). Benefits associated with the combined use of CRC, high strength steel and seismic detailing are also observed when comparing the failure mode of the columns at Blast 100, with the provision of seismic hoops resulting in a more controlled failure under extreme blast pressures (see Figure 5.20c & g). The effect is even more evident when comparing the failure modes of columns CRC-2%B-38-MMFX and CRC-2%B-75-MMFX after Blast 100, where the seismically detailed column shows a much more controlled failure with limited residual displacements (see Figure 5.20). In summary, the results confirm observations made in Series 1, and demonstrate that improvements in UHPFRC column performance from seismic detailing are possible if the behaviour is dominated by combined flexure and axial compression.

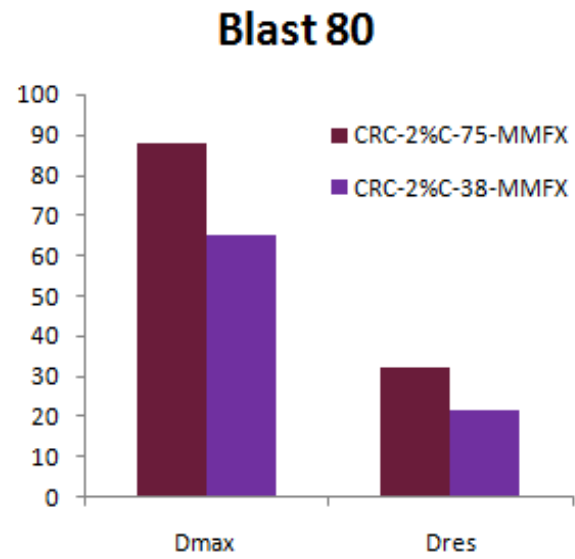
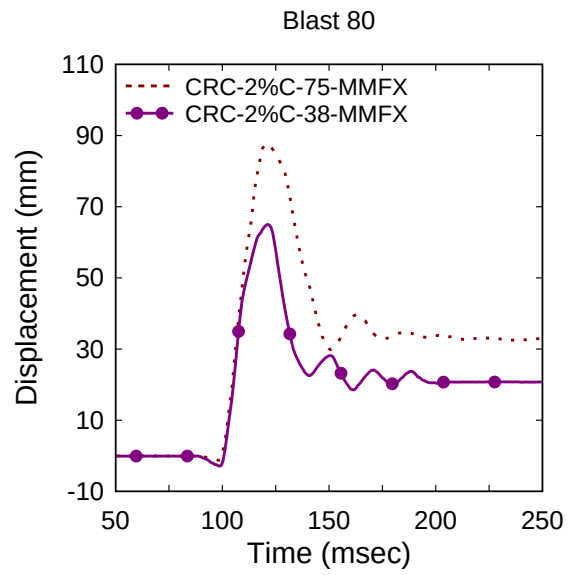
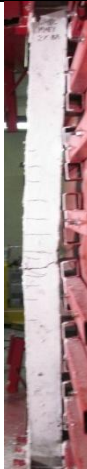
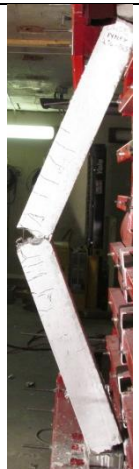


Figure 5.19 Effect of seismic detailing: comparison of response at Blast 80

					
Blast 80	Blast 100	Blast 80	Blast 100	Blast 80	Blast 100
(a) CRC-2%B-75-#3*		(b) CRC-2%B-75-MMFX		(c) CRC-2%C-75-MMFX	
					
Blast 80	Blast 100	Blast 80	Blast 100		
(d) CRC-2%D-75-MMFX		(e) CRC-3%C-75-MMFX			
					
Blast 80	Blast 100	Blast 80	Blast 100		
(f) CRC-2%B-38-MMFX		(g) CRC-2%C-38-MMFX			

*Columns tested by De Carufel, 2015

Figure 5.20 Sample photos for Series 2 columns after Blast 35-100

6 Analytical Results

6.1 Prediction of Blast Response of UHPFRC Columns

Dynamic inelastic time-history analysis was conducted to compute the displacement response of the UHPFRC test columns analytically. This was done using a single-degree-of-freedom (SDOF) model, steel models, and lumped inelasticity approach, as described in this section.

6.2 UHPFRC Compression and Tension Models

To describe the static stress-strain behaviour of confined UHPFRC in the columns a fiber reinforced concrete confinement model proposed by Aoude (2008) was used. The model modifies the expressions in the Légeron and Paultre (2003) high-strength concrete confinement model to incorporate the additional confinement provided by steel fibers. Table 6.1 summarizes the equations for peak stress/strain, and the relationships used for the ascending and descending branches of the stress-strain curve in both models. In the Légeron and Paultre (2003) model, confinement is taken into account using an "effective confinement index" ($I_E = f_{le}/f'_{co}$), where the effective confining pressure ($f_{le} = K_e f_l$) is calculated using the nominal confining pressure provided by the steel hoops (f_l) and a confinement coefficient (K_e) which takes into account the "arching action" which occurs between the levels of the transverse steel in columns having rectangular cross-section (Légeron and Paultre, 2003). In the Aoude (2008) model, the additional fiber confining pressure (f_{lf}) is obtained by multiplying the number of steel fibers per unit area (N_f) by the average fiber pullout strength ($F_{pullout}$) (Aoude, 2008):

$$f_{lf} = N_f \times F_{pullout} = \alpha v_f \frac{l_f}{d_f} \tau_{bond} \quad [6.1]$$

where α = fiber orientation factor (taken as 3/8), v_f , l_f , d_f = volume fraction, length and diameter of the fibers, respectively; and τ_{bond} = matrix bond strength, assumed to be equal to two times the tensile strength of the concrete matrix (i.e. $\tau_{bond} = 0.6 \times f'_{co}{}^{2/3}$, where f'_{co} is the compressive strength of unconfined CRC). Consistent with the Légeron and Paultre (2003) model, the stress-strain curve is defined using the point of peak confined strain and stress (ϵ'_{cc} , f'_{cc}) and the strain at 50% drop in peak confined stress (ϵ_{cc50} , $0.5f'_{cc}$). The model assumes that

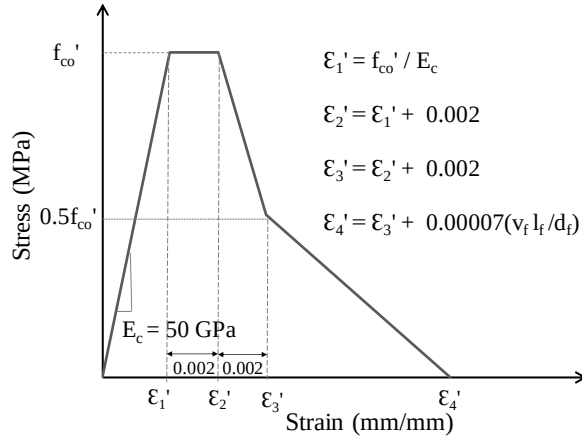
fibers and transverse reinforcement play a hybrid role in improving the post-peak behaviour of core concrete, with fibers contributing more significantly to ductility in nominally confined columns, and to a lesser extent in well confined columns. To account for this effect, the strain ε_{cc50} is modified as follows (where ε_{co50} = strain at 50% f'_{co}):

$$\varepsilon_{cc50} = \varepsilon_{co50} \left[1 + 60 \left(\frac{f_{le}}{f'_{co}} + \left(\frac{f_{lf}}{f'_{co}} \right) (1 - K_e) \right) \right] \quad [6.2]$$

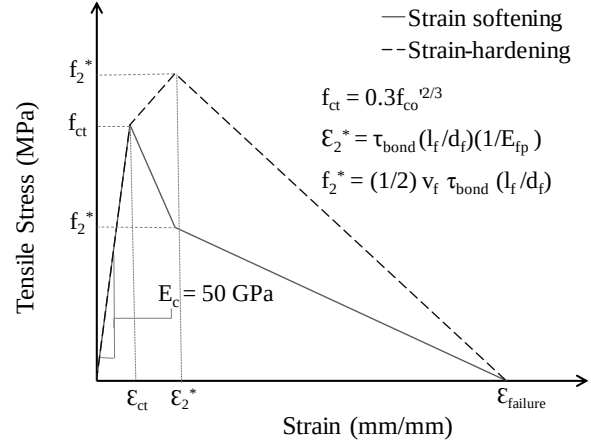
The static response of unconfined UHPFRC in the cover region of the columns was modeled using the simplified stress-strain curve presented in Figure 6.1a proposed by Hosinieh et al. (2015). The model consists of a linear ascending branch having a slope equivalent to the modulus of elasticity of UHPFRC (taken as $E_c = 50$ GPa for CRC) and a maximum stress equivalent to the peak unconfined stress of the UHPFRC (f'_{co}). The post-peak branch of the curve approximates the behaviour observed in the stress-strain data obtained from CRC cylinder tests. The tension portion of the UHPFRC stress-strain curve was modeled using a model proposed by Lok and Pei (1998). As shown in Figure 6.1b the pre-cracking portion of the curve is linear having a slope equivalent to the modulus of elasticity of CRC and a maximum stress equivalent to the cracking stress of CRC (taken as $f_{ct} = 0.3f'_{co}{}^{2/3}$). The post cracking branch of the curve is a bilinear relationship, with the first portion expressed as linear from the point of cracking to a stress $f_2^* = \frac{1}{2}v_f\tau_{bond}\frac{l_f}{d_f}$ and strain $\varepsilon_2^* = \tau_{bond}\frac{l_f}{d_f}\frac{1}{E_{fp}}$, where $E_{fp} = 200$ GPa and v_f , l_f , d_f and τ_{bond} are as defined previously. The second portion of the branch is linearly descending from the stress f_2^* to tensile stress failure (0 MPa) at a strain of 0.02 mm/mm (Lok and Pei, 1998).

Table 6.1 Summary of expressions used in the concrete and fiber reinforced concrete confinement models

Model	Peak confined stress (f'_{cc})	Peak confined strain (ϵ'_{cc})	Relationship for confined stress (f_c) as a function of confined strain (ϵ_c)		Post-peak curve defined with
			Ascending branch ($\epsilon_c \leq \epsilon'_{cc}$)	Descending branch ($\epsilon_c \geq \epsilon'_{cc}$)	
Légeron & Paultre (2003)	$f'_{co} \left[1 + 2.4 \left(\frac{f_{le}}{f'_{co}} \right)^{0.7} \right]$	$\epsilon'_{co} \left[1 + 35 \left(\frac{f_{le}}{f'_{co}} \right)^{1.2} \right]$	$f'_{cc} \left[\frac{k \left(\frac{\epsilon_c}{\epsilon'_{cc}} \right)}{k - k - \left(\frac{\epsilon_c}{\epsilon'_{cc}} \right)^k} \right]$	$f'_{cc} \left[e^{-k_1 (\epsilon_c - \epsilon'_{cc})^{k_2}} \right]$	ϵ_{cc50} = strain at 50% f'_{cc}
Aoude (2008)	$f'_{co} \left[1 + 2.4 \left(\frac{f_{le}}{f'_{co}} \right)^{0.7} + 4.1 \left(\frac{f_{lf}}{f'_{co}} \right) \right]$	$\epsilon'_{co} + 0.21 \left(\frac{f_{le}}{f'_{co}} \right)^{1.7}$			
Notes	f'_{co}, ϵ'_{co} = peak unconfined stress and strain f_{le} = effective confining pressure provided by transverse steel f_{lf} = fiber confining pressure		r, k = parameters affecting slope of ascending branch k_1, k_2 = parameters affecting shape of descending branch		



(a) Model for unconfined UHPFRC in compression



(b) Model for UHPFRC in tension

Figure 6.1 Definition of stress-strain models for UHPFRC in compression and tension

6.3 Steel Constitutive models

6.3.1 Regular Grade Steel

6.3.1.1 Reinforcing Steel Model in Tension

In order to model the stress-strain behaviour of the regular grade tensile steel reinforcement a three-part curve which combines a bilinear relation for the elastic and post yield region and a

parabolic function for the strain hardening behaviour is used. The equations for this curve are detailed in the work of Jacques et al. (2012) and are summarized in the following equations:

$$f_s = E_s \varepsilon_s \quad \text{for} \quad \varepsilon_s \leq \varepsilon_y \quad [6.3]$$

$$\varepsilon_y = f_y / E_s \quad [6.4]$$

$$f_s = f_y + (\varepsilon_s - \varepsilon_y) \left(\frac{f_{sh} - f_y}{\varepsilon_{sh} - \varepsilon_y} \right), \quad \text{for} \quad \varepsilon_y < \varepsilon_s \leq \varepsilon_{sh} \quad [6.5]$$

$$f_s = f_y + (f_u - f_y) \left[2 \frac{\varepsilon_s - \varepsilon_{sh}}{\varepsilon_u - \varepsilon_{sh}} - \left(\frac{\varepsilon_s - \varepsilon_{sh}}{\varepsilon_u - \varepsilon_{sh}} \right)^2 \right] \quad [6.6]$$

where f_s is the steel stress at strain ε_s , E_s is the steel modulus of elasticity, f_y is the steel yield stress at steel yield strain ε_y , f_{sh} is the steel strain-hardening stress at strain ε_{sh} , and f_u is the ultimate steel stress at strain ε_u . Figure 6.2 shows a schematic of the stress-strain curve for this reinforcing steel model (Jacques et al., 2012).

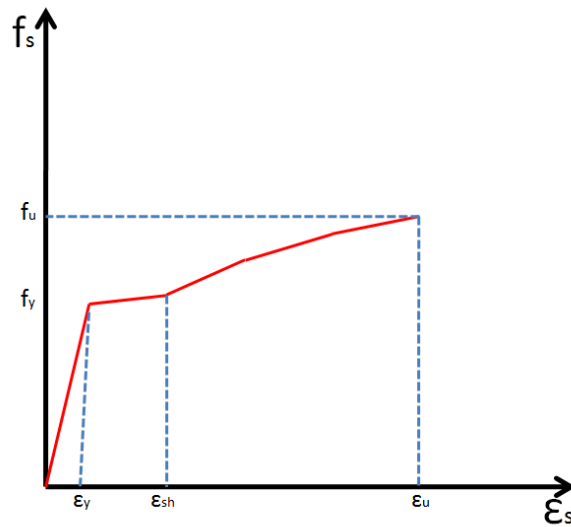


Figure 6.2 Static stress-strain strain hardening model for reinforcing steel in tension

6.3.1.2 Reinforcing Steel Model in Compression

The model proposed by Yalcin and Saatcioglu, (2000) is used to model the behaviour of longitudinal rebar in compression and account for bar buckling. In the model, the bar aspect ratio is defined as the unsupported rebar length to rebar diameter ratio. When this ratio is greater than 8.0, stability of rebar is assumed to be lost once yielding is attained. At a ratio equal to 8.0, it is assumed that the reinforcement is able to maintain some strength after yield. Finally, if the ratio is below 8.0, limited strain-hardening is assumed to occur in the stress-strain response of the steel rebar. The following equations describe the stress-strain equations used in this model:

$$f_s = f_y + (f_{s/Du} - f_{sh}) \left[2 \frac{\varepsilon_s - \varepsilon_{sh}}{\varepsilon_{s/Du} - \varepsilon_{sh}} - \left(\frac{\varepsilon_s - \varepsilon_{sh}}{\varepsilon_{s/Du} - \varepsilon_{sh}} \right)^2 \right] \text{ for } \varepsilon_s > \varepsilon_s \quad [6.7]$$

$$f_{s/Du} = f_{sh} + (f_u - f_{sh}) [48e^{-0.9(s/d_b)}] \quad [6.8]$$

$$\varepsilon_{s/Du} = \varepsilon_{sh} + (\varepsilon_u - \varepsilon_{sh}) [6e^{-0.4(s/d_b)}] \quad [6.9]$$

where $f_{s/Du}$ and $\varepsilon_{s/Du}$ are the limiting values on stress and strain respectively. Figure 6.3 shows the stress-strain relationship for the model described above.

