

Blast Performance of Ultra-High Performance Concrete Beams Tested Under Shock-Tube Induced Loads

By

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Abstract

Modern day structures are reaching higher, spanning longer and undergoing new design methods. In addition to regular loads, it is becoming increasingly important to consider the potential risks of intentional and accidental explosions on structures. In the case of reinforced concrete buildings, critical elements such as beams and columns must be designed with sufficient strength and ductility to mitigate against the effects of blast loads to safekeep the public and prevent progressive structural collapse. Recent advancements in structural materials have led to the development of ultra-high-performance concrete (UHPC) with high compressive strength, tensile resistance, toughness and energy absorption capacity, properties which are ideal for blast protection of structures. Combining UHPC with high-performance steels, such as and high strength reinforcement is another potential solution to enhance the blast resilience of structures. This experimental and analytical research program investigates the advantages of combining high performance materials to increase the blast capacity of reinforced concrete beams. The experimental program includes tests on 21 beam specimens, fourteen of which are subjected to extreme blast loading using the University of Ottawa shock-tube, with seven companion specimens tested statically. Parameters investigated include: effect of concrete type (NSC vs. UHPC), effect of steel reinforcement type (NSR vs. HSR), effect of longitudinal reinforcement ratio, effect of fiber type/content and effect of transverse reinforcement on structural performance under static and dynamic loads. The experimental study includes three series having specified material combinations as follows: series 1 (NSC & NSR), series 2 (UHPC & NSR) and series 3 (UHPC & HSR). Each dynamically tested beam specimen is subjected to gradually increasing blast shockwaves until reaching failure. Performance assessment criteria included; maximum and residual displacements, overall blast resistance and resistance to secondary fragmentation.

Results show that the specimens detailed with UHPC can resist greater blast loads with reduced mid-span displacement and debris generation when compared to beams built with conventional concrete. The combination of UHPC and high strength reinforcement further enhances blast performance and delays failure as both high strength materials balance themselves for optimum efficiency. Similarly, for specimens subjected to static loading, the use of UHPC increased the maximum load resisted by the beams, although failure mode alters from concrete crushing to rebar rupture. The combination of UHPC and high strength reinforcement further heightens beam resistance, at the expense of reduced specimen ductility.

The analytical component of this thesis presents an analysis program called *UO Resistance* which is capable of predicting structural element resistance curves and conducting a dynamic inelastic single degree of freedom (SDOF) analysis of members subjected to blast loads. Resistance curves generated using *UO Resistance* were compared to data obtained through static testing and were found to effectively predict specimen response. Similarly, dynamic analysis methods implemented in *UO Resistance* prove to be effective at predicting specimen response under blast load. Additionally, a sensitivity analysis was performed to evaluate the effect of various modeling parameters on the static and SDOF dynamic predictions of specimen response.

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Notations

Symbol	Definition
A	Area impacted by the blast pressure
d_b	Steel reinforcement bar diameter
d_f	Fibre diameter
E_c	Modulus of elasticity, concrete
E_s	Modulus of elasticity, steel
f_{ctf}	Fibre reinforced concrete tensile stress
f_{cu}	Unconfined concrete stress
f_{cuf}	Unconfined fibre reinforced concrete stress
f'_c	Compressive concrete strength
f'_{cu}	Unconfined compressive concrete strength
f'_{cuf}	Unconfined compressive fibre reinforced concrete strength
f'_{dc}	Design compressive concrete strength
f_{du}	Design steel ultimate stress
f_{dy}	Design steel yield stress
F_e	Equivalent Dynamic Load
$f_{fct,eq}$	Equivalent flexural strength
f_s	Steel stress
f_{sh}	Steel strain hardening stress
f_u	Steel ultimate stress
f_y	Steel yield stress
F_m	Mean load
I_r	Reflected impulse
K_{LM}	Load mass transformation factor
L_d	Driver length
l_f	Fibre length
m	Total mass of the system
P_r	Reflected pressure
P_d	Driver pressure
R	Resistance of the member
t_p	Positive phase duration
u	Deflection at mid-height
$\ddot{u}$	Acceleration at mid-height
v_f	Fibre volume ratio
ε_{20cu}	Unconfined concrete strain at 20% of peak stress
ε_{50cu}	Unconfined concrete strain at 50% of peak stress
ε_{cu}	Unconfined concrete strain
ε_{cuf}	Unconfined fibre reinforced concrete strain
ε'_{cu}	Unconfined concrete strain at peak stress
ε'_{cuf}	Unconfined fibre reinforced concrete strain at peak stress
ε_s	Steel strain
ε_{sh}	Steel strain hardening strain

ε_u	Steel ultimate strain
ε_y	Steel yield strain
$\dot{\varepsilon}$	Strain rate
$\dot{\varepsilon}_s$	Static strain rate
δ_{anls}	Analytical displacement
δ_{exp}	Experimental displacement
δ_{max}	Maximum displacement
δ_{res}	Residual displacement
θ_{max}	Maximum support rotation
τ_{bond}	Matrix bond strength

Acronyms

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

Chapter 1. Introduction

1.1. General Context

Modern day structures are constantly spanning longer, rising higher and are evermore at risk of extreme dynamic loading. Structural engineers must now account for the possible risk of chemical explosions, accident blasts and terrorist attacks on important infrastructure in order to ensure the public's safety. Recent incidents such as the Oklahoma City bombing (1995), the Toronto Sunrise Propane explosion (2008) and the Lac-Mégantic explosion (2013) illustrate the need for the design of blast resistant structures.

The development of new high and ultra-high performance materials such as ultra-high performance concrete (UHPC) and high strength reinforcement (HSR) offers engineers an opportunity to mitigate the risks posed by extreme blast loading. The enhanced compressive and tensile properties of UHPC in combination with the high-performance properties of HSR may offer important enhancements in blast resistance, energy absorption and overall structural ductility when compared to normal strength materials. Present day structural design standards do not fully account for the full potential of such high performance materials as a result of insufficient research data. Thus, this research program aims at providing new data and observations on both the static and dynamic response of UHPC beams detailed with conventional and high strength reinforcement to further understand the benefits that are possible with the use of these innovative materials in structural and extreme load applications.

1.2. Research Objectives

The primary objective of this research program is to complete an experimental parametric investigation in order to assess the possibility of implementing innovative concrete and steel materials in reinforced concrete beams subjected to extreme blast loading. The study examines the static and blast load response of a total of 21 beam specimens built with UHPC and high-strength reinforcement and examines the effects of various design parameters such as: concrete type, fiber properties, longitudinal steel reinforcement type, reinforcement ratio, and effect of transverse reinforcement. The advantages of utilizing high performance materials will be examined by comparing their static and dynamic response to that of a control series of specimens constructed using conventional normal strength materials. A second objective of the research program is to predict the static and dynamic response of the test beams using non-linear analysis.

1.3. Scope of Work

A total of twenty-one (21) beam specimens were designed and tested at the University of Ottawa Structures and Blast Laboratories; 7 beams were tested statically under four-point bending and 14 beam specimens were tested using the University of Ottawa's shock-tube. The design parameters investigated in the study include:

- The effect of concrete strength, comparing normal strength concrete (NSC) and ultra-high performance fiber reinforced concrete (UHPFRC);
- The effect of steel reinforcement type; comparing deformed normal strength reinforcement to ASTM A1035 Grade 100 high strength reinforcement (HSR);
- The effect of reinforcement ratio; comprised of beams detailed with 2-#4, 2-#5, 2-#6, 2-15M and 2-20M longitudinal reinforcement bars;
- The effect of steel fiber content; including normal strength concrete specimens with no fibers and UHPFRC specimens having volumetric fiber contents of: 1%, 2% and 3%;
- The effect of transverse steel reinforcement, comparing beam specimens having stirrups spaced at 100mm in the shear span to control specimens having no transverse reinforcement.

Specimens tested under quasi-static four-point bending are assessed in terms of: load resistance, ductility, failure mechanism and crack patterns. The dynamic testing is conducted using the University of Ottawa shock-tube, with specimens subjected to gradually increasing blast loads. Performance of the dynamically tested specimens is assessed based on; magnitude of the shockwave required to cause failure, maximum and residual mid-span displacements, damage and cracking, fragmentation control and failure mechanism. As part of the analytical study, a sectional analysis and single degree of freedom dynamic analysis software program called *UO Resistance* was developed in order to predict the response of the test specimens.

1.4. Thesis Breakdown

The content of this document is sub-divided into nine chapters, organized as described below:

Chapter 1 – Introduction:

- Presents the context and objectives of the research program and describes the scope of work.

Chapter 2 – Literature Review:

- Summarizes previous research conducted on the static, impact and blast performance of UHPFRC beams.

Chapter 3 – Experimental Program:

- Provides an overview of the specimen designs, materials properties, test setups and testing procedures used in the research program.

Chapter 4 – Results – Static Tests:

- Presents the detailed experimental results from the static tests completed for each specimen (beam by beam), including photographs, load-displacement relationships, crack patterns and steel strain histories.

Chapter 5 – Discussion of Static Beam Results:

- Compares and discusses the static test results with a focus on the effects of concrete type, longitudinal steel reinforcement type, reinforcement ratio, and transverse reinforcement.

Chapter 6 – Results – Dynamic Tests:

- Presents the detailed results from the experimental dynamic tests completed for each specimen (beam by beam) including: photographs, pressure-impulse time histories, pressure-displacement time histories, damage progression and steel reinforcement strains.

Chapter 7 – Discussion of Dynamic Beam Results:

- Compares and discusses the dynamic test results with a focus on the effects of concrete type, longitudinal steel reinforcement type, reinforcement ratio, volumetric fiber content, fiber type and effect of transverse reinforcement.

Chapter 8 – Static and Dynamic Analytical Predictions:

- Presents the constitutive material models, static and dynamic analysis procedures implemented in the developed *UO Resistance* software, and presents analytical predictions of specimen responses, complimented with a parametric sensitivity analysis.

Chapter 9 – Conclusion:

- Summarizes the findings and highlights of the research program and proposes recommendations for future research.

Chapter 2. Literature Review

2.1. Chapter Overview

The following chapter provides a literature review related to the research topic which focusses on the blast performance of ultra-high performance concrete (UHPC) beams built with conventional and high-strength reinforcement. The first part of the literature presents an overview of research which has been conducted on the static response of UHPC beams in flexure and shear. Thereafter, previous studies on the behaviour of UHPC materials under high strain rates, and the impact and blast resistance of UHPC structural components and beams are reviewed.

2.2. Previous Research on the Static Response of UHPC Beams

Important research has been conducted on the flexural and shear response of UHPC beams under static loading. Table 2-1 presents a summary of previous studies, with information on material properties, specimen details and test setup, alongside an overview of parameters investigated. Further details are provided in the following sub-sections.

2.2.1. Flexural Response of UHPC Beams

Beams with ordinary steel reinforcement

Yang et al. (2010) examined the flexural behaviour of 14 UHPC beam specimens having rectangular cross-section under four-point loading. The main parameters investigated relate to the longitudinal steel reinforcement ratio, the steel reinforcement distribution as well as the placement of the UHPC during casting. Results show that an increase in reinforcement ratio from 0.6% to 1.2% significantly improved the ductility index ($\frac{\Delta u}{\Delta y}$) from 1.79 to 2.64 in the beams. The post-cracking stiffness and peak resisted load of specimens detailed with a greater reinforcement ratio was found to be considerably greater than those of specimens with inferior reinforcement ratios. Rebar distribution was also found to have an impact on peak load resisted and specimen ductility. Specimens having two layers of reinforcement exhibited a lower peak load but greater ductility when compared to companions with the same amount of steel but having only one layer of reinforcement. Lastly, the authors note that by placing the concrete from the end of the beam rather than the middle, the peak load resisted can be increased by up to 15%. Similar observations have been made by Yang et al. (2010).