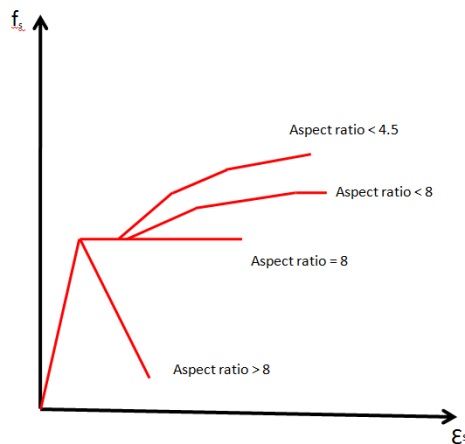


Figure 6.3 Static stress-strain relationship for reinforcing steel in compression which accounts for buckling adapted from Yalcin and Saatcioglu (2000)

6.3.2 High-Strength Steel Model

To model the stress-strain behaviour of high-strength steel, the model proposed by ACI Task Group ITG-6 can be used (ACI Innovation Task Group 6, 2010). The model can predict the response of ASTM-1035 high-strength steel having similar properties to the MMFX used in this research study. Typical stress-strain response of high-strength steel does not show a well-defined yield plateau, which is considered in this model. The model proposed by ITG-6 consists of the following three equations:

$$f_s = 200,000\varepsilon_s \text{ (MPa)} \text{ For } \varepsilon_s \leq 0.0024 \quad [6.10]$$

$$f_s = 1170 - \frac{2.96}{\varepsilon_s + 0.0019} \text{ (MPa)} \text{ For } 0.0024 < \varepsilon_s \leq 0.02 \quad [6.11]$$

$$f_s = 1040 \text{ (MPa)} \text{ For } 0.02 < \varepsilon_s \leq 0.06 \quad [6.12]$$

The ITG model was used in the analytical modelling of the MMFX columns in Series 2 Figure 6.4 compares the response obtained with the ITG model and the experimental stress-strain response for the #3 MMFX steel as obtained from coupon tests.

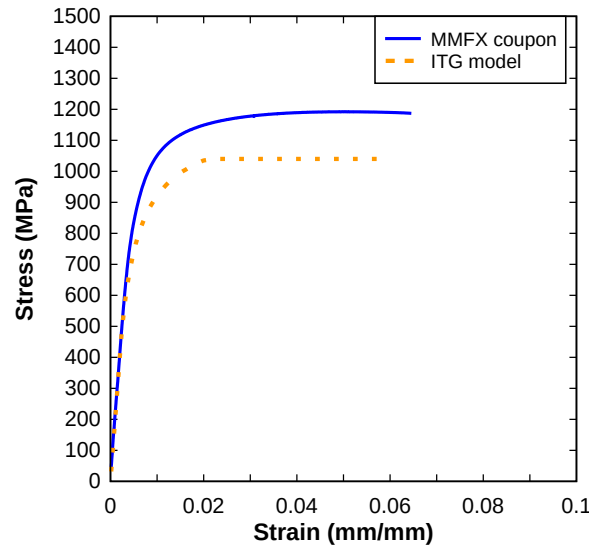


Figure 6.4 Stress-strain relationship comparison of ITG model vs. MMFX

6.4 Dynamic Analysis Using Lumped Inelasticity Approach

The dynamic analysis was conducted by solving the SDOF equation of motion given below:

$$K_{LM}(u(t))m\ddot{u}(t) + R(u(t)) = AP_r(t) \quad [6.13]$$

where $u(t)$ and $\ddot{u}(t)$ are the deflection and acceleration of the mid-height of the column, $K_{LM}(u(t))$ is the load mass transformation factor as a function of deflection, $R(u(t))$ is the resistance of the member as a function of deflection, m is the total mass of the system, A is the area impacted by the blast pressure, and $P_r(t)$ is the time-varying applied pressure. The mass was taken to be 315 kg (mass of column and load-transfer device) and the loaded area was taken to be 4.129 m² (area of the shock-tube opening).

The resistance functions used in the dynamic analysis were generated following a lumped inelasticity approach. This approach is discussed in detail by Jacques et al. (2012). The analysis begins by defining column cross-sectional dimensions, the arrangement of reinforcement and constitutive material models for UHPFRC and steel at high strain rates. Different material models are used for flexural strain compatibility analysis to establish the moment-curvature relationship. The model proposed by Aoude (2008) is used to describe the behaviour of confined UHPFRC in the column core, tension behaviour is taken into account using the Lok and Pei (1998) model and the behaviour of unconfined UHPFRC in the cover is described using the simplified model proposed by Hosinieh et al. (2015). The stress-strain relationship of longitudinal reinforcing steel consisted of two linear segments to describe elastic and post-yield behaviour, and a parabolic function to describe the effect of strain-hardening (Jacques et al., 2012). Bar buckling in compression was modeled following the model proposed by Yalcin and Saatcioglu (2000) with the stability of reinforcement expressed as a function of bar aspect ratio defined as the ratio between unsupported bar length between the adjacent ties and longitudinal bar diameter. The ITG model described in the previous section was used to model the stress-strain response of the MMFX bars in the Series 2 columns.

A bi-linear relationship proposed by Zhang and Mindess (2011) is used to account for the dynamic effects in UHPFRC under compression. This model, which was derived based on compressive impact testing of fiber reinforced concrete having static strengths of 50, 90 and 110

MPa at strain-rates ranging from 10^{-5} sec^{-1} to 10 sec^{-1} , predicts reduced strain-rate sensitivity (i.e. lower DIF) for higher strength fiber reinforced concrete, as is also expected for UHPFRC. Accordingly, the dynamic increase factor in compression (DIF_c) is given by:

$$\text{DIF}_c = \begin{cases} (\phi \dot{\epsilon})^\alpha, & \dot{\epsilon} < \dot{\epsilon}_{\text{BLT}} \\ \beta(\dot{\epsilon})^{1/3}, & \dot{\epsilon} \geq \dot{\epsilon}_{\text{BLT}} \\ \alpha = \frac{\ln(\beta \cdot \dot{\epsilon}_{\text{BLT}}^{1/3})}{\ln(\phi \cdot \dot{\epsilon}_{\text{BLT}})}, \beta = \frac{5}{9} \cdot \exp\left(\frac{-f'_{\text{cof}}}{230}\right), \phi = 10^5, \dot{\epsilon}_{\text{BLT}} = 25 \cdot \exp\left(\frac{f'_{\text{cof}}}{130}\right) \end{cases} \quad [6.14]$$

Where $\dot{\epsilon}$ is the strain rate of the steel. Research on the behaviour of UHPFRC under dynamic tension is limited and there is currently no well-established models for dynamic increase factor of UHPFRC in tension. For analysis the relationship proposed by Malvar and Ross (1998) is used to account for the strain rate effect in UHPFRC under dynamic tension, where the dynamic increase factor (DIF_t) is given by:

$$\text{DIF}_t = \begin{cases} \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^\delta, & \dot{\epsilon} \leq 1 \text{ s}^{-1} \\ \beta^* \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^{1/3}, & \dot{\epsilon} > 1 \text{ s}^{-1} \\ \delta = \frac{1}{\left(1 + \frac{8f'_{\text{cof}}}{f'_{\text{cr}}}\right)}, \text{Log } \beta^* = 6\delta - 2, f'_{\text{cr}} = 10 \text{ MPa} \end{cases} \quad [6.15]$$

For the longitudinal regular grade steel reinforcement, the expressions shown in Equation 6.16 and proposed by Saatcioglu et al. (2011) are used to determine the dynamic increase factors at yield and at ultimate stress. It is important to note that no dynamic models for high-strength steel are available in the literature and therefore analysis was conducted assuming no dynamic increase factor ($\text{DIF}=1.0$) as well as $\text{DIF}=1.1$, which corresponds with the DIF at ultimate for regular grade steel at a strain-rate of 1 s^{-1} as obtained using from Equation 6.16 (Figure 6.5f).

$$\begin{aligned} \text{at yield} &\rightarrow \text{DIF}_y = 0.034 \ln(\dot{\epsilon}) + 1.30 \geq 1.0 \\ \text{at ultimate} &\rightarrow \text{DIF}_u = 0.0101 \ln(\dot{\epsilon}) + 1.10 \geq 1.0 \end{aligned} \quad [6.16]$$

Assuming a strain rate of $\dot{\epsilon} = 1 \text{ s}^{-1}$ for the columns tested in the shock-tube, the Zhang and Mindess (2011) and Malvar and Ross (1998) models predict UHPFRC dynamic increase factors as $\text{DIF}_c = 1.20$ and $\text{DIF}_t = 1.16$, respectively, while the Saatcioglu et al. (2011) expressions predict dynamic increase factors for the regular grade reinforcing bars at yield and at ultimate as $\text{DIF}_y = 1.3$ and $\text{DIF}_u = 1.1$. The stress-strain relationships of UHPFRC and longitudinal steel under high strain rates are obtained by substituting dynamic strength parameters in place of static values. Sample static and dynamic stress-strain relationships for typical columns in Series 1 and 2 are shown in Figure 6.5.

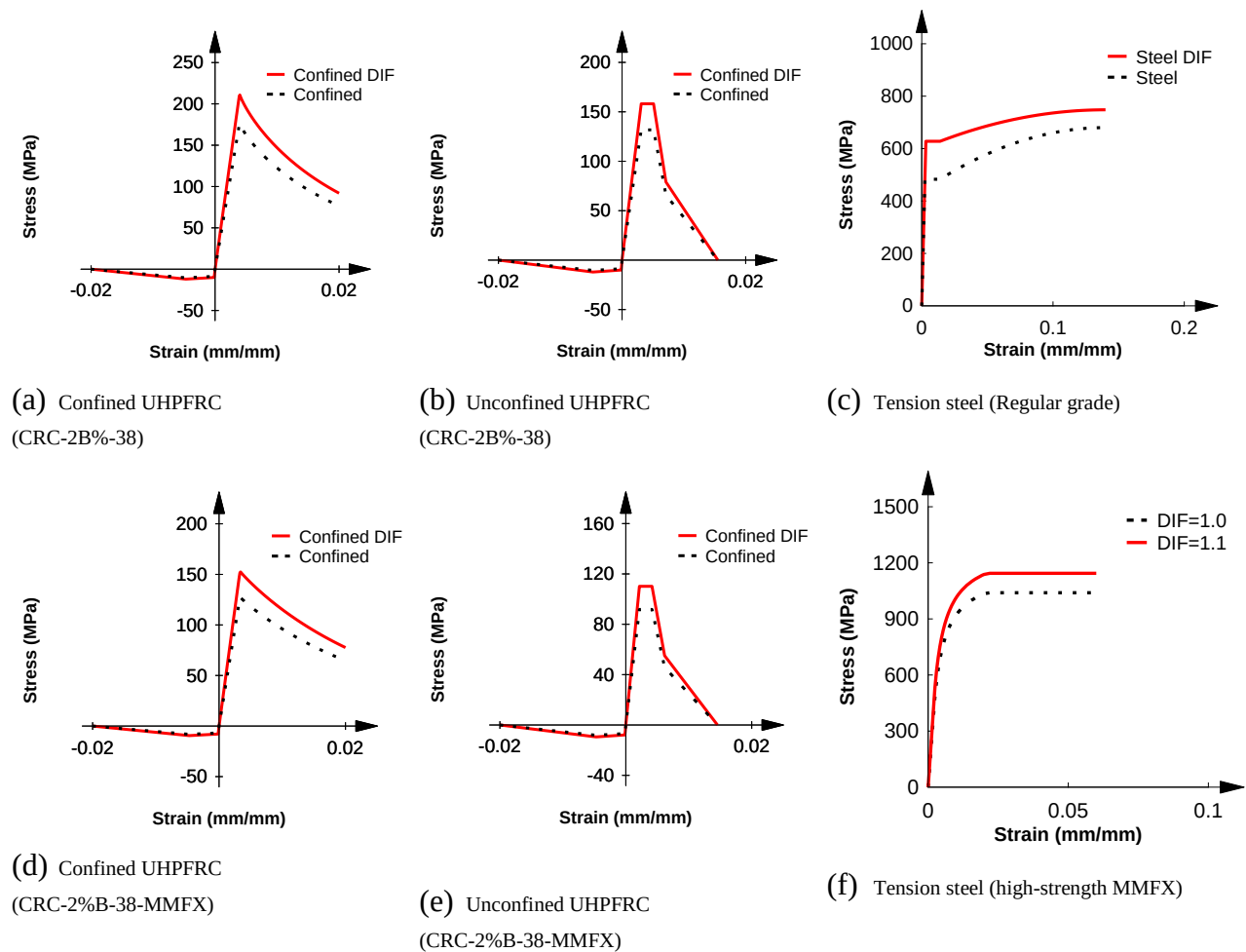
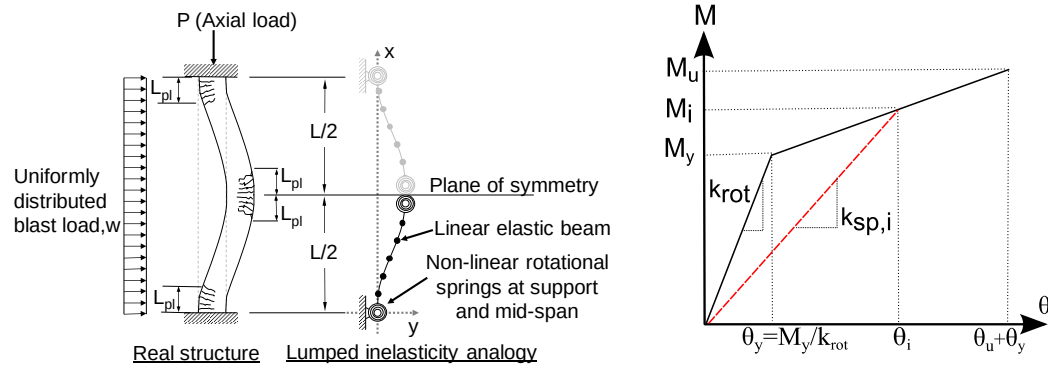


Figure 6.5 Sample stress-strain curves for confined/unconfined UHPFRC and tension steel (with dynamic effects) for columns CRC-2%B-38 and CRC-2%B-38-MMFX

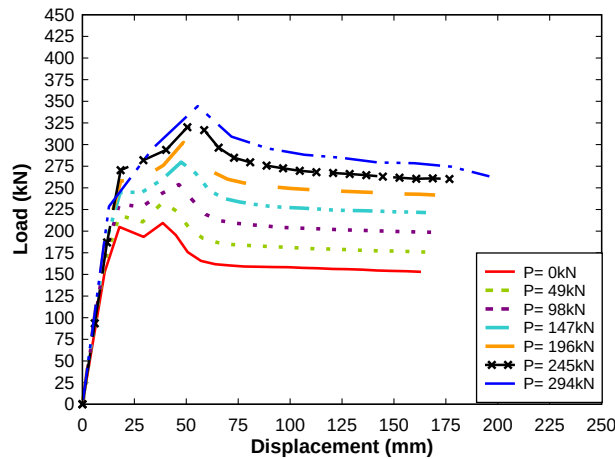


(a) Beam-column and idealized lumped inelasticity model (b) Idealized moment-rotation relationship used in derivation of rotational spring properties

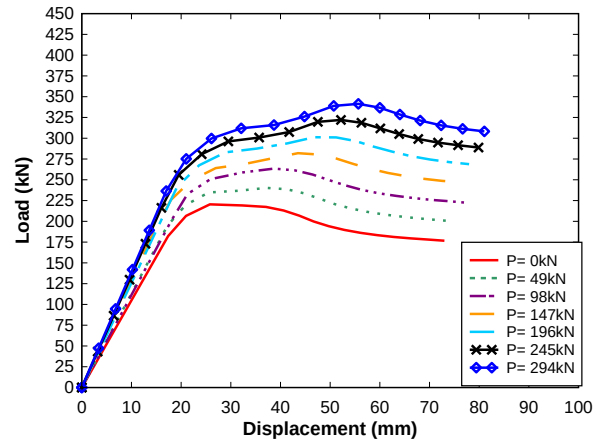
Figure 6.6 Lumped inelasticity analogy used in analysis of UHPFRC columns as defined by Jacques et al. (2012)

(adapted from Jacques et al. (2012))

Using the above constitutive material models moment-curvature relationships for UHPFRC column sections were generated. To determine the resistance functions and load-transformation factors for use in the SDOF analysis, the UHPFRC columns were idealized as half-span symmetric elastic beam-columns with non-linear rotational hinges located at critical sections. This is illustrated in Figure 6.6a. The effective pre-yield slope of the columns moment-curvature relationship was used to define the flexural rigidity (EI) of the member, while the post-yield sectional response was used to construct an idealized hinge moment-rotation relationship (see Figure 6.6b). For the analysis of the columns, the plastic hinge length at mid-height and at the supports was taken to be 135 mm, equal to the distance between the extreme compression fiber and the tension steel. The initial rotational stiffness of the partially-fixed shock-tube supports was previously found to be 903 kN·m/rad (Lloyd, 2010). The governing force-deflection equation was derived by solving the Euler-Bernoulli equation for the half-span beam-column with appropriate support and loading conditions. Resistance functions and equivalent SDOF transformation factors were constructed by solving the force-deflection equation at incrementally increasing load stages.



(a) Resistance curve for CRC-2%B-38



(b) Resistance curve for CRC-2%B-38-MMFX

Figure 6.7 Resistance curve of column CRC-2%B-38 and CRC-2%B-38-MMFX for variable axial loads and axial loads 0-294 kN showing 5 intermediate load steps

During the column tests, the axial load was reduced due to the shortening of the column under lateral blast pressures and associated horizontal deformations (Lloyd, 2015). Therefore the columns are subjected to variable axial compression, with axial loads reduced as the column deflects laterally (Lloyd, 2015). Figure 6.7 shows resistance–displacement curves for constant axial loads of different magnitudes for a typical UHPFRC column. A decrease in column load resistance occurs as the axial load reduces. To account for variable axial load, a "composite" resistance curve was generated accounting for the change in axial load as a function of lateral displacement (Jacques et al., 2012). In the current analysis five intermediate load steps (from 0 to 294 kN) were used to generate the composite resistance curves for the columns.

An equivalent triangular blast load, having the same peak reflected pressure and impulse found in the experimentally recorded pressure-time history, was used for analysis (idealized blast properties are shown in Table 6.5, Table 6.8 Table 6.9). The equation of motion was solved using the average acceleration numerical integration method (Jacques et al., 2012). The dynamic inelastic analysis procedure, from the development of the resistance functions to the solution of the equation of motion, was automated using software RCblast (Jacques, 2015).

6.5 Analysis Results

6.5.1 Series 1 – Regular Grade Steel

6.5.1.1 10M Specimens

This section discusses the analysis results for columns CRC-2%B-75 and CRC-2%C-75 which were both columns built with 10M reinforcement. The analytical prediction results for the maximum mid-span displacements for both specimens are summarized in Table 6.2. Displacement vs. Time histories of the two columns for Blast 35 and Blast 80 are shown in Figure 6.8.

For both columns, the analysis at Blast 35 is found to have under-predicted the experimentally obtained displacements, while the analysis at Blast 80 resulted in over-prediction of the displacements. At Blast 35 Column CRC-2%B-75 had an experimental maximum displacement of 17.6 mm while the analytical prediction was 16.1 mm, resulting in an error percentage of 8.6%. At Blast 80, this column had experimental and analytical displacements of 60.1 mm and 65.7 mm, respectively, resulting in an error percentage of 9.2%. Similar to its companion column, specimen CRC-2%C-75 had experimental and analytical mid-span displacements at Blast 35 of 15.7 mm and 14.5 mm (error = 7.8%). For this same column, the mid-span displacement was over- predicted by the analysis at Blast 80, with experimental and analytical displacements of 55.8 mm and 64.3 mm, respectively, resulting in an error percentage equal to 15.3%. The larger error for the column with fiber C at Blast 80 can be explained by the fact that the UHPFRC compression and tension models used in the analysis do not account for the effect of increased fiber strength on compressive and tensile behaviour of UHPFRC.

Table 6.2 Summary of analysis results for column CRC-2%B-75 and CRC-2%C-75 (Blast 35-80)

		Idealized Shockwave			Maximum Mid-span Displacement			
		P _r (kPa)	t _{df} (msec)	I _r (kPa*msec)	D _{exp} (mm)	D _{anls} (mm)	D _{anls} /D _{exp}	% Error (%)
CRC-2%B-75	Blast 35	42.9	18.5	396.2	17.6	16.1	0.91	8.6%
	Blast 80	81.3	20.4	830.0	60.1	65.7	1.09	9.2%
CRC-2%C-75	Blast 35	41.6	15.7	326.5	15.7	14.5	0.92	7.8%
	Blast 80	80.1	18.8	752.9	55.8	64.3	1.15	15.3%

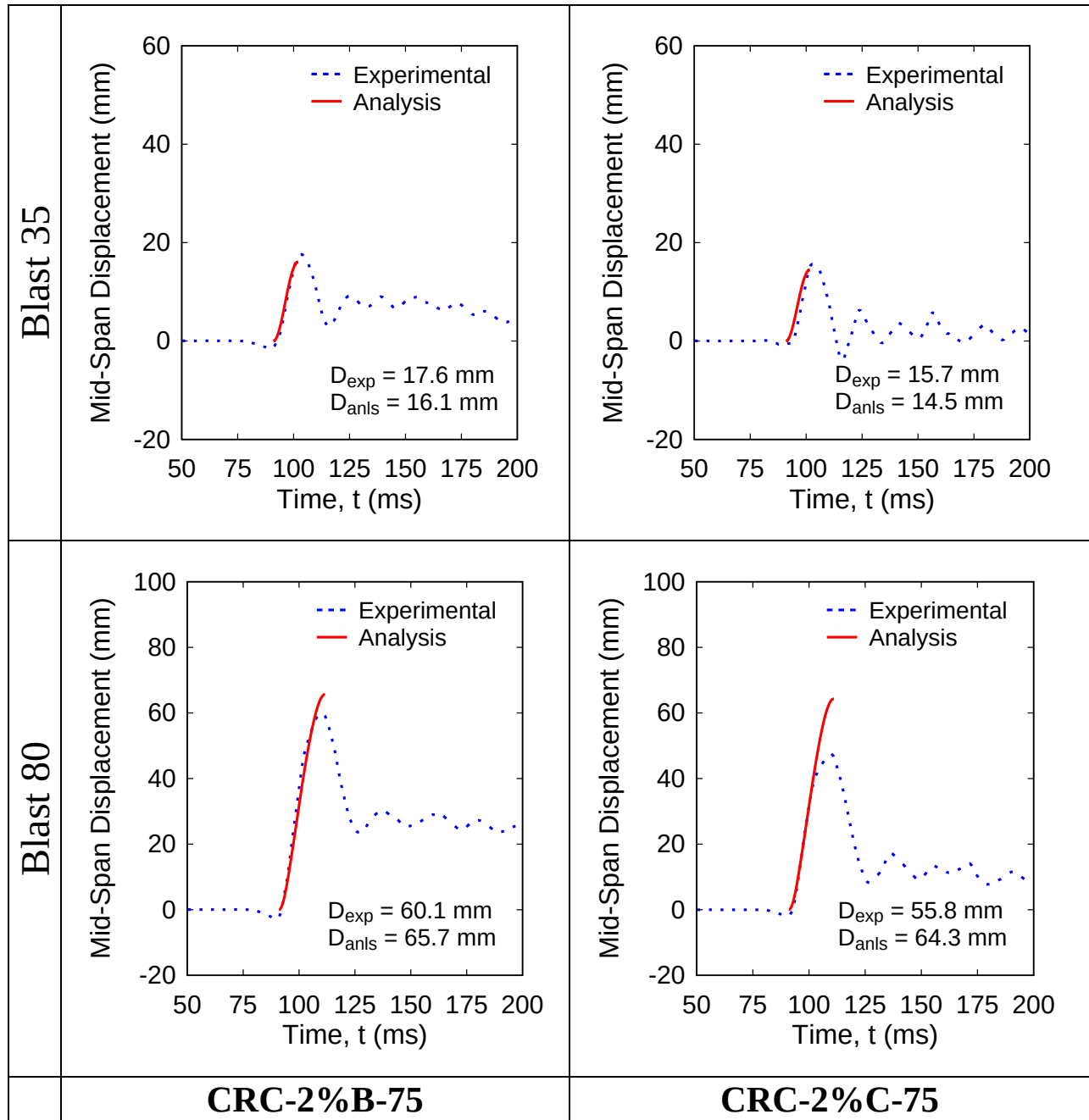


Figure 6.8 Displacement predictions time-history column CRC-2%B-75 and CRC-2%C-75 (Blast 35-80)

6.5.2 10M Seismically Designed Specimen

This section discusses the analysis results for columns CRC-2%B-38 and CRC-2%C-38. Specimens CRC-2%B-38 and CRC-2%C-38 were identical to the specimens compared in the previous section but were designed with seismic detailing with tie spacing reduced from 75 mm to 38 mm. The analytical prediction results for the maximum mid-span displacements for both specimens are summarized in Table 6.3. Displacement vs. Time histories of the two columns for Blast 35 and Blast 80 are shown in Figure 6.9.

Similar to the previous comparison, the analysis at Blast 35 was found to under-predict the mid-span displacements for both specimens. Column CRC-2%B-38 had experimental and analytical displacements of 21.8 mm and 15.6 mm at this blast, resulting in an error of 28.6%. The analysis resulted in more accurate predictions at Blast 80, with experimental and analytical displacements of 61.0 mm and 54.4 mm resulting in an error of 10.9%. Column CRC-2%C-38 had less accurate predictions when compared to column CRC-2%B-38, particularly at Blast 80. At Blast 35, the experimental and analytical displacements were 19.6mm and 13.2 mm, resulting in an error of 32.4%. The analysis for this column at Blast 80 resulted in an over-prediction of the displacements with a large error of 44.3% with analytical displacement of 59.7 mm compared to the experimental value of 41.4 mm. In general the accuracy of the analysis was reduced for the seismically detailed columns when compared to the analysis results for the companion specimens without seismic detailing. One reason returns to the lack of accurate confinement models specifically developed for UHPFRC. The increased errors associated with column CRC-2%C-38 can be explained by the fact that the UHPFRC compression and tension models used in the analysis do not account for the effect of increased fiber strength on compressive and tensile behaviour of UHPFRC, as discussed in the previous section.