Dong & Feng (2013) completed a research program on the flexural performance of 15 UHPC beam specimens having rectangular section under three-point loading. The study assessed the effect of volumetric fiber content (five fiber contents from 0% to 2%) and longitudinal steel reinforcement ratio (0.93% to 9.25%) and their impact on beam response and ductility. Beams generally showed an increase in peak capacity as the reinforcement ratio or fiber content was increased. Findings

show that to enhance the ductility of a UHPC beam specimen a minimum volumetric fiber content of 0.7% is required. Likewise, beam specimens included in this research program having a volumetric fiber content of 2% and a reinforcement ratio inferior to 4% displayed a ductile flexural response. However, specimens detailed with 2% volumetric fiber content having a longitudinal reinforcement ratio greater than 4% experienced brittle failure mechanisms.

Randl & Mészoly (2015) examined the flexural and shear response of 10 UHPC beams having I-shaped cross-section and high-strength longitudinal reinforcement under four-point bending. Most of the beams had circular shaped openings (“web-holes”) along their length. Principle parameters investigated included the effect of high strength reinforcement ratio (3% or 5% to induce different failure modes) and volumetric fiber content (1-2%). Results show that ductile flexural failures were achieved for the “under-reinforced” specimens, while diagonal tension shear failure was observed in the case of “over-reinforced” beams. In the under-reinforced series, the fiber content did not greatly affect peak capacity or ultimate deflection but affected failure mode. Under-reinforced specimens having 1% fibers displayed significant concrete crushing in the compression zone soon after specimen yielding. Conversely, specimens designed with 2% fibers experienced rupture of the high strength longitudinal reinforcement with considerably less concrete damage.

Yoo et al. (2016) investigated the flexural performance of four UHPC beams having rectangular cross-section under four-point bending. The series included one unreinforced specimen and three specimens with low amounts of longitudinal steel reinforcement which ranged from 0.53% to 1.71%. The principle research objective was to examine the effect of reinforcement ratio on the cracking response, load-carrying capacity and ductility of the UHPC specimens. The authors note that in contrast to normal strength concrete beams, a greater network of cracks was developed in the UHPC beams, further distributing the load. Moreover, the UHPC beams only experienced vertical micro-cracks until the peak load was reached (due to presence of fibers), at which point the load carrying capacity was gradually decreased with the widening of a specific principle crack. Furthermore, specimens detailed with a higher reinforcement ratio presented a greater post-cracking stiffness, yielding load and peak load capacity, although the first cracking load reduced (because of restrained shrinkage). Cracking in the compressive zone, number of cracks and crack spacing were also affected by the increase in reinforcement ratio. The authors presented a method to predict the moment-curvature response of the UHPC beams using AFGC and JSCE recommendations and sectional analysis incorporating a linear compression model and direct tension model obtained from inverse analysis of notched prisms.

Beams with prestressed reinforcement

Yang & Fang (2009) tested a total of 4 UHPC prestressed T-beam specimens under four-point cyclic loading. The researchers examined the effect of level of pre-stressing of the steel tendons and the effect of normal strength steel reinforcement. The ductility index of the prestressed beams was found to reduce as the effective prestressing force became higher. Furthermore, by increasing the pre-stressing ratio of jacking stress to guaranteed rupture strength from 0% to 65%, the failure mechanism is altered from concrete crushing followed by eventual tendon rupture, to sudden tendon rupture at yielding. Correspondingly, at a pre-stressing ratio to 65, the specimen deflection capacity was reduced by 72% when compared to a non-prestressed specimen. Results show that by providing additional normal strength longitudinal reinforcement in addition to the pre-stressed tendons, the post-cracking stiffness and peak load resisted is increased. The research paper also presents an analytical approach to predict the flexural response of the research program beam specimens and exhibited good agreement with experimental results.

Yang et al. (2011) studied the flexural response of four large-scale prestressed beams subjected to four-point bending. The main parameters investigated are crack propagation and failure mechanism. Results demonstrate that with the use of UHPC, minor flexural cracks form and continue to distribute the imposed loading. A similar trend was found in all specimens; crack widths remained similar until attaining peak load, at which point one specific crack widened exponentially. Displacement concentration at the location of principle crack eventually led to steel fiber pullout and specimen failure. The authors also proposed an analytical procedure which takes tensile softening into account using an inverse analysis based on the load-crack mouth opening displacement relationship. The ratios of flexural capacity from numerical analysis to the experimental results fluctuated between 0.81 to 1.21, indicating reasonable agreement between the tests and analysis.

Beams with composite UHPC layers

A few studies have also examined the potential of using UHPC layers (in the tension or compression zone) to enhance the response of reinforced concrete beams. Noshiravani & Bruhwiler (2010) explored the potential of strengthening a reinforced concrete beam with a 50 mm thick UHPFRC reinforced layer. The main parameters investigated were the UHPFRC reinforcement ratio and the effect of composite layers on reinforced concrete beam response. Findings demonstrate that the addition of a UHPFRC layer having a 6.6% steel reinforcement ratio significantly enhances the response and ensured a ductile flexural response. Moreover, the effect on longitudinal reinforcement type played a crucial role in the response of the beam element, as the smooth reinforcement bars led to a purely flexural failure mechanism, whereas the identical specimen with deformed rebar experienced a flexural-shear failure. Overall, providing an additional UHPFRC layer improved beam deformation capacity as well as overall peak load resistance. Similar observations have been made by Noshiravani & Bruhwiler (2013).

2.2.2. Shear Response of UHPC Beams

Several researchers have also investigated the shear response of UHPC beams, both analytically and experimentally. Summaries of three of the experimental studies are provided below, further details on other investigations can be found in Table 2-1.

Previous Research on the Shear Response UHPC Beams

Telleen et al. (2010), studied the static response of 2 UHPC I-beam specimens under three-point loading. The primary parameter investigated was the effect of transverse reinforcement on the shear response of the fiber-reinforced UHPC beams. The research program was also complimented with an analytical study focused on shear predictions based on a truss model approach. Findings show that in order to ensure a ductile flexural failure, transverse reinforcement should be provided, as the specimen reinforced with stirrups spaced at 150mm throughout the shear span experienced a maximum displacement approximately 3.5 greater than the specimen without shear reinforcement. Shear response predictions yielded acceptable results, however the authors noted that further experimental work should be conducted in order to validate the analytical approach.

Voo et al. (2010), investigated the shear performance of 8 UHPFRC pre-stressed I beam elements having no transverse reinforcement. The main parameters studied were the shear span to depth ratio, steel fiber type and volumetric fiber content. Results obtained in this study exhibit that by providing adequate volumetric fiber content, such that strain hardening after tensile cracking is initiated, results in significant distribution of shear cracking before the failure diagonal shear crack develops. Additionally, by increasing the volumetric steel fiber content from 1.0% to 1.5% the peak resisted load increased from 450kN to 530kN. Lastly, it was found that by increasing the shear span to effective depth ratio from 2.5 to 4.5, there was a significant decrease in shear capacity of approximately 62%.

Baby et al. (2014) studied the shear behaviour of 11 I-shaped UHPFRC beam specimens subjected to four-point bending. The main parameters investigated consist of: type of shear reinforcement (stirrups and/or fibers, or neither) and the type of longitudinal steel reinforcement (pre-stressed or ordinary). Interestingly, the results show that a UHPFRC beam specimen's shear capacity is dependant on: the shape of the cross-section, orientation of steel fiber reinforcement and applied pre-stressing force. Additionally, the only specimens to experience a flexural failure mechanism were those detailed with a combination of longitudinal and transverse steel reinforcement. Beam specimens constructed with steel fibers and no transverse steel reinforcement delayed the shear failure which resulted in multiple diagonal cracks, as opposed to specimens without steel fibers or stirrups which suffered from a sudden and principle shear crack. Ultra high performance concrete beams having steel fibers resisted peak loads up to 250% greater than non-fiber reinforced ultra high performance concrete beams. The study also found that beams reinforced with short steel fibers experienced peak loads up to 15% greater than that of beams having long steel fibers.

Table 2-1 Summary of Static Response of UHPC Beams Literature (1/2)

Focus of study	Study	No. of beams	Beam type		Beam dimensions				f_c' (MPa)	Tension steel			Transverse steel		Fibers		Parameters investigated							
			Ordinary or Pre-stressed?	Cross-section type	B (mm)	H (mm)	L (mm)	a/d ratio		Yield strength (MPa)	Reinf. Ratio (%)	Prestressing Yield Strength (MPa)	Yield strength (MPa)	Reinf. Ratio (%)	Fiber type	Fiber content (%)	Fiber content	Beam depth	a/d ratio	Tension reinf. Ratio	Shear reinf. ratio	High-strength steel	Distribution of steel	Composite UHPC layers
Flexure	(Yang & Fang, 2009)	4	O/P	I	100	300	4800	5.9	127-138	394	4.6	1820-1942	-	-	SF	1.5			X	X			X	
	(Yang, Joh, & Kim, 2010)	14	O	R	180	270	2900	4.2-5.2	190-197	400	0-1.96	-	-	-	SF	2.0			X	X			X	
	Yang et al. (2011)	4	P	I	200	600-1300	4500-7000	2.5-4.5	158.2-188.4	-	0.31-0.33	-	-	-	SF	2.0		X	X	X				
	Dong & Feng (2013)	15	O	S	200	200	2100	6.5	155-180	400	0.93-9.25	-	-	-	SF	2.0-4.0	X			X				
	Han and Xiangguo (2014)	15	O	R	200	200	2400	5.9	>150	400	0.36-3.6	-	-	-	SF	0-2.0	X			X			X	
	(Randl & Meszoly, 2015)	10	O	I	200	350	3500	3.4	148-167	800-1100	2.96-5.11		550	1.86	SF	1.0-2.0	X			X		X		
	(Yoo, Banthia, & Yoon, 2016)	4	R	R	200	270	2500	4.4-4.7	196.7	400	0-1.71	-	-	-	SF	2.0			X	X			X	
Shear	(Tellean, Noshiravani, Galrito, & Bruhwiler, 2010)	2	O	I	110	200	1000	2.8	147-152	500	4.00		500	0.94	SF	0.6					X			
	Voo etl (2010)	8	P	I	500	650	8600	1.8-4.5	122-140	-	2.64	-	-	-	SF	1.0-1.5	X		X					
	Ciprian et al. (2011)	12	O	I	120	223	1600	1.25	>150	400	2.00	-	-	-	SF	1.5-2.55	X							
	Xia et al. (2011)	5	O	R	40	110	350	1.75	154-172	400	2.50	-	-	-	-				X	X				
	Yang, Joh, & Kim (2012)	12	P	I	500	700	3720-4700	2.5-3.4	160-190	-	-	-	-	-	SF	1.0-2.0	X		X					
	(Baby, Marchant, & Toutlemonde, 2014)	11	O/P	I	270	380	3000	2.5	149-212	551	1.95-4.27	1772	561	0.14	SF	2.5-4.7	X			X	X			

I: (I-Beam) ; R: (Rectangular Beam) ; S: (Square Beam); SF: (Steel Fibre) ; O: (Ordinary non pre-stressed reinforcement) ; P : (pre-stressed reinforcement)

Table 2-2 Summary of Static Response of UHPC Beams Literature (2/2)

Focus of study	Study	No. of beams	Beam type		Beam dimensions				F'c (MPa)	Tension steel			Transverse steel		Fibers		Parameters investigated							
			Ordinary or Pre-stressed?	Cross-section type	B (mm)	H (mm)	L (mm)	a/d ratio		Yield strength (MPa)	Reinf. Ratio (%)	Prestressing Yield Strength (MPa)	Yield strength (MPa)	Reinf. Ratio (%)	Fiber type	Fiber content (%)	Fiber content	Beam depth	a/d ratio	Tension reinf. ratio	Shear reinf. ratio	High-strength steel	Distribution of steel	Composite UHPC layers
Composite	(Noshiravani & Bruhwiler, 2010)	6	O/P-C	R	150	265-300	1600	3.1-3.4	42-150	500-710	6.6	-	-	0.09-0.17	SF	-		X	X	X	X			X
	(Noshiravani & Bruhwiler, 2013)	2	O/P-C	R	150	300	2980	3.0	41.5-150	565-710	3.9-7.3	626	-	0.09-0.15	SF	-					X			X

I: (I-Beam) ; R: (Rectangular Beam) ; S: (Square Beam); SF: (Steel Fibre); O: (Ordinary non pre-stressed reinforcement) ; P : (pre-stressed reinforcement)

2.3. Previous Research on the Impact and Blast Response of UHPC

This section summarizes previous research which has examined the behaviour of ultra-high performance concrete under impact or blast loading.