Table 6.3 Summary of analysis results for column CRC-2%B-38 and CRC-2%C-38 (Blast 35-80)

		Idealized Shockwave			Maximum Mid-span Displacement			
		P _r (kPa)	t _{df} (msec)	I _r (kPa*msec)	D _{exp} (mm)	D _{anls} (mm)	D _{anls} /D _{exp}	% Error (%)
CRC-2%B-38	Blast 35	41.6	19.4	402.5	21.8	15.6	0.71	28.6%
	Blast 80	75.1	21.4	802.7	61.0	54.4	0.89	10.9%
CRC-2%C-38	Blast 35	41.5	19.1	396.1	19.6	13.2	0.68	32.4%
	Blast 80	80.3	18.6	746.0	41.4	59.7	1.44	44.3%

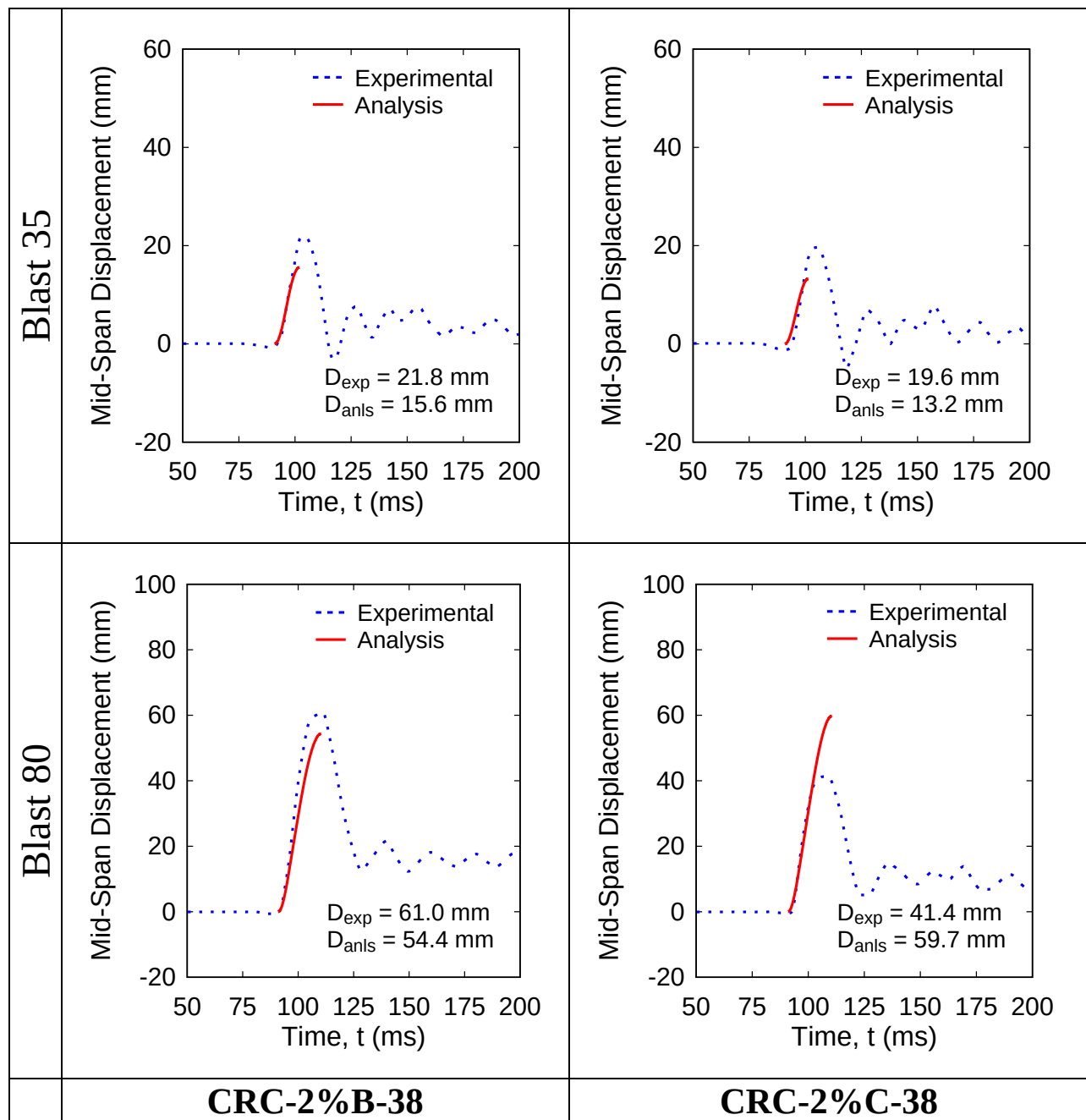


Figure 6.9 Displacement predictions time-history column CRC-2%B-38 and CRC-2%C-38 (Blast 35-80)

6.5.2.1 15M Specimens

The last specimens compared in series 1 are the columns reinforced with 15M longitudinal bars: CRC-2%B-75-15M and CRC-2%C-75-15M. The analytical prediction results for the maximum mid-span displacements for both specimens are summarized in Table 6.4. Displacement vs. Time histories of the two columns for Blast 35, Blast 80 and Blast 100 are shown in Figure 6.10. For both columns, data was experimentally recorded at Blast 100 thus analytical predictions for this blast are also provided.

In general these two columns had the least accurate predictions for all columns in series 1, with displacements under predicted by a significant margin at each blast. Column CRC-2%B-75-15M had experimental and analytical displacements for Blast 35, Blast 80, and Blast 100 of 19.9 mm & 12.2mm, 52.0 mm & 32.3 mm, and 87.8 & 44.1 respectively. These predictions resulted in errors of 38.7%, 37.9%, and 49.8%. Similar to its companion, column CRC-2%C-75-15M had experimental and analytical displacements for Blast 35, Blast 80, and Blast 100 of 20.2 mm & 12.6 mm, 50.6 mm & 35.5 mm, and 78.8 & 45.4 respectively. These predictions resulted in errors of 37.6%, 29.8%, and 42.4%. It is noted that while the use of 15M vs. 10M bars resulted in improvement of overall blast resistance, the use of larger bars did not result in significant reduction of experimental displacements at equivalent blasts. For example, at Blast 80 columns CRC-2%B-75-15M and CRC-2%B-75 had experimental displacements of 51 mm and 61 mm; the 16% reduction in maximum displacements is lower than one would expect from the doubling of bar area. Further research on specimens with 15M reinforcement is recommended to enrich the experimental database for specimens having larger reinforcement ratio.

Table 6.4 Summary of analysis results for column CRC-2%B-75-15M and CRC-2%C-75-15M (Blast 35-100)

		Idealized Shockwave			Maximum Mid-span Displacement			
		P_r (kPa)	t_{df} (msec)	I_r (kPa*msec)	D_{exp} (mm)	D_{anls} (mm)	D_{anls}/D_{exp}	% Error (%)
CRC-2%B-75-15M	Blast 35	41.7	18.9	393.6	19.9	12.2	0.61	38.7%
	Blast 80	76.8	20.6	790.0	52.0	32.3	0.62	37.9%
	Blast 100	85.6	22.4	959.1	87.8	44.1	0.50	49.8%
CRC-2%C-75-15M	Blast 35	40.7	18.8	381.8	20.2	12.6	0.62	37.6%
	Blast 80	74.6	21.5	800.2	50.6	35.5	0.70	29.8%
	Blast 100	85.8	20.3	869.4	78.8	45.4	0.58	42.4%

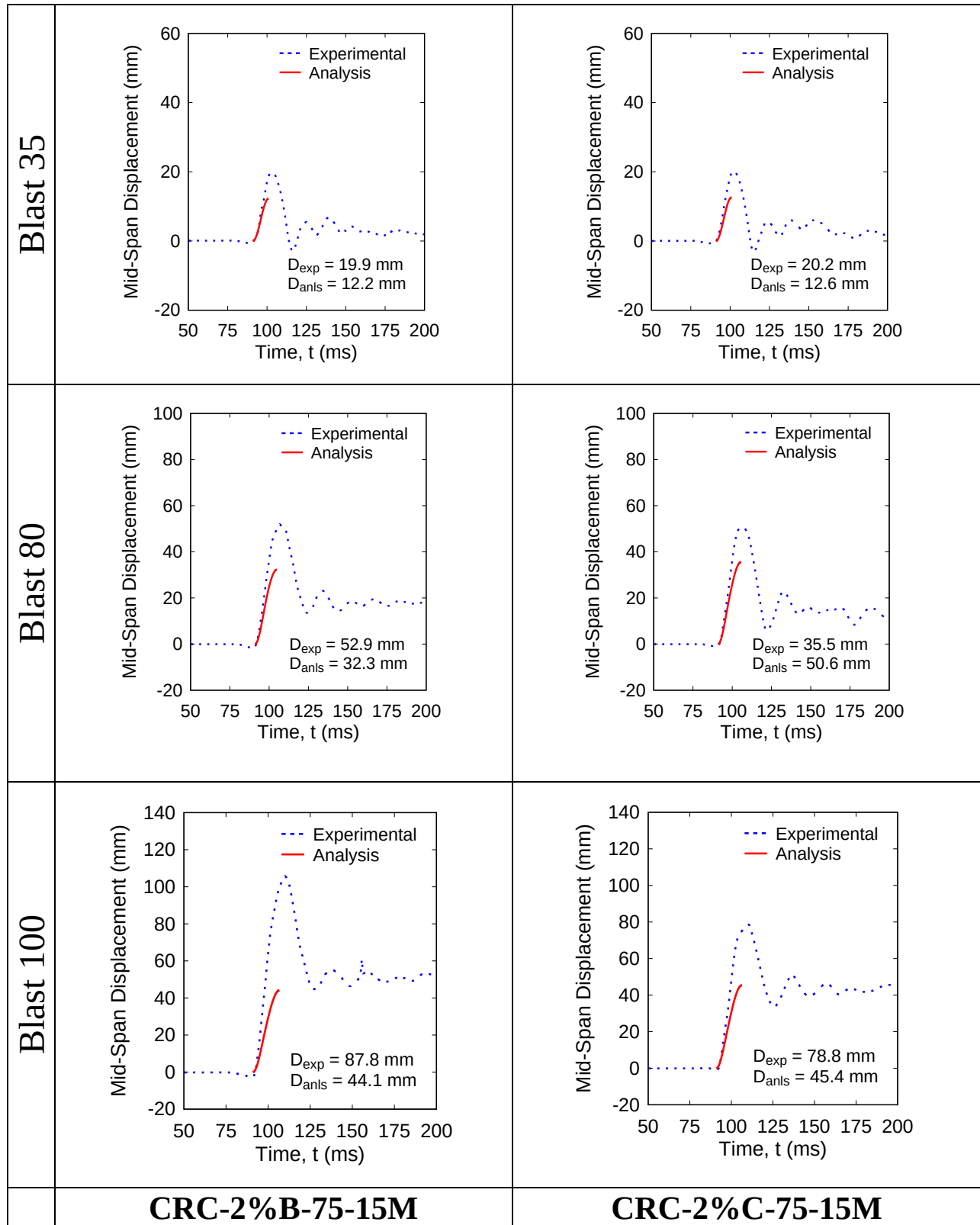


Figure 6.10 Displacement predictions time-history column CRC-2%B-75-15M and CRC-2%C-75-15M (Blast 35-

6.5.2.2 Series 1 - Overall Results

Experimental and analytical maximum mid-span displacements ($D_{\max}$ and D_{anls} respectively) and maximum displacement ratios ($D_{\text{anls}}/D_{\max}$) for the series 1 at Blasts 35, 80 & 100 pressure levels are presented in Table 6.5. This table also reports statistical data relating to the accuracy of predicted results for maximum displacement ratios (standard deviation, mean and coefficient of variation). Overall, the results show reasonable agreement between the computed and recorded response quantities. Considering all blast scenarios, the average displacement ratio ($D_{\text{anls}}/D_{\max}$) for the series 1 columns is 0.82, with a percent error of 28.1%, and corresponding standard deviation (Std Dev) of 0.27 and coefficient of variation (CoV) of 32.6%. Blast 35 resulted in average displacement ratio $D_{\text{anls}}/D_{\max} = 0.74$ (with Std Dev = 0.14 and CoV = 18.8%). The analysis results are generally similar at Blast 80 loading, with an average displacement ratio $D_{\text{anls}}/D_{\max} = 0.98$ (with Std Dev = 0.31 and CoV = 31.2%).

Table 6.5 Summary of analysis results for Series 1 columns (Blast 35-100)

		Idealized Shockwave			Maximum Mid-span Displacement				
		P _r	t _{df}	I _r	D _{exp}	D _{anls}	D _{anls} /D _{exp}	% Error	
		(kPa)	(msec)	(kPa*msec)	(mm)	(mm)		(%)	
CRC-2%B-75	Blast 35	42.9	18.5	396.2	17.6	16.1	0.91	8.6%	
	Blast 80	81.3	20.4	830.0	60.1	65.7	1.09	9.2%	
CRC-2%C-75	Blast 35	41.6	15.7	326.5	15.7	14.5	0.92	7.8%	
	Blast 80	80.1	18.8	752.9	55.8	64.3	1.15	15.3%	
CRC-2%B-38	Blast 35	41.6	19.4	402.5	21.8	15.6	0.71	28.6%	
	Blast 80	75.1	21.4	802.7	61.0	54.4	0.89	10.9%	
CRC-2%C-38	Blast 35	41.5	19.1	396.1	19.6	13.2	0.68	32.4%	
	Blast 80	80.3	18.6	746.0	41.4	59.7	1.44	44.3%	
CRC-2%B-75-15M	Blast 35	41.7	18.9	393.6	19.9	12.2	0.61	38.7%	
	Blast 80	76.8	20.6	790.0	52.0	32.3	0.62	37.9%	
	Blast 100	85.6	22.4	959.1	87.8	44.1	0.50	49.8%	
CRC-2%C-75-15M	Blast 35	40.7	18.8	381.8	20.2	12.6	0.62	37.6%	
	Blast 80	74.6	21.5	800.2	50.6	35.5	0.70	29.8%	
	Blast 100	85.8	20.3	869.4	78.8	45.4	0.58	42.4%	
Statistical Analysis					Blast #	Mean	Std Dev	COV	% Error
					35-80	0.82	0.27	32.6%	28.10%
					35	0.74	0.14	18.8%	25.60%
					80	0.98	0.31	31.2%	24.60%

6.5.3 Series 2 – High-Strength MMFX Steel

6.5.3.1 *Non-Seismically Detailed MMFX Specimens*

This section discusses the analysis results for columns CRC-2%B-75-MMFX and CRC-2%C-75-MMFX which were built with non-seismic detailing and high-strength MMFX reinforcement. As discussed previously the analysis in this series was conducted using two DIF cases. Since no established dynamic increase factor exists for high-strength steel, a DIF of 1.1 was used, corresponding to the DIF used at ultimate for regular grade steel. The second case was conducted using no dynamic increase factor (DIF=1.0). The analytical prediction results for the maximum mid-span displacements are summarized in Table 6.6. Displacement vs. Time histories of the two columns for Blast 35 and Blast 80 are shown in Figure 6.11, while Figure 6.12 shows a comparison bar chart of both DIF cases. Column CRC-2%B-75-MMFX and CRC-2%C-75-MMFX were firstly compared to predictions using the DIF=1.1. At blast 35, specimen CRC-2%B-75-MMFX had experimental and analytical displacements of 20.39 mm and 18.5 mm, resulting in an error of 9.3%. The case using no DIF resulted in a lower error of 7.3%. It is to note that column CRC-2%B-75-MMFX had no experimental data to compare at Blast 80. The companion specimen with C fibers had lower prediction accuracy at Blast 35 and 80 when compared to column CRC-2%B-75-MMFX. The analytical displacement for column CRC-2%C-75-MMFX at Blast 35 for a DIF of 1.1 and 1.0 were 16.7 mm and 17.3 mm respectively. Comparing these results to the experimental value of 23.89 mm resulted in an error of 30% & 27.5%. For Blast 80, the analytical displacements at Blast 80 for a DIF of 1.1 and 1.0 were 61.2 mm and 56.8 mm. Both cases under predicted the experimental displacement value of 87.94 mm with an error of 30.4% for a DIF=1.1 and 35.4% for a DIF=1.0. Potential sources of error include the lack of well-established stress-strain and DIF models for high strength steel - further research is recommended.

Table 6.6 Summary of analysis results for column CRC-2%B-75-MMFX and CRC-2%C-75-MMFX (Blast 35-80)

		Idealized Shockwave			Maximum Mid-span Displacement			
		P _r (kPa)	t _{df} (msec)	I _r (kPa*msec)	D _{exp} (mm)	D _{anls} (mm)	D _{anls} /D _{exp}	% Error
		DIF=1.1						
CRC-2%B-75-MMFX	Blast 35	39.5	22.6	445.4	20.39	18.5	0.91	9.3%
CRC-2%C-75-MMFX	Blast 35	37.0	21.6	399.1	23.89	16.7	0.70	30.0%
	Blast 80	72.0	21.6	779.3	87.94	61.2	0.70	30.4%
		DIF=1.0						
CRC-2%B-75-MMFX	Blast 35	39.5	22.6	445.4	20.39	18.9	0.93	7.3%
CRC-2%C-75-MMFX	Blast 35	37.0	21.6	399.1	23.89	17.3	0.72	27.5%
	Blast 80	72.0	21.6	779.3	87.94	56.8	0.65	35.4%

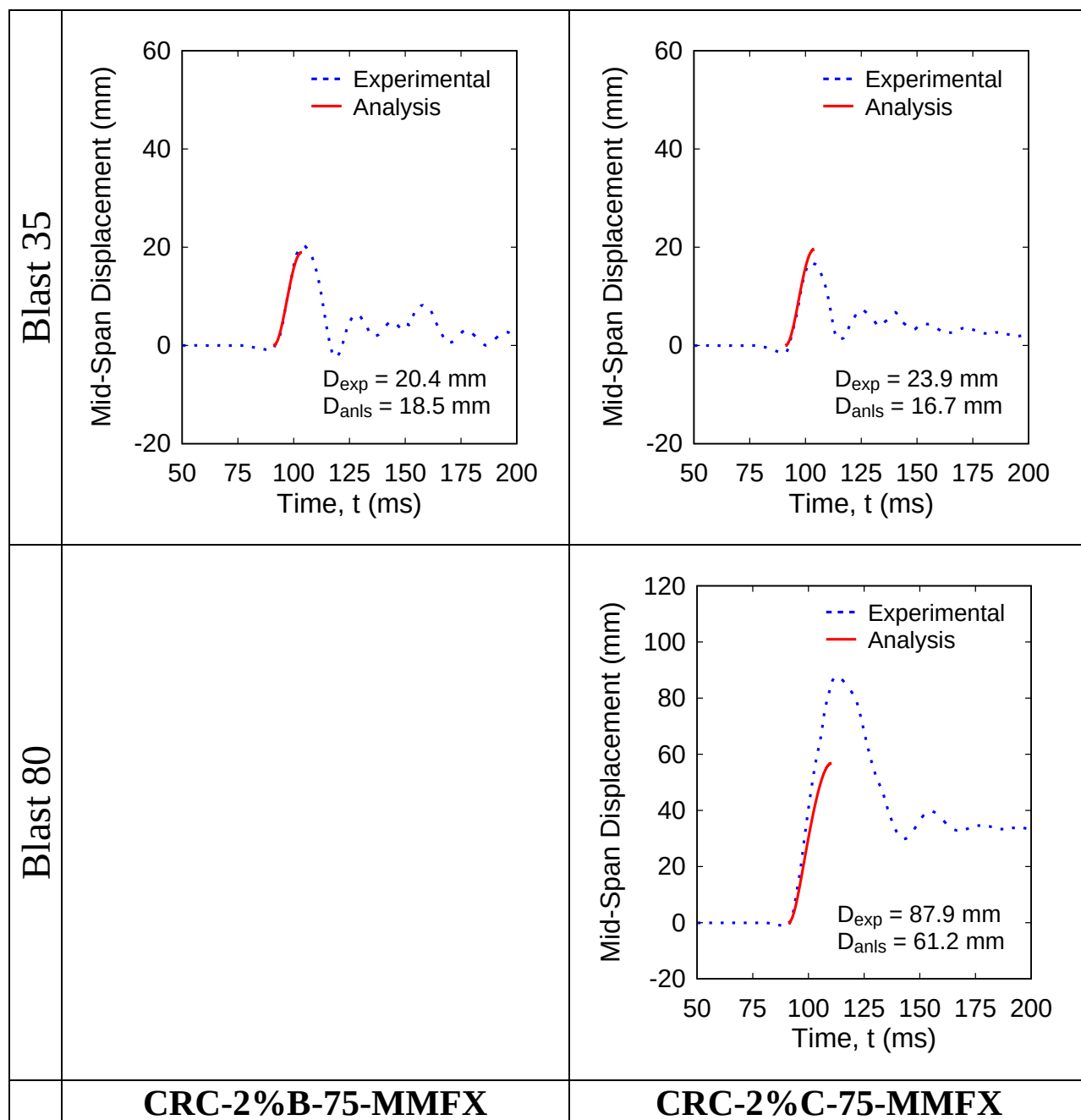
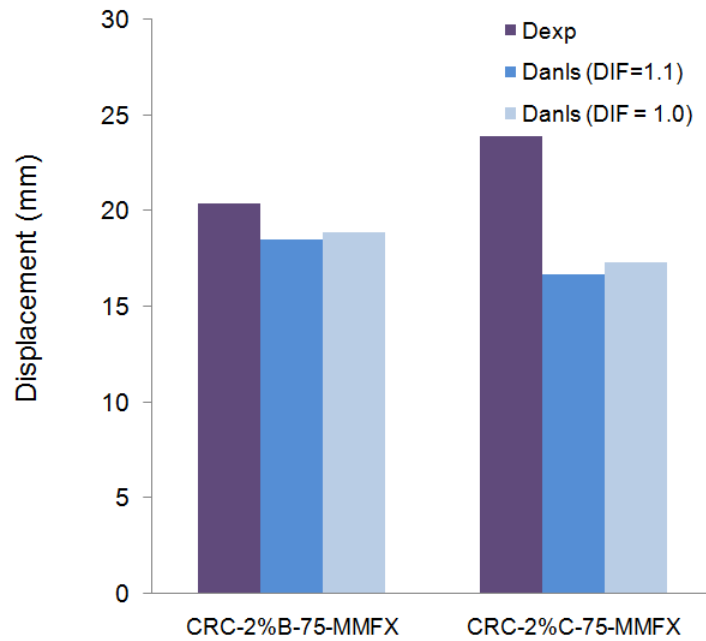
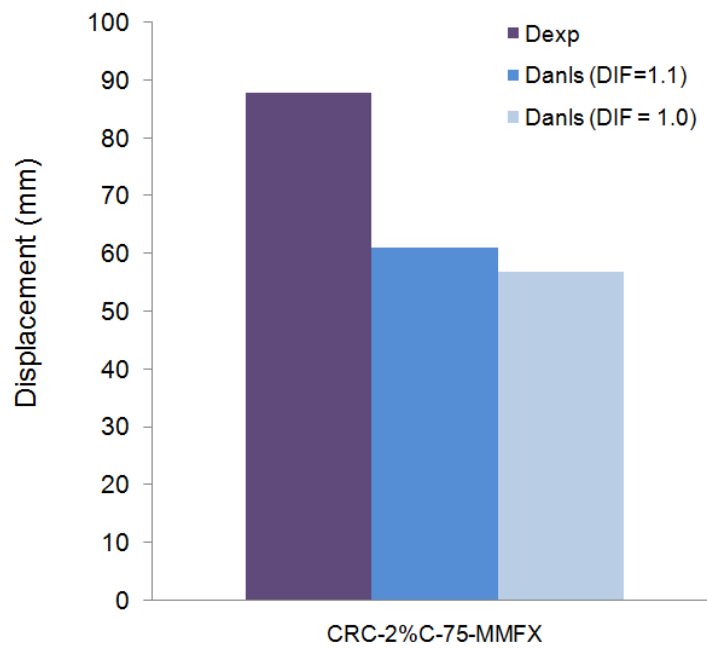


Figure 6.11 Displacement predictions time-history column CRC-2%B-75-MMFX and CRC-2%C-75-MMFX
(Blast 35-80)



(a) Blast 35 displacements results



(b) Blast 80 displacements results

Figure 6.12 Comparison of displacement predictions at Blast 35-80 for columns CRC-2%B-75-MMFX and CRC-2%C-75-MMFX for the cases of $DIF_t=1.0$ and 1.1

6.5.3.2 Seismically Detailed MMFX

This section discusses the analysis results for columns CRC-2%B-38-MMFX and CRC-2%C-38-MMFX which were built with -seismic detailing and high-strength MMFX reinforcement. The analytical prediction results for the maximum mid-span displacements are summarized in Table 6.7. Displacement vs. Time histories of the two columns for Blast 35 and Blast 80 are shown in Figure 6.13, while Figure 6.14 shows a comparison bar chart of both DIF cases.

The results of the analysis for these columns are generally more accurate when compared to those obtained for the non-seismic specimens discussed in the previous section. The analytical displacements of specimen CRC-2%B-38-MMFX at Blast 35 for a DIF of 1.1 and 1.0 were 19.3 mm and 19.6 mm respectively. Comparing these results to the experimental value of 16.86 mm resulted in errors of 30% & 27.5%. The displacements for Blast 80 were also over-predicted in both cases with analytical displacement predictions of 54.4 mm for DIF=1.1, and 51.2 mm for DIF=1.0. These displacements resulted in errors of 17.5% and 10.6% respectively. In the case of specimen CRC-2%C-38-MMFX, the analytical displacements at Blast 35 for DIF of 1.1 and 1.0 were 17.5 mm and 18.2 mm, respectively. Both cases under-predicted the experimental displacement of 20.48 mm resulting in errors of 14.4% and 11.1%, respectively. At Blast 80, the analytical displacements for DIF of 1.1 and 1.0 were 61.8 mm and 57.1 mm, respectively, under predicting the experimental displacement of 65.05 mm and resulting in errors of 4.9% and 12.2%.

Table 6.7 of analysis results for column CRC-2%B-38-MMFX and CRC-2%C-38-MMFX (Blast 35-80)

		Idealized Shockwave			Maximum Mid-span Displacement			
		P _r (kPa)	t _{df} (msec)	I _r (kPa*msec)	D _{exp} (mm)	D _{anls} (mm)	D _{anls} /D _{exp}	% Error (%)
		DIF=1.1						
CRC-2%B-38-MMFX	Blast 35	39.5	25.2	497.6	16.86	19.3	1.14	14.2%
	Blast 80	72.0	21.2	764.2	46.31	54.4	1.17	17.5%
CRC-2%C-38-MMFX	Blast 35	38.4	21.1	405.7	20.48	17.5	0.86	14.4%
	Blast 80	71.9	21.9	787.1	65.05	61.8	0.95	4.9%
		DIF=1.0						
CRC-2%B-38-MMFX	Blast 35	39.5	25.2	497.6	16.86	19.6	1.16	16.1%
	Blast 80	72.0	21.2	764.2	46.31	51.2	1.11	10.6%
CRC-2%C-38-MMFX	Blast 35	38.4	21.1	405.7	20.48	18.2	0.89	11.1%
	Blast 80	71.9	21.9	787.1	65.05	57.1	0.88	12.2%