2.3.1. Material Response of UHPC at high-strain rates

Several researchers have examined the material response of ultra-high performance fiber reinforced concrete under high strain-rates, mostly from tests on small-scale specimens tested under impact. Research on flexural response using drop-weight and drop hammer testing has been conducted by Millard et al. (2009) and others. The strain-rate behavior of UHPC in direct tension has also been studied by Wile et al. (2012). Testing under higher strain rates in tension and compression using Split-Hopkin Pressure Bar (SHPB) has been conducted by Rong et al. (2010), Lai (2008) and others. Limited research exists on the behavior of CRC (the UHPC used in this research) at higher strain-rates, however the material's behavior under impact loads was studied by Bindiganavile et al. (2002). Overall, this research has demonstrated the superior performance of UHPC under dynamic impact and high-strain loading when compared to conventional concrete. This research has also confirmed the strain-rate sensitivity of UHPC which shows increased strength under dynamic tension, compression and flexure. Table 2-3 provides a summary of some of previous investigations alongside the main conclusions from each study.

Table 2-3 Summary of UHPC Impact Response at High Strain Rates.

Study	No. of Specimens	Specimen Type	Type of test	Strain rate s^{-1}	f'_c (MPa)	Fiber type	Fiber content (%)	Conclusions
Lai et al. (2008)	6	Cyl,	Split Hopkinson Pressure Bar	35.5-139.5	134-211	S	0-4	- Elastic modulus and peak stress of UHPC increase as the strain rate increases - By increasing fibre content, material damage is reduced and the repeated impact resistance of UHPC is improved.
Millard et al. (2009)	8	Cyl. Cubes & Prisms	Drop Hammer	1.0-1.66	125-200	S	1.5-6.0	- Flexural dynamic increase factor relation was developed for design purposes (strain rate vs. DIF). - No dynamic increase factor should be attributed to the shear resistance at high loading rates.

Study	No. of Specimens	Specimen Type	Type of test	Strain rate s^{-1}	f'_c (MPa)	Fiber type	Fiber content (%)	Conclusions
Rong et al. (2010)	12	Cyl. & Prisms.	Split Hopkinson Pressure Bar	25.9-92.9	135-225	S	0.0-3.0	- Impact resistance of UHPC is improved with increased volumetric steel fibre content. - Peak compressive strength is increased with higher strain rates. - Numerical analysis concludes that the Johnson Holmquist concrete material constitutive model can be used for dynamic compression predictions.
Wile et al. (2012)	12	Dog Bone	Direct Tension	0.0001-0.1	230-250	S	2.0-3.0	- Increased post-cracking strength up to 20% under dynamic loading. - Increased energy absorption capacity up to 40% under dynamic loading. - Energy absorption and strain-hardening modulus increased with an increase in volumetric fibre content. - Authors note UHPC is ideal material for moderate strain-rate applications (e.g. seismic).
Pyo et al. (2016)	12	Dog Bone	Direct Tension	0.0001-24	180	S	1.5	- Long smooth steel fibres produce the highest tensile resistance at high strain rates - Tensile resistance of UHPC at high strain rates was significantly higher than at a static rate. - Tensile behaviour of UHPC specimens is highly influenced by specimen size, notably when evaluating the post-cracking strength.

2.3.2. Previous Impact Studies on UHPC Beams

The behaviour of UHPC structural members under impact loads has been studied by several researchers. Most of the previous studies have been conducted on slab components. These studies have demonstrated the superior impact performance of UHPC when compared to conventional concrete (increased impact capacity, improved deformation control, enhanced fragmentation resistance). A few researchers have also examined the impact behavior of UHPC beams as summarized in Table 2-4 and detailed below.

Table 2-4: Summary of UHPC Beams Impact Response at High Strain Rates.

Study	No. of Specimens	Specimen Type	Dimensions (BxHxL)	Reinforcement Ratio	Type of test	Impact Weight (kg)	Drop Height (m)	f'_c (MPa)	Fiber type	Fiber content (%)
Bindiganavile et al. (2002)	12	Rect.	0.1x0.1x0.3	-	Drop Hammer	60	0.2-1.0	200	SF	0.75-6.0
Fujikake (2014)	5	I	0.15x0.2x1.7	1.49	Drop Hammer	400	0.8-1.6	214	SF	2.0
Yoo et al. (2015)	4	Rect.	0.2x0.27x2.9	0-2.0	Drop Hammer	270	1.6	197-202	SF	2.0
Hyunh et al. (2015)	16	Rect.	0.15x0.2x2.0 (note: columns)	0-1.3	Drop Hammer	500	1.8	80-161	-	-
Yoo et al. (2017)	8	Rect.	0.15x0.22x2.5	1.5	Drop Hammer	510	0.9	202-212	SF	0-2.0

Bindiganavile et al. (2002) examined the effect of impact loading on unreinforced UHPC prisms using an instrumented drop weight impact machine. A total of twelve UHPC specimens having the dimensions shown in Table 2-4 and made of CRC composite were tested. The authors found that UHPC specimens subjected to impact tests were on average twice as strong as conventional FRC specimens and dissipated three to four times more energy. However, UHPC was also found to be less stress-rate sensitive than conventional FRC. The authors noted that UHPC is an ideal material to implement in structures where resistance to impact loading is required.

Fujikake (2014) studied the impact response of five I-beams using a drop hammer apparatus. Among the parameters investigated are; drop hammer height and effect of concrete type. The results showed that UHPC beams can significantly reduce the maximum deflection under impact loading when compared to conventional concrete (up to 40% improvement). Additionally, the analytical approach presented by the author, which incorporated a two degrees of freedom mass-spring-damper system, produced results that were in good agreement with the experimental results.

Yoo et al. (2017) investigated the impact behaviour of a total of eight rectangular UHPC beams using an instrumented drop-weight impact machine. The authors varied the steel fibre content (0% vs. 2%), fibre type (smooth vs. twisted), and fiber length in the specimens. The results indicated that maximum and residual mid-span displacements under impact loads can be significantly reduced by implementing steel fibres in UHPC, while also improving post-impact beam residual capacity after impact. The beams with the longer fibres showed lower mid-span displacements and increased residual capacities when compared to companions with shorter fibers, correlating with previous research findings (e.g. Pyo et al. (2016)). Hence, the authors conclude that by providing a longer steel fibre, both the impact resistance and residual performance is enhanced. Higher moment capacities were obtained for the beams under impact loading when compared to static loading, however no noticeable differences in moment ratios were observed when comparing beams with different fiber contents, types and lengths.

2.3.3. Previous Blast Studies on UHPC Structural Elements

To the knowledge of the author no previous research exists on the blast performance of UHPC beams, however a few studies have examined the blast performance of UHPC columns and slabs as detailed below and summarized in Table 2-5.

Blast tests of UHPC Columns

Roller et al. (2013) studied the blast response of 10 concrete columns ranging from normal strength concrete to UHPC under live close-in explosives. The circular test columns had a 150mm diameter cross-section and had different normal strength reinforcement ratios ranging from 2% to 8%. Both a retrofit procedure and new design was implemented in this research program. The principle parameters investigated were: concrete compressive strength, flexural reinforcement ratio and the addition of a thin layer of UHPC (energy absorbing material) to existing columns for enhanced ductility and residual load capacities. Results show that implementing a higher performance concrete material such as UHPC significantly increases column post-blast residual capacity (up to 70%). Furthermore, adding a thin layer of energy absorbing material to an existing column considerably increases the residual capacity of the structural member. However, the authors note that more experimental work on a broader range of columns should be completed before prescribing any design considerations.

Li & Wu (2015) examined the effect of scaled distance and axial force on the response of four reinforced ultra-high performance concrete columns. The columns in this study were tested under close-in blasts, and the investigation varied the TNT equivalent charge weight as well as the axial load imposed on the column. The authors found that reinforced UHPC columns have enhanced blast resistance when compared to normal strength reinforced concrete columns. Moreover, the two columns subjected to a blast load while axially loaded with 1000kN experienced a decreased mid-span deflection due to the initial compression stress on the tension side of the lateral blast.

Zhang & Wu (2015) studied the close-range blast behaviour of eight UHPFRC-filled double-skin steel tube (CFDST) columns using live-explosive testing. The column cross-sections were varied;

three square and five circular, with the authors noting that the axial load capacities of both cross-section types were very similar (circular= 4000 kN; square= 4025 kN). The authors developed a unique damage index to evaluate the post-blast residual performance of the specimens. It was found that all specimens had a damage index equal or inferior to 0.4, suggesting that all specimens maintained more than 60% of their axial load capacity after being exposed to an extreme blast load. Lastly, the primary failure mechanism of the square columns was found to be localised buckling at mid span or column extremities, whereas the circular specimens suffered tension steel rupture when subjected to high damage levels.

Xu et al. (2016) examined the blast performance of 14 UHPFRC column specimens under close-in explosive testing. The parameters investigated consisted of concrete strength (UHPFRC vs. HSRC), scaled distance and the axial capacity of the specimens. The authors found that using UHPFRC is an effective means of resisting the shockwaves generated by the explosive charge which enhances the blast resistance substantially. Moreover, it was found that the mid-span deflection of the columns subjected to an axial load were smaller than those without; as the axial load modifies the columns boundary conditions and limits end rotation which out-weigh the P-delta effects.

Previous research undertaken at the University of Ottawa by Burrell (2012) and DeCarufel (2016) illustrates the effectiveness of implementing high performance materials such as UHPC and high strength steel reinforcement in columns subjected to extreme blast loading. Both authors studied the blast performance of partially fixed columns having cross-sectional dimensions of 152mm x 152mm using the University of Ottawa's modern shock tube testing facility. Burrell (2012), investigated the effects of: concrete type (normal strength vs. UHPC), fibre type, fibre content (0-6%) and seismic detailing. The findings from the research program are as follows:

- The addition of steel fibres to conventional concrete reduces the column's maximum and residual mid-span displacement when compared to non-fibre reinforced concrete columns;
- Columns designed with seismic detailing experienced smaller maximum displacement by approximately 27% when compared to control specimens. Hence the additional transverse reinforcement provided both enhanced ductility and confinement;
- The results show that the use of UHPC drastically reduces the mid-span displacements when compared to normal strength concrete columns, while also increasing overall blast capacity. This is a result of the increased compressive and tensile strength of the UHPC. However, the author found that the high-strength and toughness of the UHPC concrete eventually led to reinforcement steel rupture failure modes.