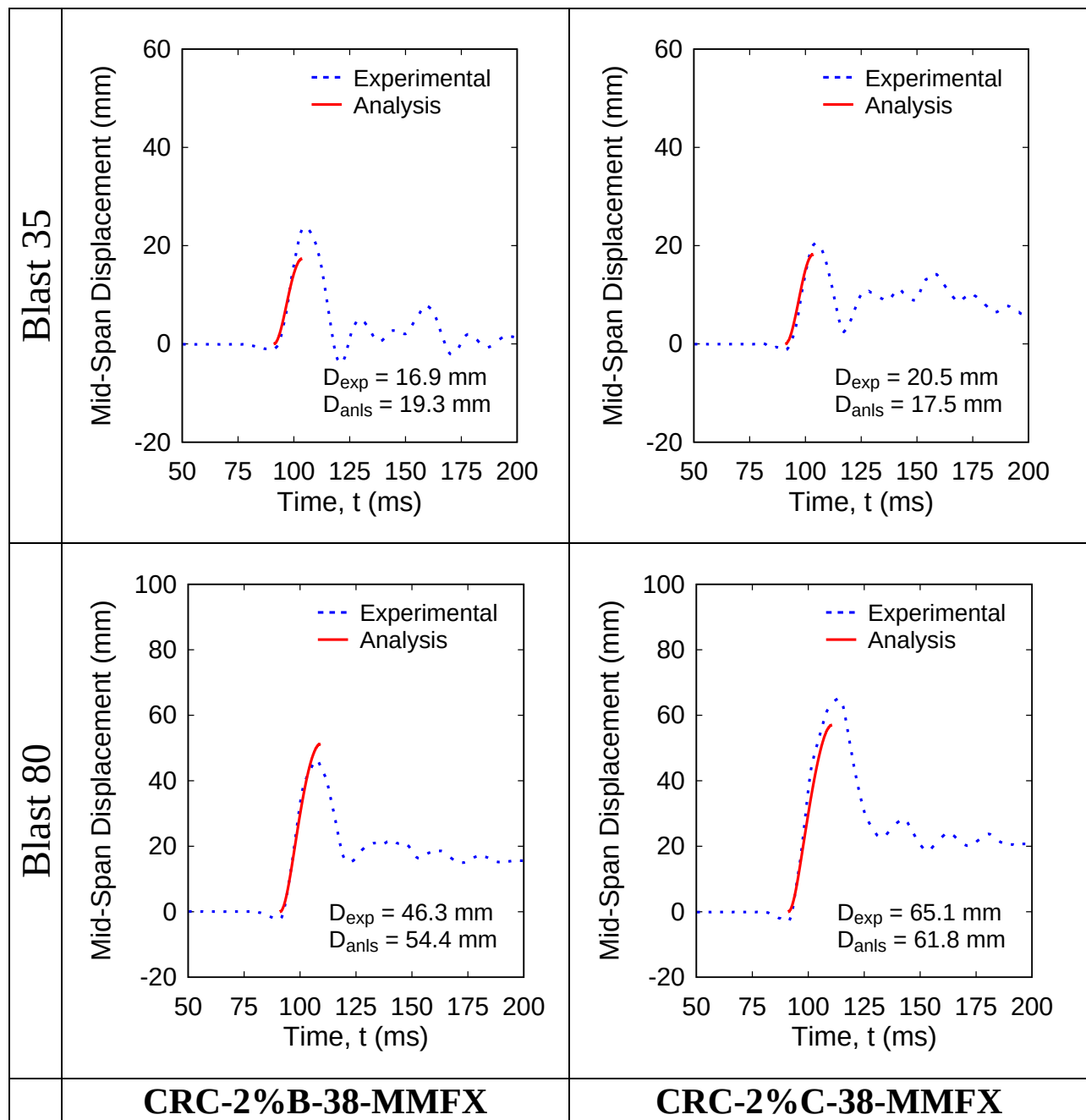
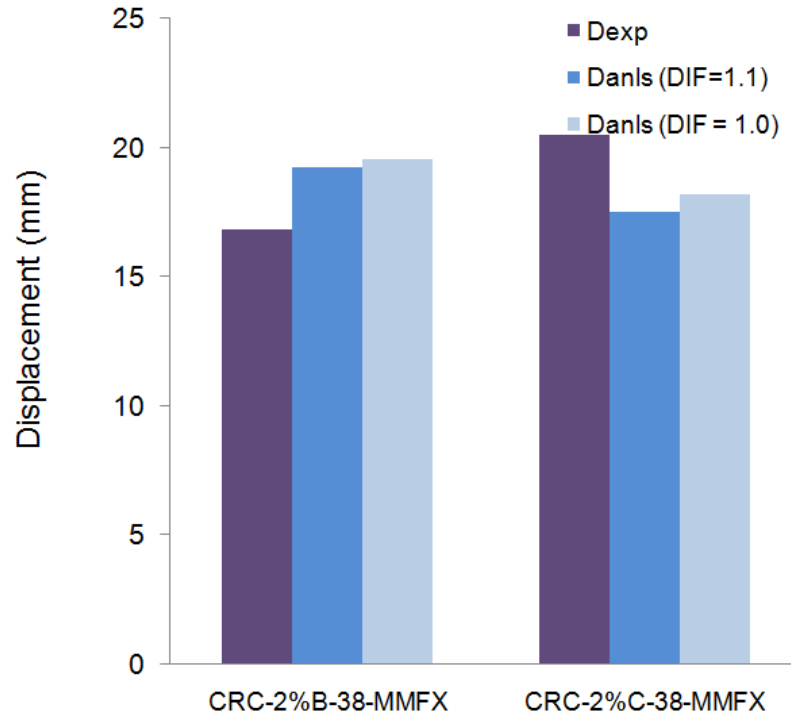
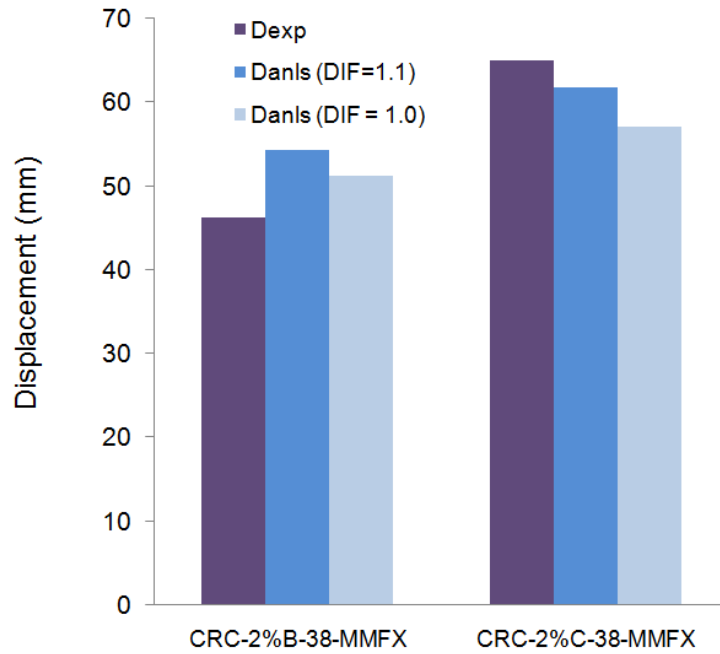


Figure 6.13 Displacement predictions time-history column CRC-2%B-38-MMFX and CRC-2%C-38-MMFX
(Blast 35-80)



(a) Blast 35 displacements results



(b) Blast 80 displacements results

Figure 6.14 Comparison of displacement predictions at Blast 35-80 for columns CRC-2%B-38-MMFX and CRC-2%C-38-MMFX for the cases of $DIF=1.0$ and 1.

6.5.3.3 Series 2- Overall Results

Experimental and analytical maximum mid-span displacements ($D_{\max}$ and D_{anls} respectively) and maximum displacement ratios ($D_{\text{anls}}/D_{\max}$) for the series 2 columns, CRC-2%B-75-MMFX, CRC-2%C-75-MMFX, CRC-2%B-38-MMFX and CRC-2%C-38-MMFX, at Blasts 35 & 80 pressure levels are presented in Table 6.8 and Table 6.9 . As mentioned earlier, no dynamic models for high-strength steel are available in the literature; therefore analysis was conducted assuming no dynamic increase factor ($DIF_t=1.0$), in addition to a separate case of $DIF_t=1.1$ which corresponds with the DIF at ultimate for regular grade steel at a strain-rate of 1 s^{-1} . The tables also report statistical data relating to the accuracy of predicted results for maximum displacement ratios (standard deviation, mean and coefficient of variation). Overall, the results show improved predictions between the computed and recorded response quantities when compared to Series 1. Considering all blast scenarios, the average displacement ratio ($D_{\text{anls}}/D_{\max}$) for the Series 2 columns is 0.92, with a percent error of 17.2%, and corresponding standard deviation (Std Dev) of 0.19 and coefficient of variation (CoV) of 20.8%. Blast 35 resulted in average displacement ratio $D_{\text{anls}}/D_{\max} = 0.90$ (with Std Dev = 0.18 and CoV = 20.2%). The analysis results are generally similar at Blast 80 loading, with an average displacement ratio $D_{\text{anls}}/D_{\max} = 0.94$ (with Std Dev = 0.23 and CoV = 25.0%).

Table 6.8 Summary of analysis results for Series 2 columns with DIF=1.1 (Blast 35-100)

		Idealized Shockwave			Maximum Mid-span Displacement				
		P _r	t _{df}	I _r	D _{exp}	D _{anls} DIF=1.1	D _{anls} /D _{exp}	% Error	
		(kPa)	(msec)	(kPa*msec)	(mm)	(mm)		(%)	
CRC-2%B-75-MMFX	Blast 35	39.5	22.6	445.4	20.39	18.5	0.91	9.3%	
CRC-2%C-75-MMFX	Blast 35	37.0	21.6	399.1	23.89	16.7	0.70	30.0%	
	Blast 80	72.0	21.6	779.3	87.94	61.2	0.70	30.4%	
CRC-2%B-38-MMFX	Blast 35	39.5	25.2	497.6	16.86	19.3	1.14	14.2%	
	Blast 80	72.0	21.2	764.2	46.31	54.4	1.17	17.5%	
CRC-2%C-38-MMFX	Blast 35	38.4	21.1	405.7	20.48	17.5	0.86	14.4%	
	Blast 80	71.9	21.9	787.1	65.05	61.8	0.95	4.9%	
Statistical Analysis					Blast #	Mean	Std Dev	COV	% Error
					35-80	0.92	0.19	20.8%	17.2%
					35	0.90	0.18	20.2%	17.0%
					80	0.94	0.23	25.0%	17.6%

Table 6.9 Summary of analysis results for Series 2 columns with DIF=1.0 (Blast 35-100)

		Idealized Shockwave			Maximum Mid-span Displacement				
		P _r	t _{df}	I _r	D _{exp}	D _{anls} DIF=1.0	D _{anls} /D _{exp}	% Error	
		(kPa)	(msec)	(kPa*msec)	(mm)	(mm)		(%)	
CRC-2%B-75-MMFX	Blast 35	39.5	22.6	445.4	20.39	18.9	0.93	7.3%	
CRC-2%C-75-MMFX	Blast 35	37.0	21.6	399.1	23.89	17.3	0.72	27.5%	
	Blast 80	72.0	21.6	779.3	87.94	56.8	0.65	35.4%	
CRC-2%B-38-MMFX	Blast 35	39.5	25.2	497.6	16.86	19.6	1.16	16.1%	
	Blast 80	72.0	21.2	764.2	46.31	51.2	1.11	10.6%	
CRC-2%C-38-MMFX	Blast 35	38.4	21.1	405.7	20.48	18.2	0.89	11.1%	
	Blast 80	71.9	21.9	787.1	65.05	57.1	0.88	12.2%	
Statistical Analysis					Blast #	Mean	Std Dev	COV	% Error
					35-80	0.90	0.19	20.5%	17.2%
					35	0.93	0.18	19.6%	15.5%
					80	0.88	0.23	26.1%	19.4%

6.5.4 Sources of error

When considering the analysis results in Sections 6.5 there are several potential sources of error which could have led to inaccuracies with the predictions for maximum displacement.

One potential source of error includes inaccuracies associated with the stress-strain and strain-rate models considered for UHPFRC in the analysis. For example there are no confinement models specifically developed for UHPFRC available in the literature. Furthermore the accuracy of the simplified unconfined compression and tension models needs further study through comparison and calibration with larger sets of material testing data. In terms of the strain-rate models, researchers have shown that the Malvar and Ross, (1998) model, which was originally developed for normal-strength conventional concrete, may produce inaccurate DIF_t estimates for UHPFRC (Milard et al. 2010). Similarly, as noted earlier there is a lack of DIF models for high-strength steel reinforcement. Further research is recommended in order to develop accurate stress-strain and strain-rate models that can be used in the blast analysis of UHPFRC columns.

Another potential source of error relates to the choice of plastic hinge length in the analysis. In the current study the plastic hinge length (L_p) at midspan was assumed to be equal to the column effective depth (d), based on assumptions used in the analysis of conventional reinforced concrete columns. Examination of the post-blast photos of the UHPFRC columns shows that this assumption may not be valid for UHPFRC. In addition, it is noted that the columns suffered from a 3 point failure mechanism with no distinct formation of plastic hinge at extreme blast pressures; this failure mode differs from the assumptions made in the lumped inelasticity approach.

Finally, in the current study the effect of accumulated damage under repeated loading was not considered as a test parameter. Earlier tests conducted on conventional reinforced concrete columns by Khadom et al. (2012) have indicated that repeated loading can result in approximately 15% increase in column deformations at failure pressures. Further research is recommended to study the effect of accumulated damage in UHPFRC columns.

7 Conclusion & Recommendations for Future Research

7.1 Conclusions

The thesis presented the results from twelve UHPFRC columns tested under blast loading. The following conclusions can be drawn from this study:

- The results showed that the use of UHPFRC significantly improves the blast resistance of columns, resulting in a better control of maximum and residual displacements at equivalent blast loads, and an ability to sustain larger blast loads before failure;
- The use of UHPFRC had an important effect on failure mode and resulted in increased damage tolerance in the columns, with an ability to eliminate secondary blast fragments;
- The use of fibers with optimized properties (increased aspect-ratio, increased length and tensile strength) led to improvements in the blast performance of the UHPFRC columns, with reductions in maximum and residual displacements at equivalent blasts loads;
- The use of UHPFRC in combination with seismic detailing was shown to enhance the blast performance of the columns, resulting in reduced displacements at equivalent blast pressures, and an ability to sustain larger blast loads before failure;
- Increasing the fiber content from 2% to 3% was shown to improve the blast performance of the UHPFRC columns, resulting in reduced displacements at equivalent loads and an ability to sustain increased blast pressures before failure;
- The failure mode for all UHPFRC columns containing 10M reinforcement was tension rebar rupture. This mode of failure may be linked to the high compressive strength of UHPFRC which results in the development of large tensile strains, leading to rupture of the tension steel reinforcement. The use of larger 15M bars was shown to be an effective means of delaying rupture of tension steel;
- The use of UHPFRC in combination with high-strength steel (MMFX) was shown to enhance the blast performance of the columns, resulting in reduced displacements at equivalent blast pressures;
- SDOF analysis can be used to predict the blast response of UHPFRC columns with reasonable accuracy, however further research is recommended to develop stress-strain and strain-rate models that can be used in the blast analysis of UHPFRC columns.

7.2 Recommendations for Future Research

The following recommendations for future research would be beneficial to further the understanding on the behaviour of UHPFRC columns under blast loading:

- The high strength steel used in this research program was of American #3 size ($A_b = 71 \text{ mm}^2$), smaller than the regular grade steel rebar in the companion column with 10M reinforcement ($A_b = 100 \text{ mm}^2$). Further testing on columns having 10M MMFX steel reinforcement should be conducted to allow for direct comparison with the regular grade steel column tests performed in this study. It would also be beneficial to examine the effect of increased high-strength steel reinforcement ratio on UHPFRC column blast performance.
- The columns in this study were subjected to repeated loads; further research is needed to study the effect of accumulated damage on the blast response of UHPFRC columns.
- Further investigation of the effects of fibre type along with fibre anchorage properties such as hooked-end, crimped and twisted fibres, on UHPFRC column performance.
- Investigate the effect of high strain rates on UHPFRC in compression and tension with the purpose of developing proper DIF models that could be used for analysis and design.
- Determination of optimal fibre content for specific member sizes and reinforcement detailing to offer proper recommendation for design. These recommendations would be needed to avoid placement difficulties during casting depending on the size of the member along with its reinforcement details.