In a follow-up study, DeCarufel (2016) examined the effects of concrete type, longitudinal steel reinforcement ratio, seismic detailing, longitudinal steel reinforcement type (normal strength vs. high strength), fibre type and fibre content on UHPC column response. The experimental research program results yielded the following conclusions:

- Similar to Burrell (2012), results showed that UHPC significantly improves the blast resistance of reinforced concrete columns when compared to normal strength concrete specimens. The use of UHPC also prevents buckling of the longitudinal reinforcement while also enhancing the damage tolerance and reducing secondary blast fragmentation.
- Implementing high strength steel reinforcement was found to be an effective way of preventing pre-mature rupture of tension steel reinforcement, hence, resulting in an increased column blast resistance which fully utilizing the UHPC compressive strength.
- Providing high transverse steel reinforcement ratio improves the columns blast capacity, but presents more noticeable improvements in normal strength concrete columns when compared to UHPC specimens.

Blast tests of UHPC Slabs

Cavill et al. (2006) tested a total of seven UHPFRC slab panels, having a length of 2000mm, a width of 1000mm and thickness' varying from 50-100mm under live blast loads. Five of the seven panels were reinforced with high strength pre-stressed reinforcement, while the other two had no steel reinforcement. The panels were subject to both close and far range shockwaves. Results show that implementing *Ductal* (a proprietary UHPFRC) provides an effective solution for blast and impact resistance. The UHPFRC slab panels displayed high ductility and no signs of fragmentation, with the pre-stressed panels showing an even higher energy absorption with an ability to sustain deflections up to span/28 without fracture. Additional impact and ballistic tests were also conducted and presented in the report.

Yi et al. (2012) investigated the close-range blast performance of six concrete panels having a length of 1000mm, width of 1000mm and thickness of 150mm. The parameters investigated consisted of concrete strength and steel reinforcement ratio. The results obtained from deflection, strain and accelerometers measurements from the blast test reveal that UHPFRC specimens had a higher blast-resistance than normal strength concrete specimens. Additionally, the authors found that by providing short steel fibers and steel rebar, the brittle nature of such high-performance concrete can be negated.

Melançon (2015), conducted a parametric study on a total of fourteen one-way slab panels at the University of Ottawa. The slabs were subjected to far-range blasts using the University of Ottawa's modern shock tube. Among the parameters investigated in this research project are: effect of concrete strength, effect of tension steel reinforcement strength and effect of volumetric steel fibre content. Results show that the use of UHPFRC in flexural elements such as slabs, greatly reduces the maximum and residual deflections sustained from blast loads. Implementing high strength tension reinforcement increased the blast load resistance of the element in addition to reducing the maximum and residual mid-span displacements. The author notes that by increasing the

volumetric steel fibre reinforcement content, specimens experienced reduced spalling, cracking and fragmentation and debris generation upon failure.

Li et al (2015) studied the live blast performance of 4 two-way slab panels having a length of 2000mm, a width of 2000mm and a thickness of 100mm. The primary parameters investigated were; reinforcement ratio, tensile steel reinforcement strength and concrete strength. The authors found that steel reinforcement ratio play a significant role in blast resistance. Moreover, implementing a steel reinforcement having a higher yield stress can considerably reduce the damage of the slab panel and even allow the slab to overcome a blast that a lower grade steel panel would have failed at. Lastly, the results show that using UHPFRC improves the slabs blast resistance when compared to normal strength reinforced concrete.

2.4. Summary and Research Needs

The literature review provided an overview of previous research conducted on UHPC under static and dynamic loading.

A significant number of studies have examined the shear and flexural behaviour of UHPC beams under static loading. These studies have included beams having different cross-section types, and detailed with ordinarily-reinforced and prestressed reinforcement. Researchers have examined the effects of such parameters as concrete type, fiber content, fiber type, distribution of longitudinal reinforcement and use of composite UHPC layers. However, a survey of the literature indicates limited studies which have examined the effect of high-strength reinforcement on the flexural behaviour of UHPC beams.

The previous sections clearly identify the benefits of using of ultra-high performance concrete for enhanced protection against impact and blast loads. Research on the impact behaviour of UHPC confirms the high-strain rate sensitivity of UHPC in tension and compression and demonstrates the improved energy-absorption capacity of UHPC when compared to conventional plain and fiber-reinforced concretes. Several researchers have examined the impact response of UHPC structural elements, with a few studies focussing on beams.

The response of UHPC structural members under blast loads has also been studied in the literature. Previous blast research indicates that UHPC can significantly increase the energy absorption capacity of a structural member, reducing mid-span displacements as well as limiting the amount of fragmentation and debris generated upon ultimate failure. However, a survey of the literature indicates a lack of research data on the blast performance of beams. Furthermore, the potential benefits of implementing UHPC and high-strength reinforcement in flexural members subjected to extreme dynamic loading requires further study.

The proposed research program aims to fill these gaps in knowledge by enriching the experimental database on the blast performance of UHPC flexural members and investigating the benefits of combining UHPC and high strength steel reinforcement to enhance structural response.

Table 2-5 Summary of Blast Response of UHPFRC Elements.

Focus of study	Study	No. of Elements	Element Details		Element dimensions			f_c'' (MPa)	Tension steel			Transverse steel		Fibers		Parameters investigated					
			Ordinary or Pre-stressed?	Cross-section type	B (mm) or Dia (mm)	H (mm)	L (mm)		Yield strength (MPa)	Reinf. Ratio (%)	Prestressing Yield Strength (MPa)	Yield strength (MPa)	Reinf. Ratio (%)	Fiber type	Fiber content (%)	Fiber content	Axial Load	Tension reinf. ratio	High-strength steel	Far Range Blast	Close range blast
Columns	Burrell (2012)	13	O	R	152	152	2468	30-165	400	1.72	-	400	-	S	0-4.0	X	X			X	
	Roller et al. (2013)	10	O	C	150	-	-	40-130	500	2.2-8.0	-	400	0.06	S	-	X		X			X
	Li & Wu (2015)	4	O	R	200	200	2500	110-150	400	1.0	-	400	0.25	S	2.5		X			X	X
	Zhang & Wu (2015)	8	O	C	210	210	2500	170	360	-	-	-	-	S	2.0		X				X
	DeCarufel (2016)	17	O	R	152	152	2468	30-165	400-1150	1.2-4.9	-	400	-	S	0-3.0	X	X	X	X	X	
	Xu et al. (2016)	14	O	R	200	200	2500	148	400-1450	4.5	-	400	-	S	2.5		X		X		X
Slabs	Cavill et al. (2006)	7	P-S	R	1000	50-100	2000	160-200	-	-	1850	-	-	S	2.0				X	X	X
	Yi et al. (2012)	6	O	R	1000	150	1000	25.6-203	400	-	-	-	-	S	2.0			X			X
	Li et al. (2015)	4	O	R	2000	100	2000	56.6-130	300-1750	-	-	-	-	S	2.0-6.0	X		X	X		X
	Melancon (2015)	14	O	R	440	80-100	2440	50-150	430-1000	-	-	-	-	S	0.0-2.0	X			X	X	

C: Circular; ; R: (Rectangular Beam)

S: (Steel Fibre)

O: (Ordinary non pre-stressed reinforcement) ; P : (pre-stressed reinforcement)

Chapter 3. Experimental Program

3.1. General Overview

This experimental research program aims at examining the blast and quasi-static response of ultra-high performance concrete (UHPC) beams with conventional and high-strength steel reinforcement. Beam specimens were designed to highlight the effect of: concrete type, longitudinal reinforcement ratio, longitudinal reinforcement type, transverse reinforcement, as well as fiber type and content.

The details of the test program are summarized in this chapter, including: beam specimen design, material properties, specimen preparation, description of dynamic and quasi-static experimental setups, instrumentation and experimental procedure.

3.2. Description of Test Specimens

A total of twenty-one reinforced concrete beams were designed for this research program. Fourteen specimens were tested under simulated blast loading using the University of Ottawa's Shock Tube, and seven companion beams were tested under quasi-static four-point bending. The experimental test matrix includes: six normal strength concrete (NSC) control specimens detailed with conventional normal strength longitudinal reinforcement (NSR), ten beams built with compact-reinforced composite (CRC), a proprietary UHPC, and detailed with normal strength longitudinal reinforcement (NSR), and lastly five UHPC beams designed with high strength longitudinal reinforcement (HSR). Beam design details are summarized in Figure 3-1 and Table 3-1.

All twenty-one specimens had a constant length of 2440 mm, a height of 250 mm and a width of 125 mm. The clear cover to the bottom of the longitudinal reinforcement was maintained at 40 mm. All beams have identical unsupported spans of 2232 mm, with a constant moment region of 750 mm and shear spans of 741 mm, resulting in a shear span to depth ratio (a/d) of 3.7. The beams were detailed with different longitudinal reinforcement ratios and types. Two reinforcement bars were provided on the tension face and consisted of either: #4 (US Size), 15M, 20M (Canadian Size) normal strength bars, or #4, #5, #6 (US Size) high strength Grade 100 MMFX bars. Reinforcement comprised of #4, #5 and 15M rebar were detailed with 90° hooks, while the 20M rebar were provided with 180° to provide adequate development length to avoid a pull-out failure (the #6 high-strength bars were kept straight over the entire beam length due to difficulties in bending this bar type). Thirteen of the twenty-one specimens were designed with U-shaped stirrups made from 6 mm non-deformed wire acting as transverse reinforcement spaced at 100 mm in the shear spans. Although no continuous compression reinforcement was provided, 6 mm non-deformed steel bars were utilized in the shear span to help position transverse and longitudinal reinforcement. Additionally, two types of steel fibers (type A and B) were implemented in the

UHPC specimens at fiber contents ranging from 1% to 3% by volume of concrete (78-234 kg/m³). The fibers had different tensile strengths and aspect ratios. As shown in Table 3-1, the beam specimens can be categorized in three series based on the combination of concrete type and steel reinforcement type. Series 1 consist of normal strength concrete and normal strength longitudinal steel reinforcement (control), Series 2 is comprised of all specimens constructed with UHPC and normal strength longitudinal reinforcement (UHPC + NSR) and Series 3 includes UHPC specimens having high strength steel (UHPC + HSR). Beam specimens are referred to using the following nomenclature:

CONCRETE TYPE – FIBER % AND TYPE – STEEL REBAR SIZE – TYPE OF REBAR – STIRRUPS

For instance, CRC-2%A-15M-S is a UHPC beam built with CRC having 2% fibers of type “A”, and detailed with 15M normal strength longitudinal reinforcement. In comparison, the “HSR” in the nomenclature of CRC-2%A-#5-HSR-S indicates the use of #5 high-strength bars in this specimen. The “S” at the end of the specimen IDs indicates the use of stirrups spaced at 100 mm (the “0” in the ID of other specimens indicates the beams were built without transverse reinforcement).