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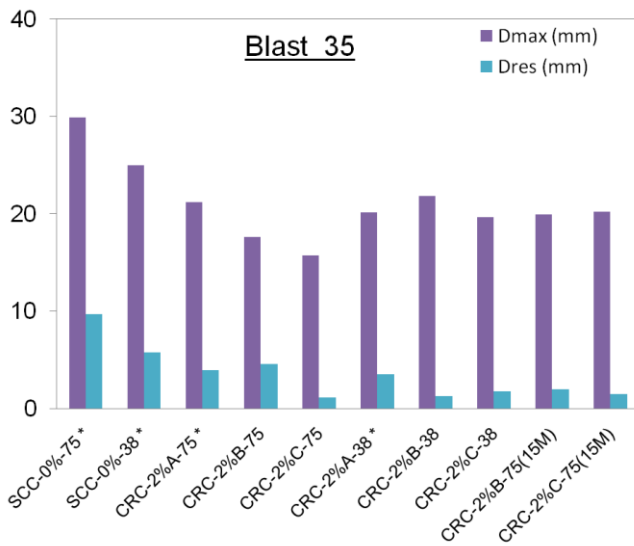
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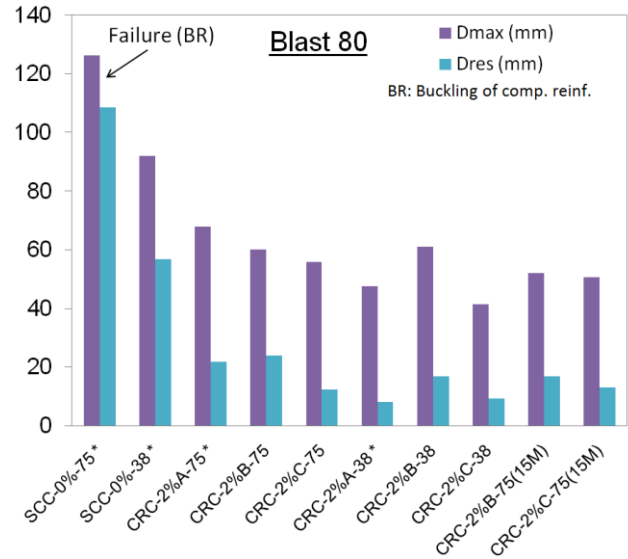
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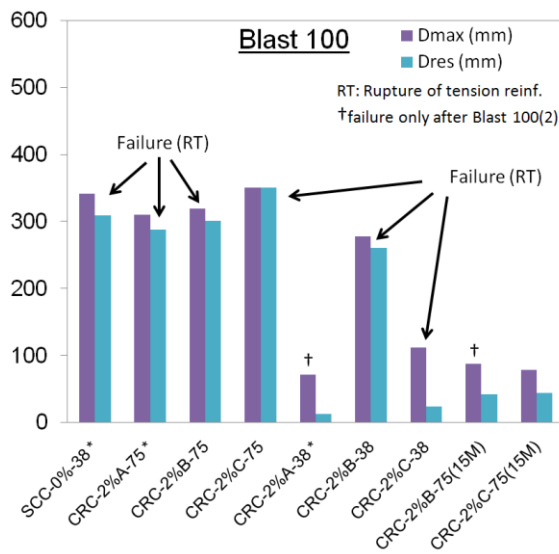
9 Appendix



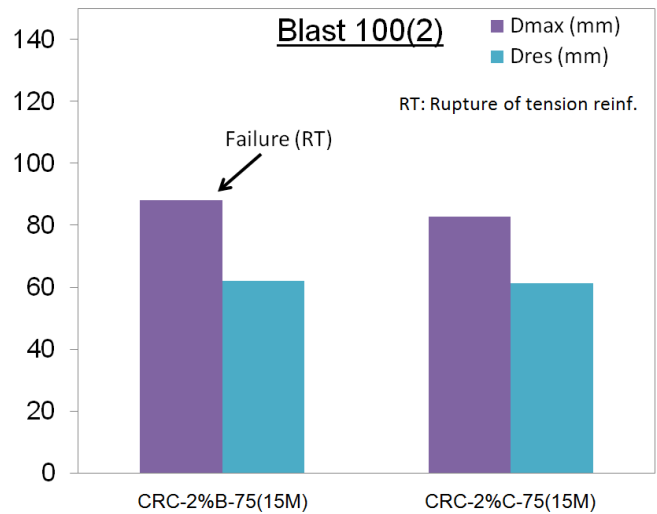
(a)



(b)



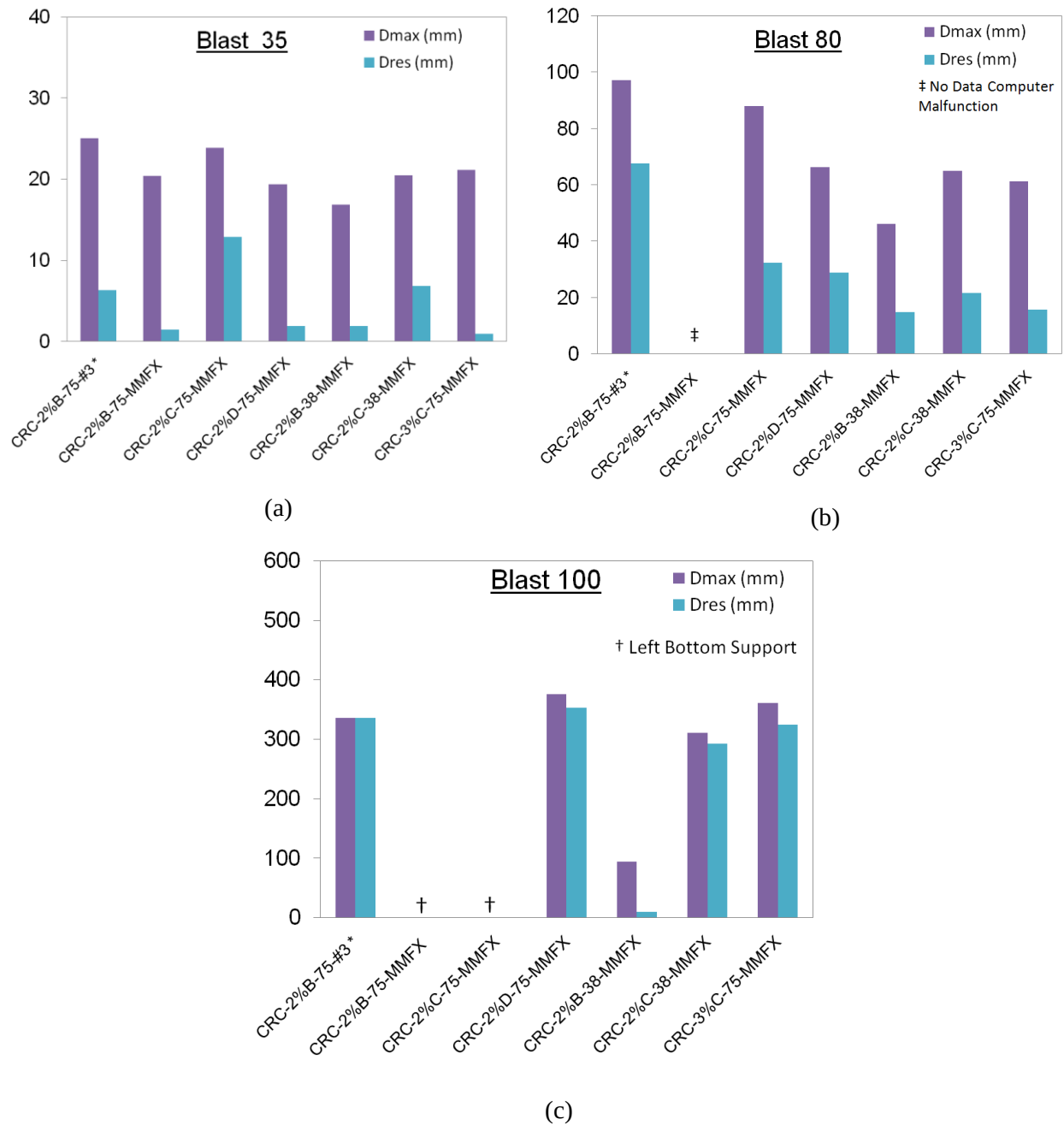
(c)



(d)

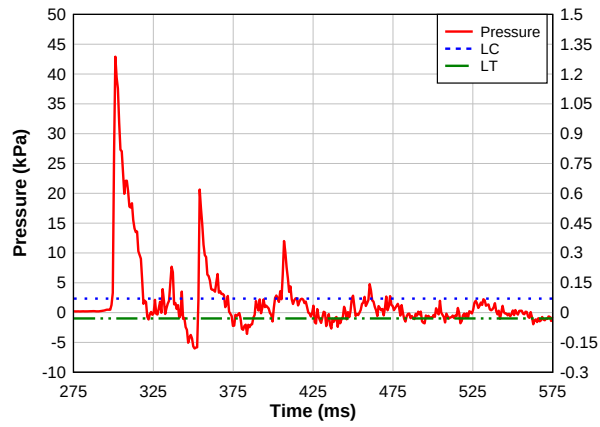
*Specimens tested by Burrell (2012)

Figure 9.1 Series 1: Maximum& residual displacements: a) blast 35, b) Blast 80, c) blast 100 and d) Blast 100(2)

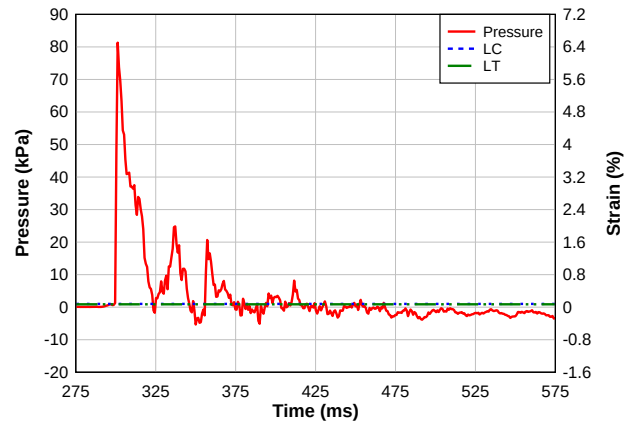


*Specimen tested by De Carufel (2015)

Figure 9.2 Series 2: Maximum& residual displacements: a) blast 35, b) Blast 80 and c) blast 100 and d) Blast 100