Table 3-1 Specimen Series and Specifications

Series	Specimen ID	Concrete Type	Steel Fiber Properties		Steel Reinforcement Properties				Test Type *
			I. D	γ_f (%)	Steel Type	Flexural Steel	ρ (%)	Stirrup Spacing (mm)	
<i>Control</i>	NSC-0%-#4-0	NS	-	-	NSR	2 - #4	1.0	-	D
	NSC-0%-15M-0	NS	-	-	NSR	2 – 15M	1.5	-	D
	NSC-0%-15M-S	NS	-	-	NSR	2 – 15M	1.5	100	D/S
	NSC-0%-20M-S	NS	-	-	NSR	2 – 20M	2.4	100	D/S
<i>UHPC + NSR</i>	CRC-2%A-#4-0	UHPC	A	2.0%	NSR	2 - #4	1.0	-	D/S
	CRC-2%A-15M-S	UHPC	A	2.0%	NSR	2 – 15M	1.5	100	D/S
	CRC-2%A-20M-S	UHPC	A	2.0%	NSR	2 – 20M	2.4	100	D/S
	CRC-2%A-20M-0	UHPC	A	2.0%	NSR	2 – 20M	2.4	-	D
	CRC-1%B-15M-0	UHPC	B	1.0%	NSR	2 – 15M	1.5	-	D
	CRC-2%B-15M-0	UHPC	B	2.0%	NSR	2 – 15M	1.5	-	D
	CRC-3%B-15M-0	UHPC	B	3.0%	NSR	2 – 15M	1.5	-	D
<i>UHPC + HSR</i>	CRC-2%A-#4-HSR-S	UHPC	A	2.0%	HSR	2 - #4	1.0	100	D/S
	CRC-2%A-#5-HSR-S	UHPC	A	2.0%	HSR	2 - #5	1.5	100	D
	CRC-2%A-#6-HSR-S	UHPC	A	2.0%	HSR	2 - #6	2.3	100	D/S

*D = Dynamic Test

*S = Static Test

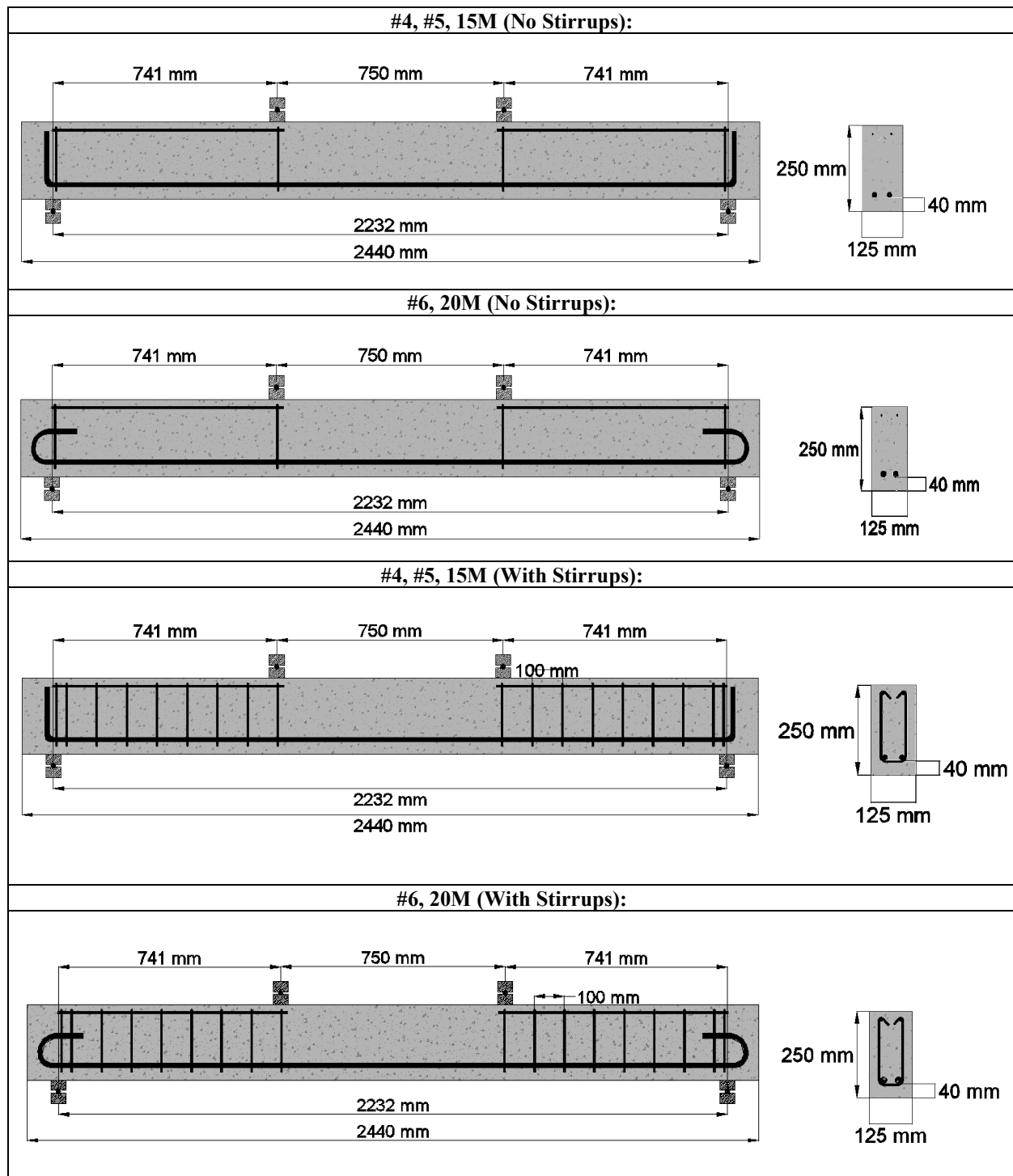


Figure 3-1 Specimen dimensions and reinforcement details

3.2.1. Control Series

A total of six control specimens were cast in this experimental research program. The control series are all constructed using normal strength concrete and normal strength longitudinal reinforcement. Four control specimens were tested dynamically using the shock tube: NSC-0%-#4-0, NSC-0%-15M-0, NSC-0%-15M-S and NSC-0%-20M-S. The series includes 3 different reinforcement ratios (with #4, 15M or 20M bars) and includes specimens with and without shear reinforcement. Among these, two companion beams were also tested statically; NSC-0%-15M-S, NSC-0%-20M-S. The control series acts as a benchmark to record any improvement and/or change in response when migrating towards high performance materials.

3.2.2. NSR UHPC Series

The second series is referred to as the NSR UHPC set, designed with normal strength longitudinal reinforcement in combination with a proprietary ultra high performance concrete (CRC: compact reinforced composite). The series is comprised of 10 specimens for which rebar size ranges between #4, 15M and 20M. Six of the ten specimens have no transverse reinforcement versus four beams that are detailed with stirrups spaced at 100 mm. Furthermore, the effect of two types of structural steel fibers are examined in this series (Types A and B; see next section). The following three specimens; CRC-1%B-15M-0, CRC-2%B-15M-0, CRC-3%B-15M-0, examine the effect of volumetric fiber content as the content ranges between 1, 2 and 3% (Type B). All other beams included in the series are detailed having a constant volumetric ratio of 2% (Type A). In addition to the seven specimens tested dynamically using the shock tube the following static companion beams are also tested: CRC-2%A-#4-0, CRC-2%A-15M-S, CRC-2%A-20M-S.

3.2.3. MMFX UHPC Series

The third series is comprised of five specimens having both high strength longitudinal reinforcement (MMFX) and UHPC. Specimens in this series examine the effect of high-strength longitudinal reinforcement ratio with the beams detailed with #4, #5 or #6 bars. All specimens have transverse reinforcement spaced at 100 mm in the shear span and a constant volumetric fiber content of 2% fiber type A. The following three specimens are tested using the shock tube: CRC-2%A-#4-HSR-S, CRC-2%A-#5-HSR-S, CRC-2%A-#6-HSR-S, all having static companion beams to the exception of CRC-2%A-#5-HSR-S.

3.3. Materials

3.3.1. Concrete Parameters

Two types of concrete were utilized in this experimental program. The first being a conventional normal strength concrete mix designed to achieve a compressive strength of 40MPa; mixture components and quantities are presented in Table 3-2. The normal strength concrete mix was used for all six of the control series specimens.

The second type of concrete is a proprietary ultra-high performance concrete called CRC (*Compact Reinforcement Composite*) originally developed in 1986 by Aalborg Portland, and currently marketed by CRC Technology in Denmark (Hi-Con, 2016). CRC is designed to reach compressive strengths in the range of 140-200MPa and a tensile strength of 30-40 MPa at a strain of 0.4-0.5% (Hi-Con, 2016). CRC was implemented for the construction of fifteen specimens included in series two and three. The *CRC JointCast* premix used in this study is dry mortar blend comprised of *Densit Binder* (Portland cement and micro silica), quartz sand and aggregates no larger than 4 mm in diameter. The pre-packaged product comes with a superplasticizer already included in the dry mix. Depending on the fiber type used, volumetric fiber content ranging from 2-6% can be used with this mix. CRC specifications as provided by the manufacturer are presented in Table 3-3.

Table 3-2 Normal Strength Concrete Properties

Component	Content (Kg/m³)
Cement	312.5
Water	187.5
Sand	625
½" Aggregate	1250

Table 3-3 CRC Properties

Component	Fraction by Weight	Density (Kg/m³)	Content (kg/m³ of Concrete)
Densit Binder	39	Cement: 3150 Micro-Silica: 2250	1146
Quartz Sand			
0 - 0.25mm	8	2640	236
0.25 - 1 mm	14	2640	404
1 – 4 mm	27	2740	809
Water	7	1000	202
Water-Binder Ratio	-	-	0.18
Steel Fiber (V_f =2%)	5	7500	156



Figure 3-2 Normal Strength Concrete Mix



Figure 3-3 CRC JointCast Proprietary Mix

3.3.2. Steel Reinforcement Parameters

A total of three different steel reinforcement types were included in this experimental research program. The first type consists of deformed normal strength steel reinforcement (sizes: #4 US, 15M and 20M Canadian) used as longitudinal reinforcement in sixteen specimens. The second is deformed high strength reinforcement steel referred to as MMFX (Sizes: #4, #5, #6 US) also used as longitudinal reinforcement in a total of five specimens. Lastly, non-deformed 6.3 mm steel wire was implemented as transverse reinforcement in nine beams in addition to compression reinforcement in the shear span of all specimens to facilitate construction of the cages.

Tensile tests were completed for a total of three coupons for each steel reinforcement bar size and type using a *Galdabini Sun 60 Universal Testing Machine* as depicted in Figure 3-4. The tensile stress-strain relationship was obtained by imposing a tensile strain rate of 7.5 mm/min all while recording the resulting displacement using an axial extensometer (Model 3542 Epsilon Technology Corporation) with an initial gauge length of 50mm. The average material properties; bar diameter (D_b), bar area (A_b), yield strain (ϵ_y) and yield stress (f_y), ultimate strain (ϵ_u) and ultimate stress (f_u), alongside the rupture strain (ϵ_r) are summarized in Table 3-4. Stress-strain curves are presented in Figure 3-5 to Figure 3-8, while a comparison of curves from each bar type is presented Figure 3-9. Because the MMFX bars do not show a well-defined yield point, the 0.2% offset method was used to determine the properties at yield for this bar type.

Table 3-4 Steel Reinforcement Mechanical Properties

ID	Reinforcement Type	Bar Diameter D_b (mm)	Bar Area, A_b (mm ²)	Yield		Ultimate		Rupture
				ϵ_y	f_y (MPa)	ϵ_u	f_u (MPa)	ϵ_r
6.3 NS	Non-Deformed	6.35	32	0.0024	540	0.051	610	0.051
#4	Regular	12.7	129	0.0027	604	0.11	908	0.12
15M	Regular	16.0	200	0.0023	440	0.14	586	0.15
20M	Regular	19.5	300	0.0024	462	0.13	605	0.17
#4HS	High Strength	12.7	129	0.0067	904	0.056	1077	0.18
#5HS	High Strength	15.875	200	0.0070	929	0.0485	1217	0.048
#6HS	High Strength	19.05	284	0.0066	855	0.0528	1153	0.058



Figure 3-4 Galdabini Steel Coupon Testing Setup

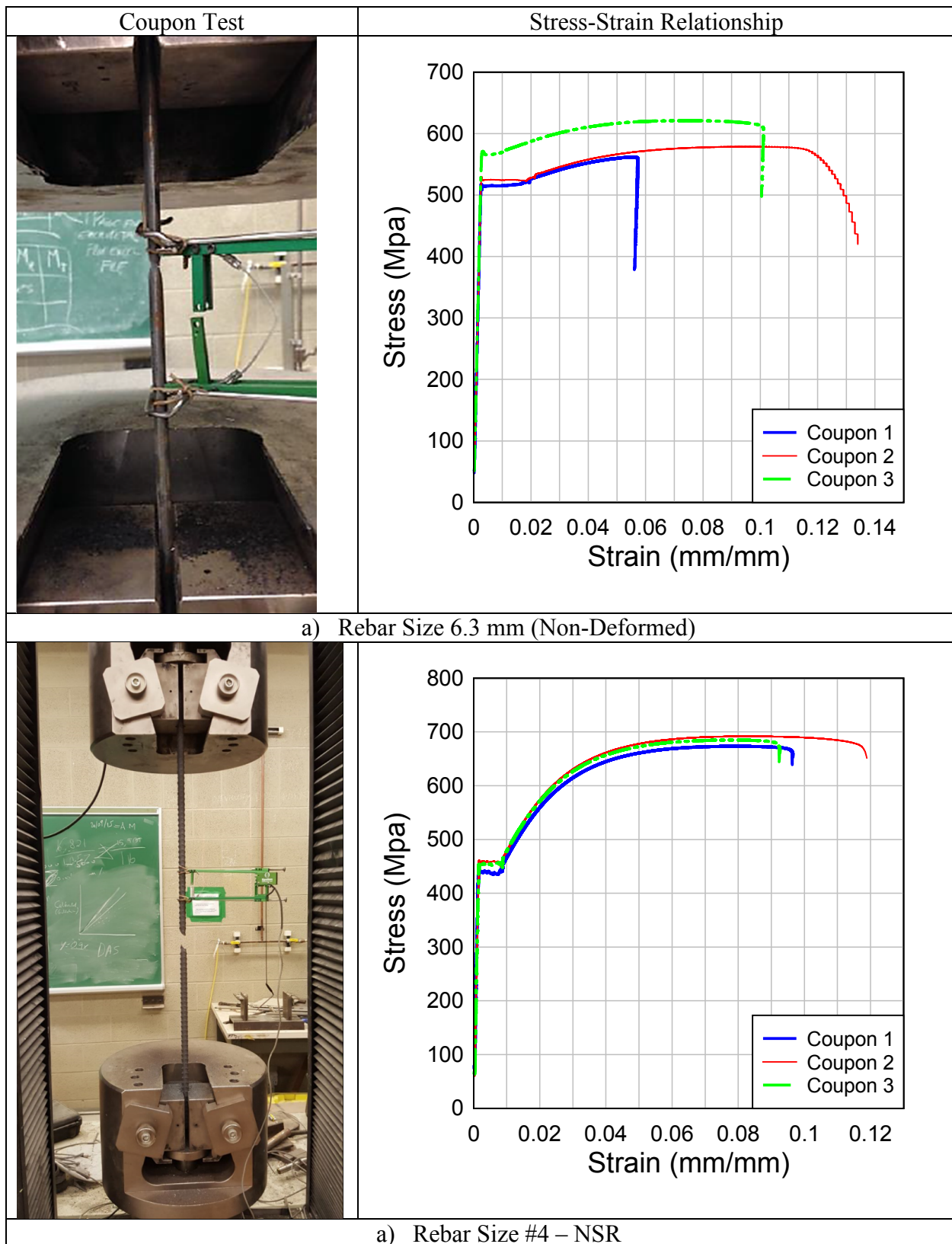


Figure 3-5 Steel Reinforcement Stress-Strain Relationship and Photographs (6.3mm and #4 NSR)

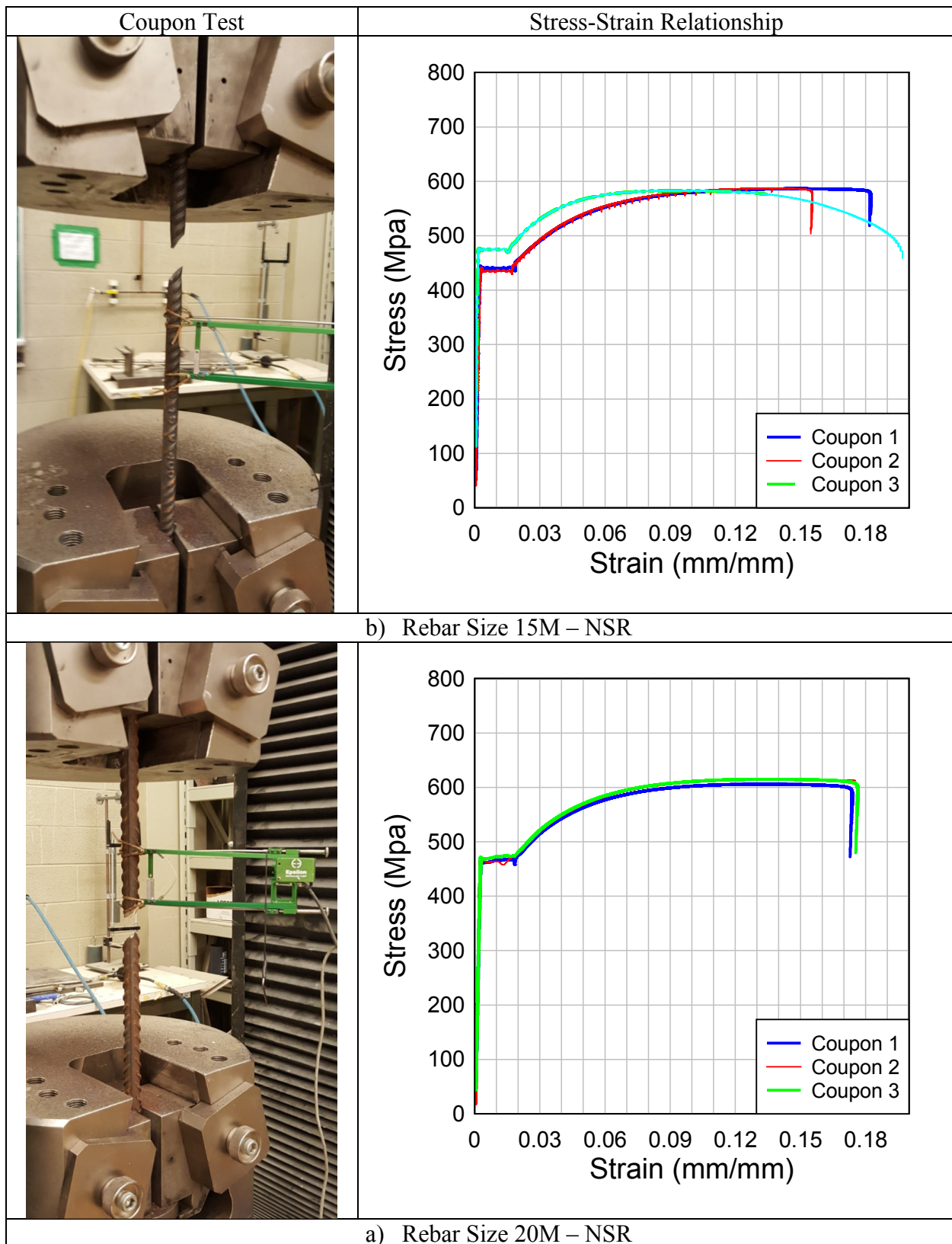
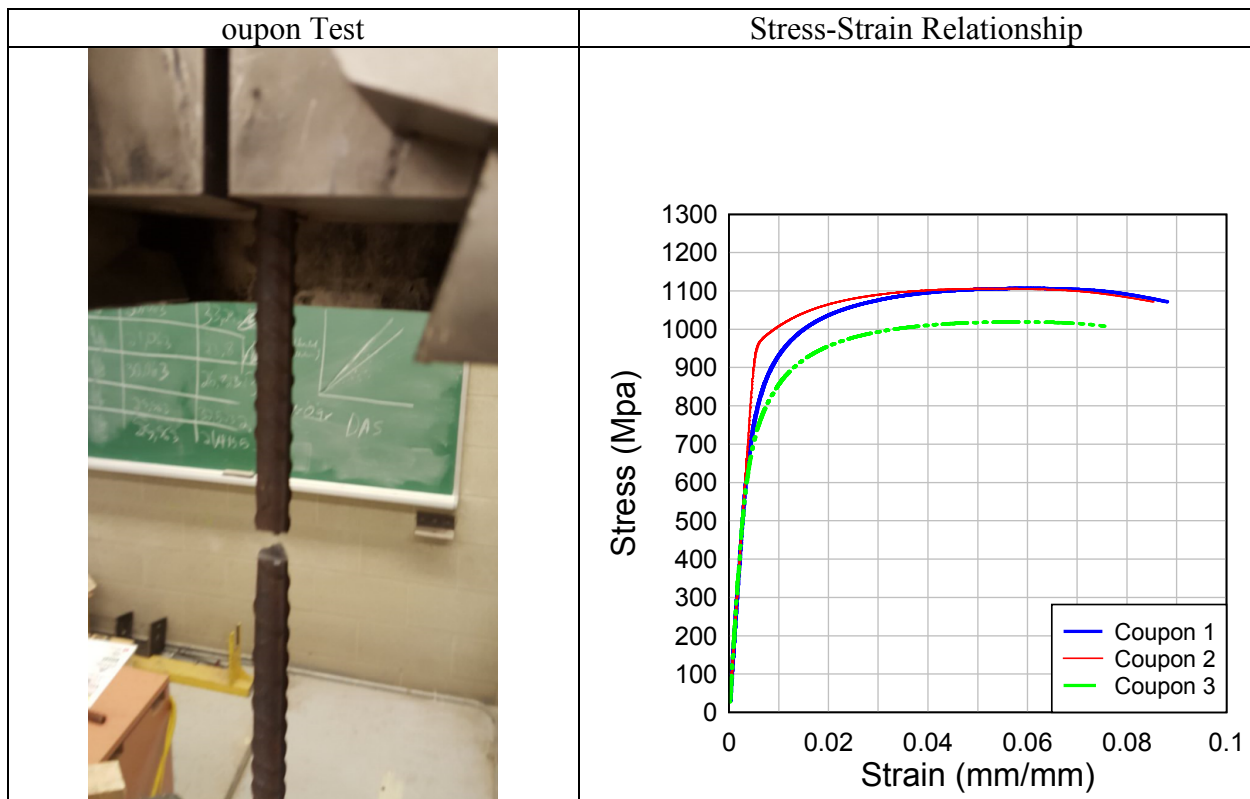
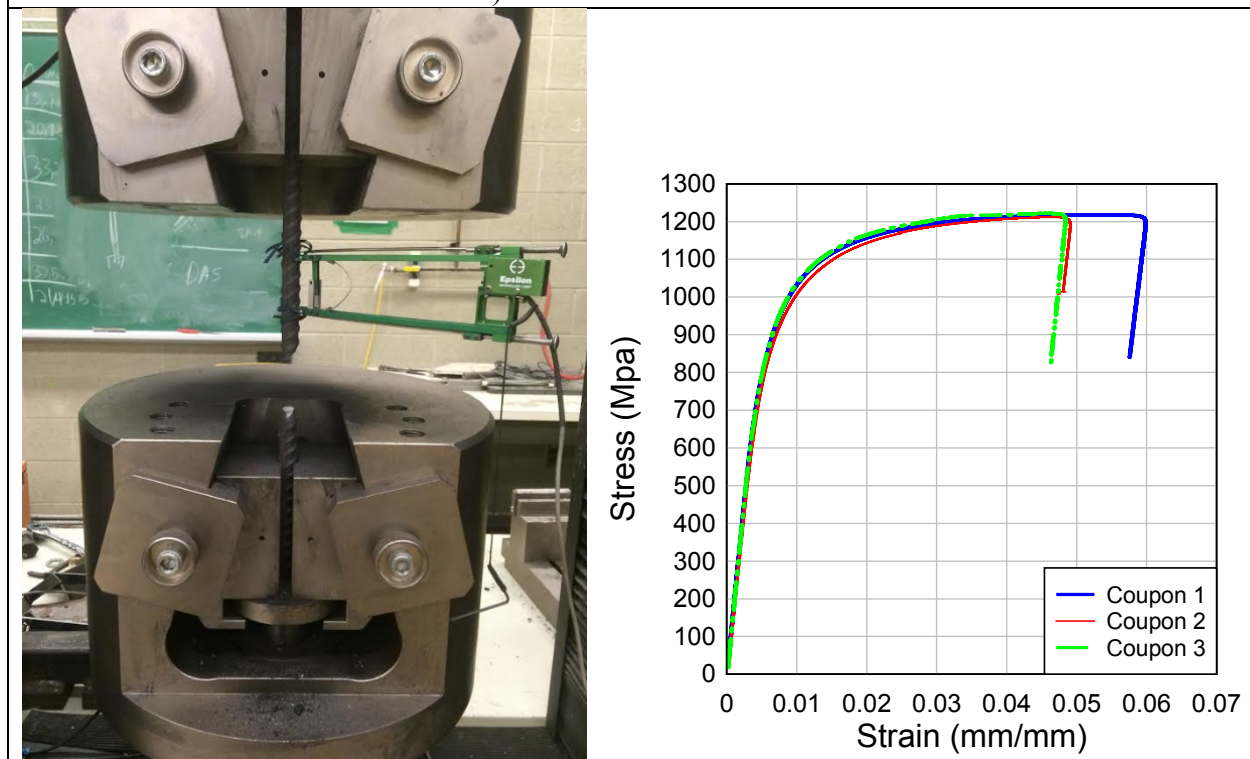