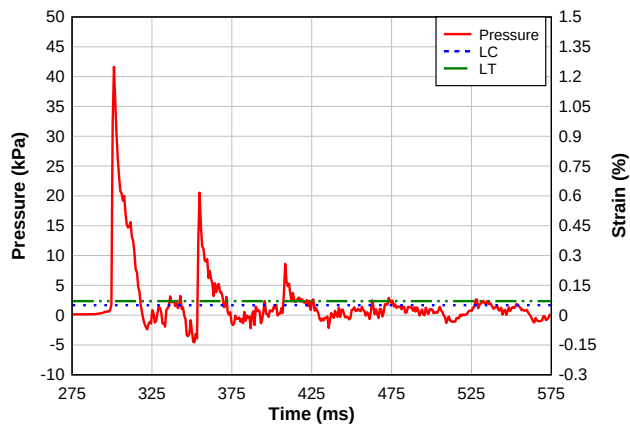


Blast 35

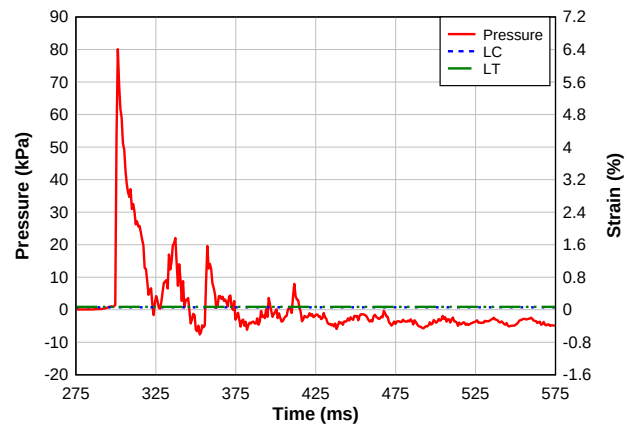


Blast 80

Figure 9.3 Strain-time histories for column CRC-2%B-75 at Blast 35 and Blast 80

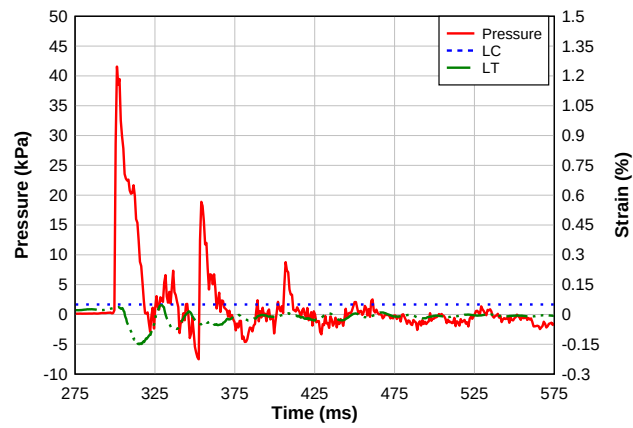


Blast 35

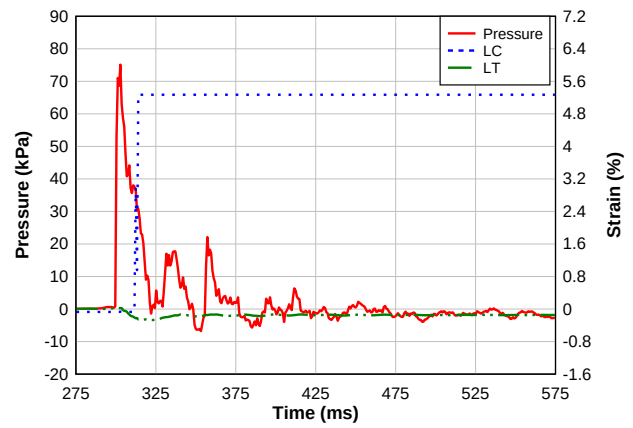


Blast 80

Figure 9.4 Strain-time histories for column CRC-2%C-75 at Blast 35 and Blast 80

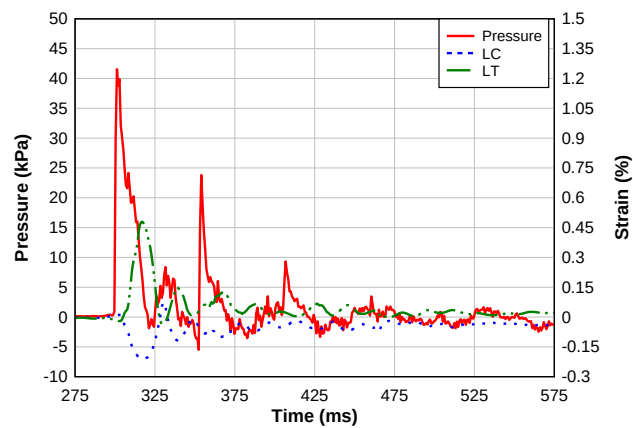


Blast 35

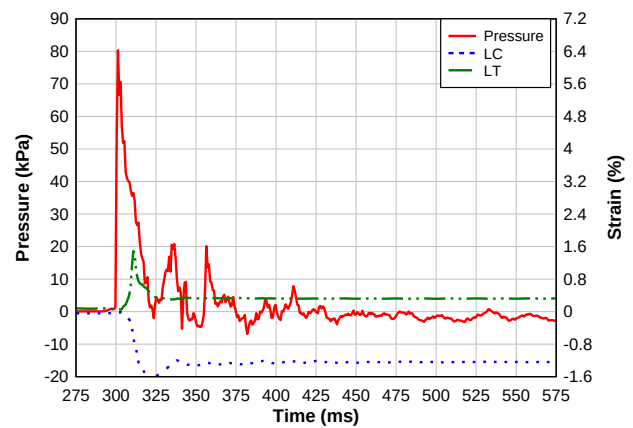


Blast 80

Figure 9.5 Strain-time histories for column CRC-2%B-38 at Blast 35 and Blast 80

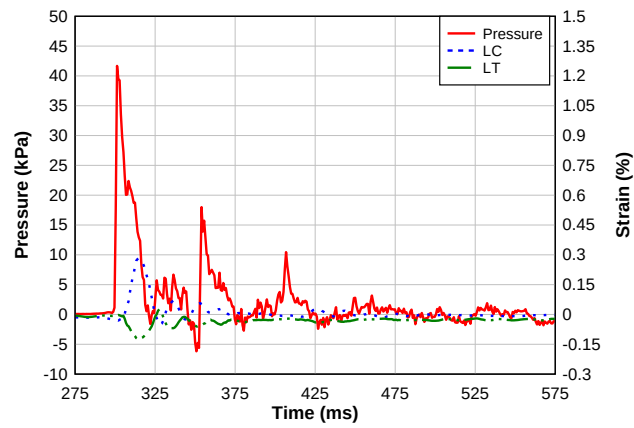


Blast 35

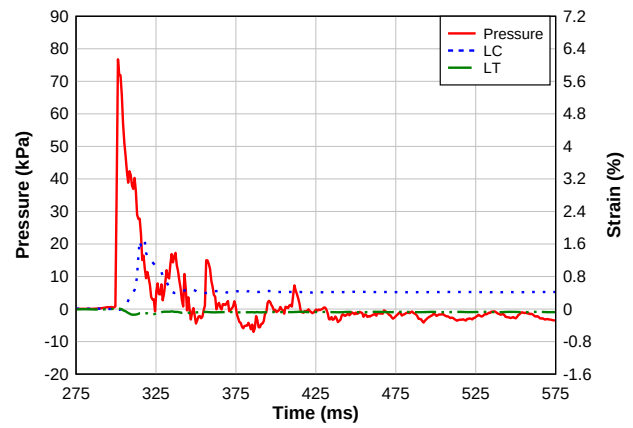


Blast 80

Figure 9.6 Strain-time histories for column CRC-2%C-38 at Blast 35 and Blast 80

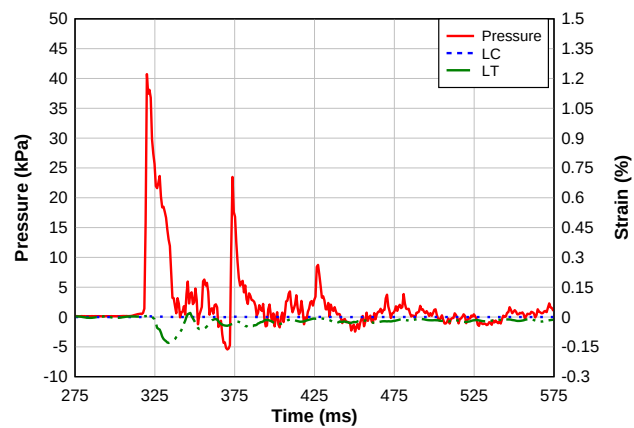


Blast 35

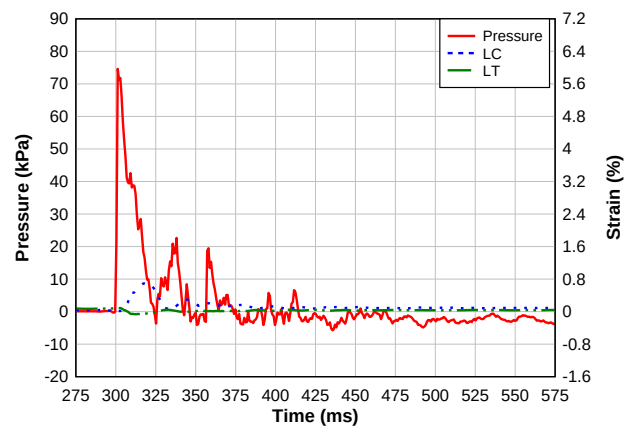


Blast 80

Figure 9.7 Strain-time histories for column CRC-2%B-75-15M at Blast 35 and Blast 80

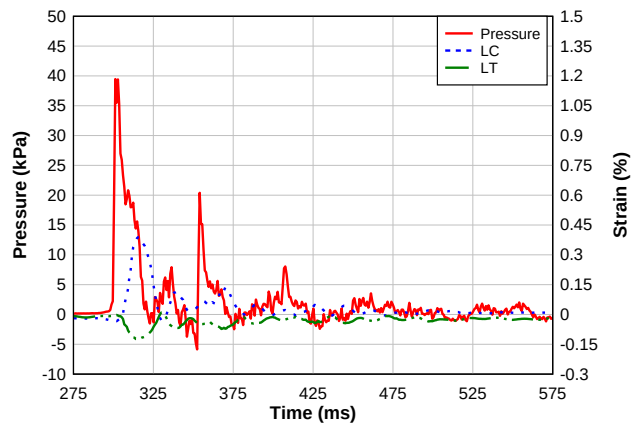


Blast 35



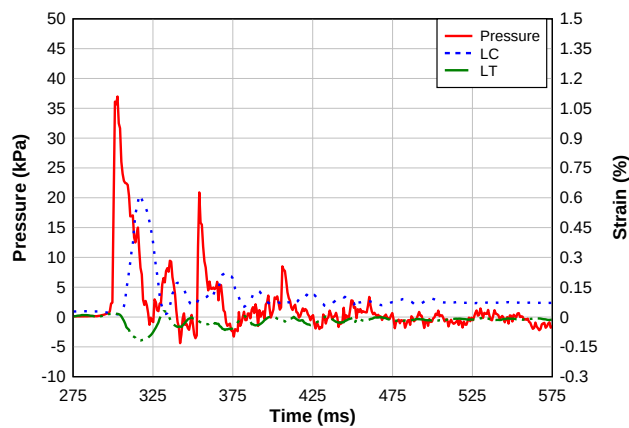
Blast 80

Figure 9.8 Strain-time histories for column CRC-2%C-75-15M at Blast 35 and Blast 80

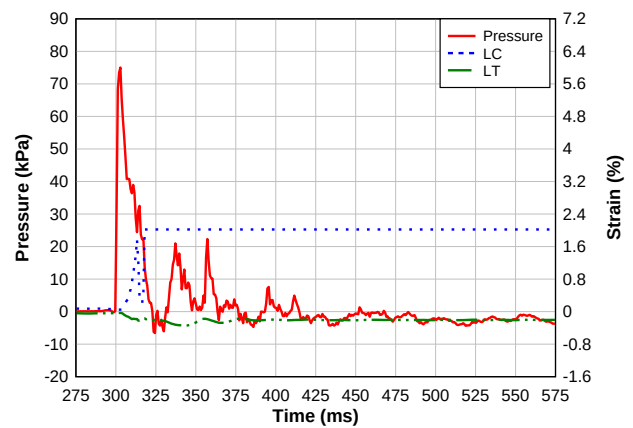


Blast 35

Figure 9.9 Strain-time histories for column CRC-2%B-75-MMFX at Blast 35 and Blast 80

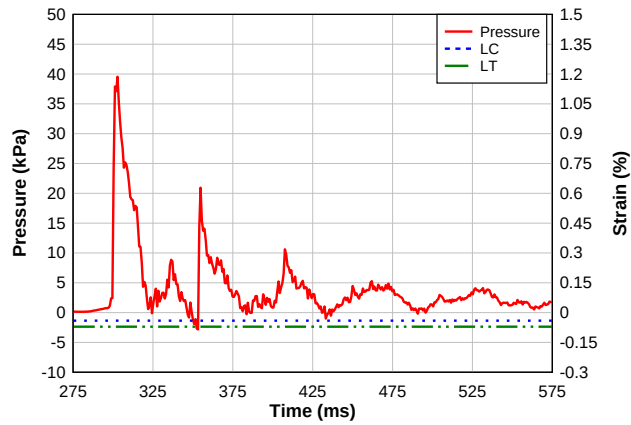


Blast 35

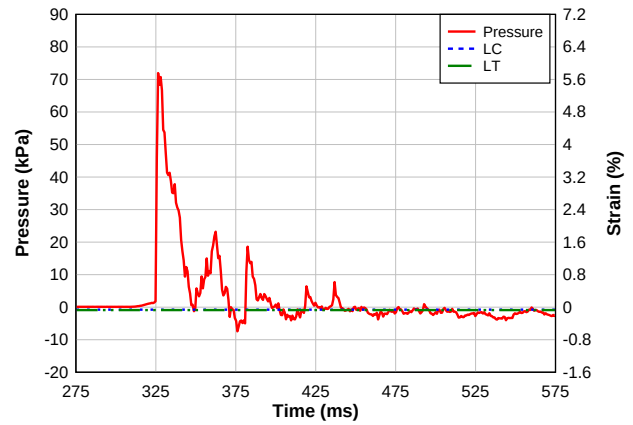


Blast 80

Figure 9.10 Strain-time histories for column CRC-2%C-75-MMFX at Blast 35 and Blast 80

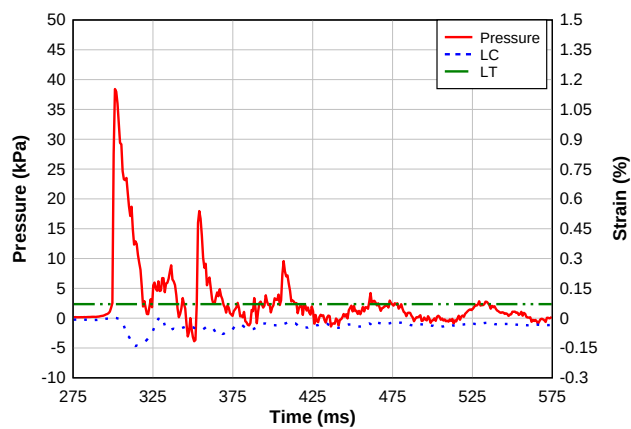


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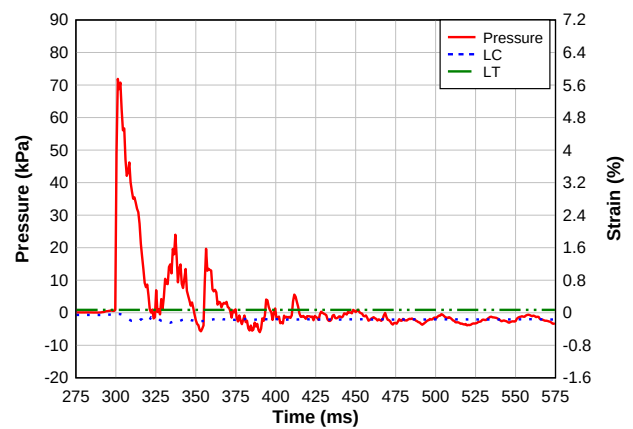


Blast 80

Figure 9.11 Strain-time histories for column CRC-2%B-38-MMFX at Blast 35 and Blast 80

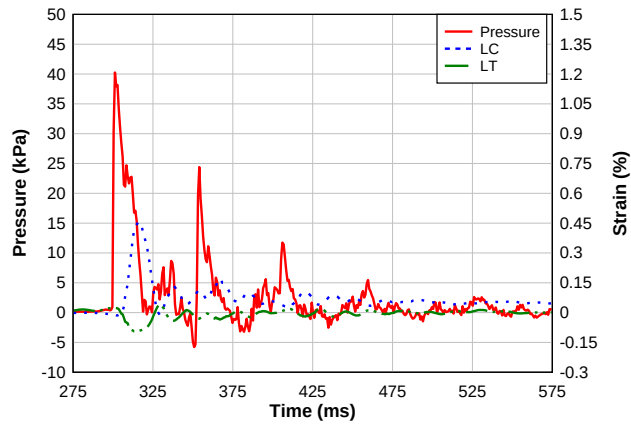


Blast 35

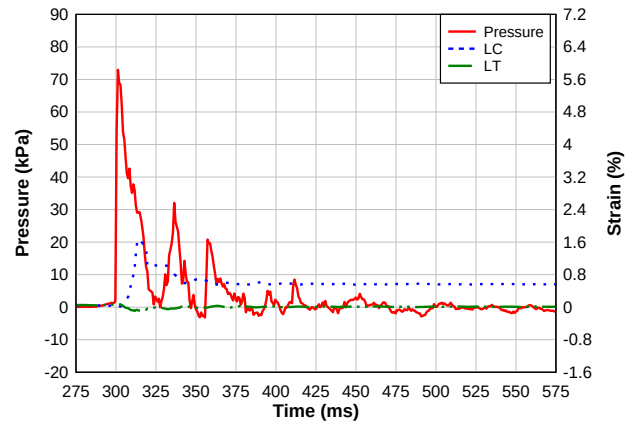


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Figure 9.12 Strain-time histories for column CRC-2%C-38-MMFX at Blast 35 and Blast 80

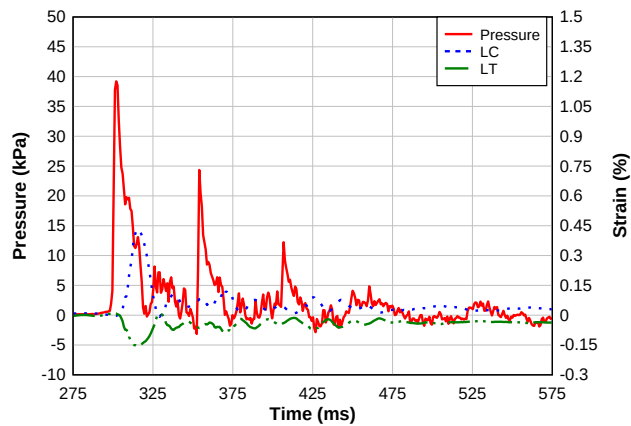


Blast 35

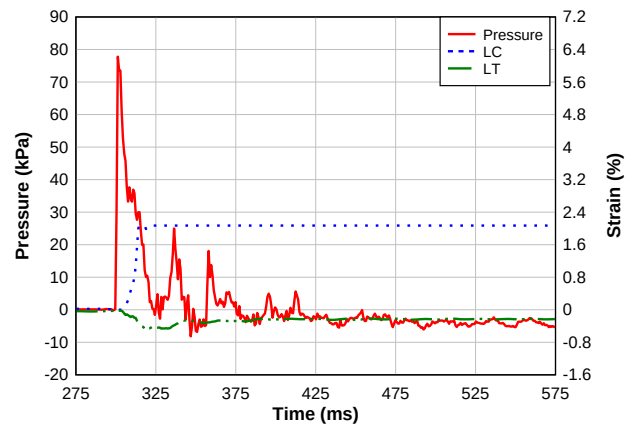


Blast 80

Figure 9.13 Strain-time histories for column CRC-3%C-75-MMFX at Blast 35 and Blast 80



Blast 35



Blast 80

Figure 9.14 Strain-time histories for column CRC-2%D-75-MMFX at Blast 35 and Blast 80