Figure 3-6 Steel Reinforcement Stress-Strain Relationship and Photographs (15M and 20M)



b) Rebar Size #4 – MMFX



b) Rebar Size #5 – MMFX

Figure 3-7 Steel Reinforcement Stress-Strain Relationship and Photographs (MMFX: #4 and #5)

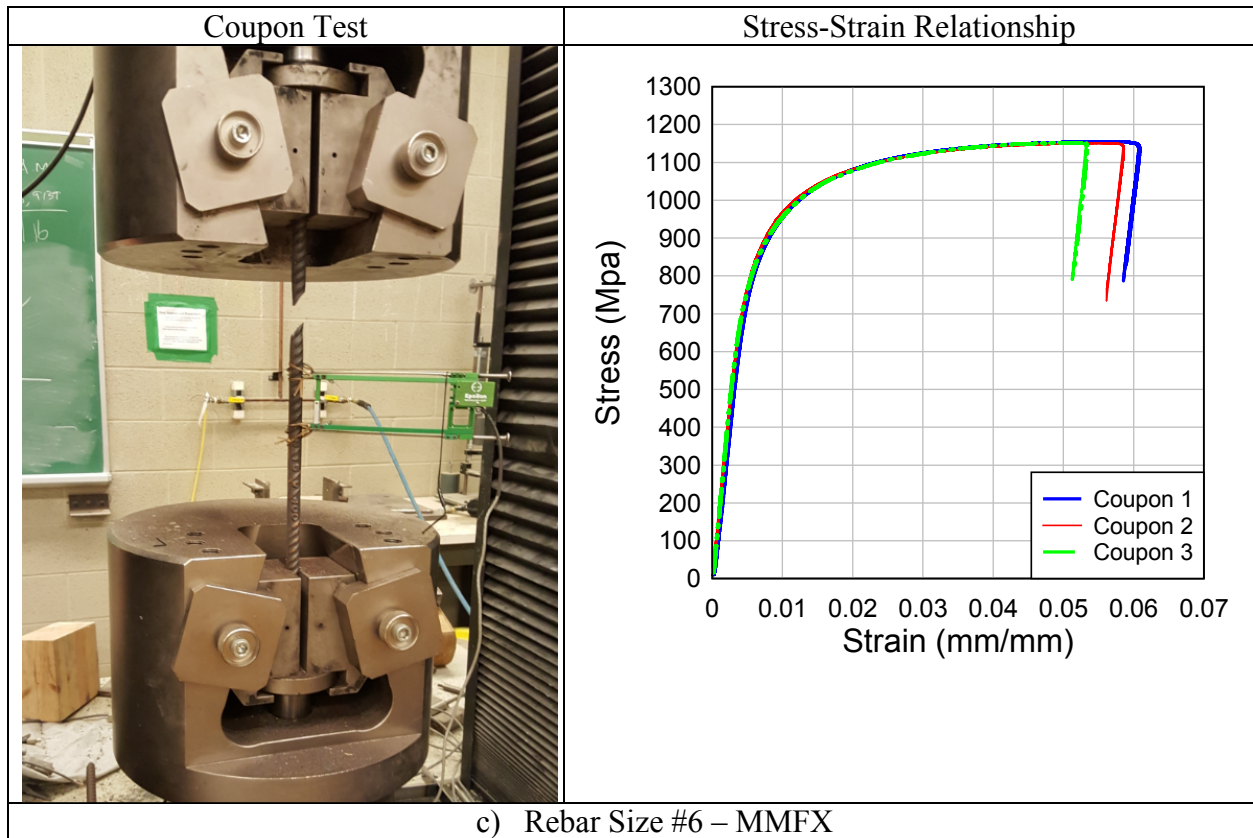


Figure 3-8 Steel Reinforcement Stress-Strain Relationship and Photographs (MMFX: #6)

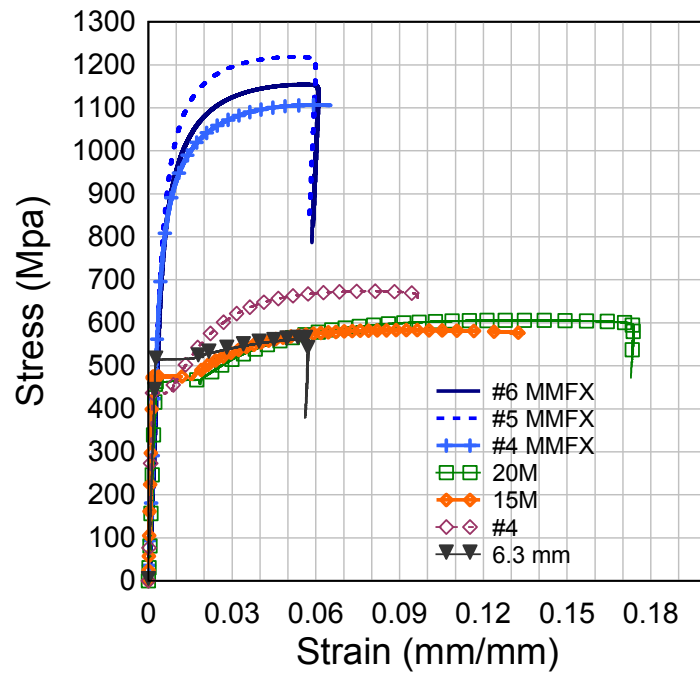


Figure 3-9 Comparison of the Stress-Strain Relationships for all Bar Types

3.3.3. Steel Fiber parameters

Two types of smooth straight steel fibers having a length (l_f) of 13 mm were implemented in the experimental program; a summary of fiber properties can be found in Table 3-5 alongside fiber photographs in Figure 3-10. Fiber type A, is manufactured by *BEKAERT* under the *Dramix OL 13/.20* brand, and has an aspect ratio (l_f/d_f) of 62, with a tensile strength of 2750 MPa; fiber type A was included in 9 UHPC beam specimens. Fiber type B, manufactured under the *BELM-0.3/12-3150* brand, has an aspect ratio of 43 and a tensile strength of 3150 MPa; fiber type B was implemented in 3 UHPC specimens to examine the effect of fiber type and content.

Table 3-5 Steel Fiber Properties

Fiber ID	Fiber Name	Length l_f (mm)	Diameter D_f (mm)	Aspect Ratio (mm/mm)	Tensile Strength (MPa)
A	OL 13/.20	13	0.21	62	2750
B	BELM-0.3/13-3150	13	0.30	43	3150



Fiber Type A: OL 13/.20



Fiber Type B: BELM-0.3/13-3150

Figure 3-10 Steel Fiber Photographs

3.4. Construction of Test Specimens

The twenty-one beam specimens included in this research program were constructed, instrumented and cast in the structures laboratory at the University of Ottawa. The procedures undertaken to construct the beam specimens are summarized in the following sub-sections.

3.4.1. Preparation and Casting

The first step in the construction phase was constructing the beam formwork using 4'x8'x3/4" plywood sheets as well as 2"x4" studs in order to provide lateral stiffness. The second step was to bend and prepare the steel reinforcement cages. The #4, #5 and 15M specimens were provided with 90-degree hooks having an embedment length of twelve times the bar diameter (150mm (#4) & 180mm (#5 and 15M)). The #6 and 20M specimens were provided with 180-degree hooks having a 100mm embedment length beyond the bending radius. Stirrups were manufactured by

bending the 6.3 mm steel wire using a manually operated jig. Photographs of the cages after construction are presented in Figure 3-11

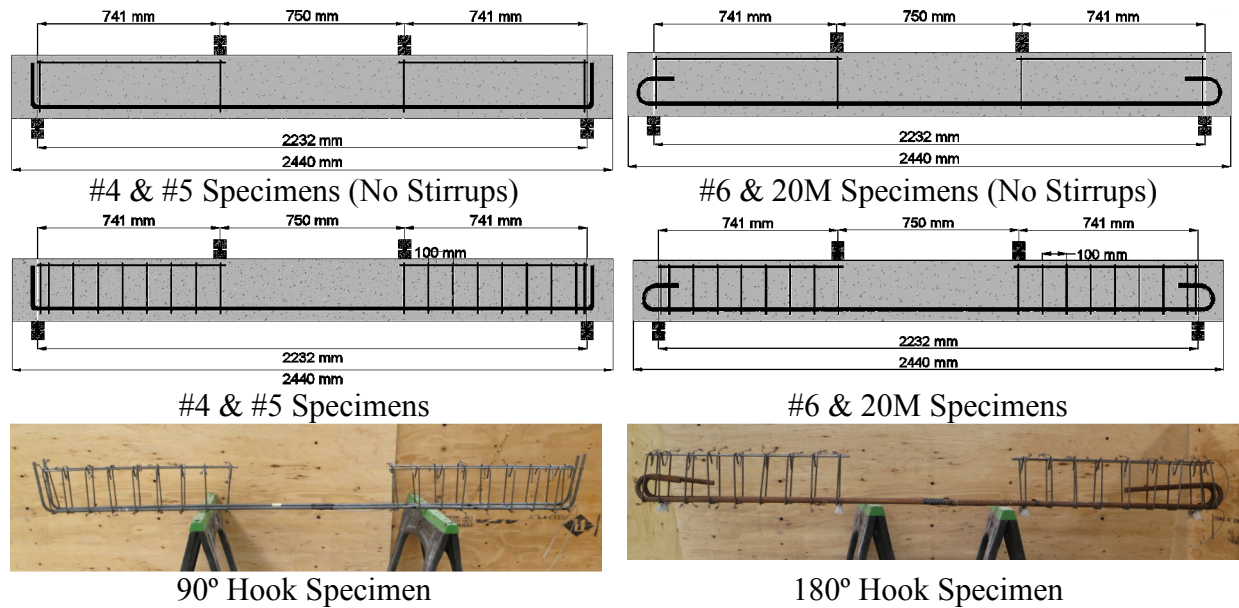


Figure 3-11 Steel Reinforcement Details

Once the steel cages were complete, reinforcement lugs were removed over small area approximately 15mm by 8mm using a belt-grinder and smoothed with sanding paper in order to attach one strain gauge (350-ohm gauge for dynamic specimens, 120-ohm gauge for static specimens) on each reinforcing bar on the tension side, as illustrated in Figure 3-12. Prior to inserting the steel cages into the formwork, a thin layer of oil was applied on the inner surfaces of the forms in order to allow for their reuse. Next, the steel cages were inserted in the formwork onto 40mm plastic chairs; hence specimens detailed with 6.3mm transverse reinforcement had clear cover of 35mm, whereas specimens without any stirrups had a clear cover of 40 mm.

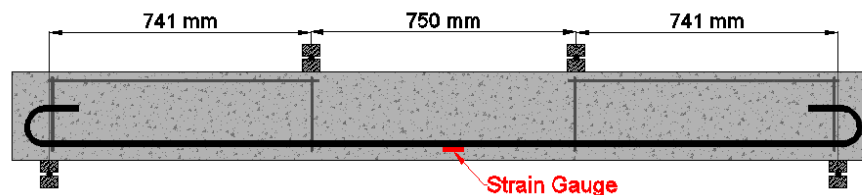


Figure 3-12 Strain Gauge Location

Both the normal strength concrete and the proprietary UHPC (CRC) concrete was mixed and cast in the structures laboratory at the University of Ottawa using a pan mixer with a 225 L (8 ft³) capacity. Once the concrete specimens were cast, moist burlap was installed around the beams and form, further covered by plastic tarps, this curing was maintained for a total of 7 days at which point the burlap was removed and the specimens were air-cured for the remainder of the 28 days.



a) Pan Mixer



b) Formwork and Steel Cages



c) Casting Beam Specimen



d) Moist-Curing (7 days)

Figure 3-13 Casting Phase Photographs

3.4.2. Fresh State Properties

Two types of fresh state material testing were conducted during the casting phase of the 21 specimens. For the conventional normal strength concrete mix the ASTM C143 “Standard Test Method for Slump of Hydraulic-Cement Concrete” was conducted and summarized in Table 3-6. For the fresh state properties of the proprietary UHPC mix, the ASTM C1611 “Standard Test Method for Slump Flow of Self-Consolidating Concrete” and ASMT C1437-15 “Standard Test Method for Flow of Hydraulic Cement Mortar” were conducted and are summarized in Table 3-6.

Table 3-6 Fresh State Material Properties

Series	Specimen ID	Concrete Type	Steel Fiber Properties		ASTM C143 Slump (mm)	ASTM C1611 Slump Flow (mm)		ASTM C1437-15 Mortar Flow (mm)
			LD	vf (%)		slump	Avg. Slump	
<i>Control</i>	NSC-0%-#4-0	NS	N/A	0.0%	125	-	-	-
	NSC-0%-15M-0	NS	N/A	0.0%	125	-	-	-
	NSC-0%-15M-S	NS	N/A	0.0%	125	-	-	-
	NSC-0%-15M-S	NS	N/A	0.0%	125	-	-	-
	NSC-0%-20M-S	NS	N/A	0.0%	125	-	-	-
	NSC-0%-20M-S	NS	N/A	0.0%	125	-	-	-
<i>UHPC + NSR</i>	CRC-2%A-#4-0	UHPC	A	2.0%	-	460/475	470	150/150
	CRC-2%A-#4-0	UHPC	A	2.0%	-	460/475	470	150/150
	CRC-2%A-20M-0	UHPC	A	2.0%	-	485/470	478	190/190
	CRC-2%A-15M-S	UHPC	A	2.0%	-	490/470	480	175/175
	CRC-2%A-15M-S	UHPC	A	2.0%	-	490/470	480	175/175
	CRC-2%A-20M-S	UHPC	A	2.0%	-	475/465	470	170/170
	CRC-2%A-20M-S	UHPC	A	2.0%	-	475/465	470	170/170
	CRC-1%B-15M-0	UHPC	B	1.0%	-	520/530	525	195/190
	CRC-2%B-15M-0	UHPC	B	2.0%	-	490/460	475	190/190
	CRC-3%B-15M-0	UHPC	B	3.0%	-	470/460	465	190/190
<i>UHPC + HSR</i>	CRC-2%A-#4-HSR-S	UHPC	A	2.0%	-	485/470	478	190/190
	CRC-2%A-#4-HSR-S	UHPC	A	2.0%	-	485/470	478	190/190
	CRC-2%A-#5-HSR-S	UHPC	A	2.0%	-	475/470	472.5	160/160
	CRC-2%A-#6-HSR-S	UHPC	A	2.0%	-	475/470	472.5	160/160
	CRC-2%A-#6-HSR-S	UHPC	A	2.0%	-	460/460	460	155/155

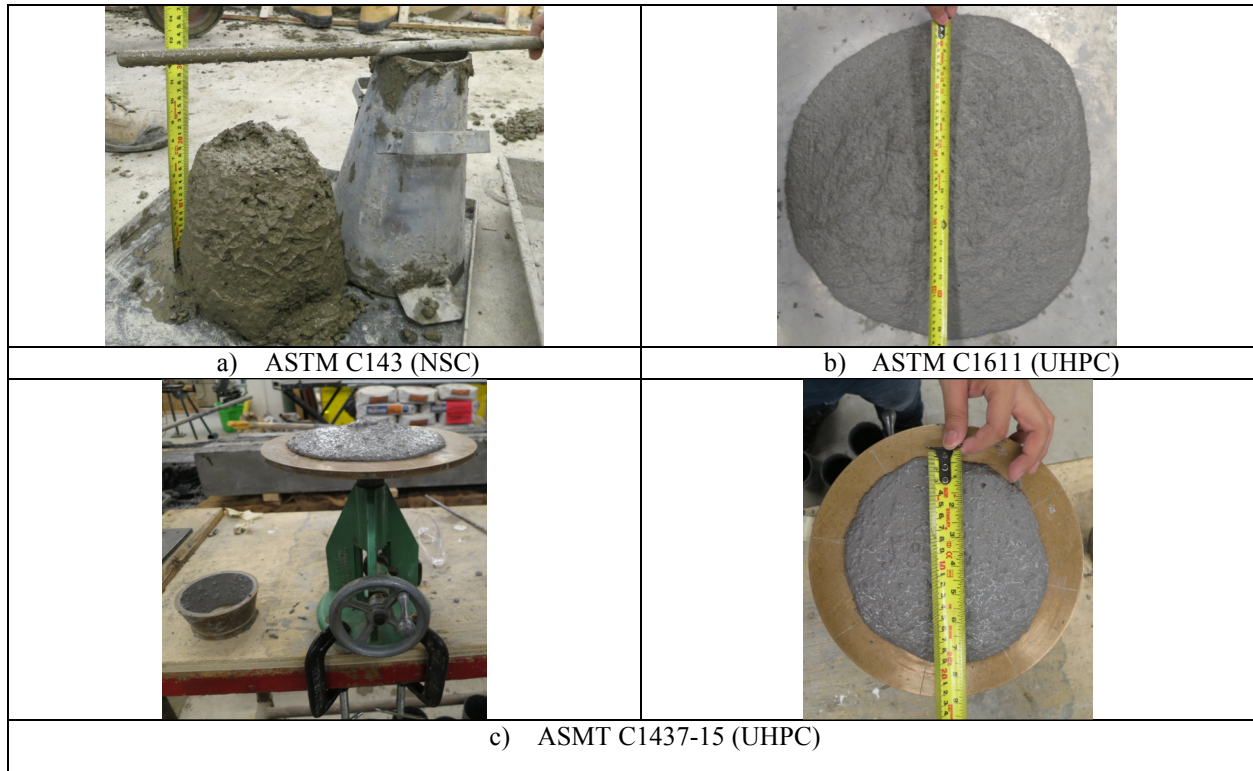


Figure 3-14 Fresh Material Tests Photographs

3.4.3. Hardened State Properties

The Compressive strength of a minimum of three concrete cylinders (100 mm x 200 mm) for each beam specimen was tested in accordance to ASTM C39 “Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens”; results are summarized in Table 3-7. Compressive stress-strain relationships were also obtained using a PILOT CONTROLS machine with 1000 KN capacity and the test setup illustrated in Figure 3-15 which is comprised of two bearing plates at the top and bottom of the concrete cylinder alongside two LVDTs, one on each lateral extremity. Typical stress-strain relationships are presented in Figure 3-16. Results show that UHPC resists greater peak loads, while also proving to have greater ductility when compared to the normal strength concrete specimens. Comparing stress-strain relationships presented in Figure 3-16 c) through Figure 3-16 e), demonstrate that a higher fiber content increases the overall peak compressive strength as well as the maximum strain at failure. Moreover, concrete cylinders containing fibers generated far less debris at failure; this can be seen when comparing Figure 3-17 a) and Figure 3-17 b). Photographs of concrete cylinder failures are depicted in Figure 3-17.



a) PILOT Compression Testing Machine



b) Deformation Measurement Setup

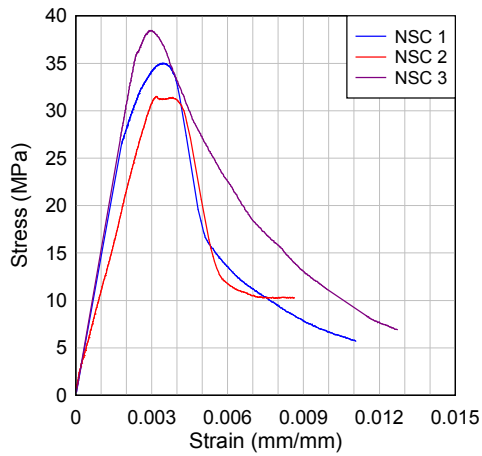
Figure 3-15 Concrete Cylinder Compression Test Setup

Table 3-7 Concrete Compressive Strength Summary

Specimen ID	Concrete Mix	Fiber Type	Fiber Ratio	Cylinder Test Compressive Strength (MPa)				Type of Test
				#1	#2	#3	AVG	
NSC-0%-#4-0	NSC	-	-	32.94	38.07	31.23	34.08	D
NSC-0%-15M-0	NSC	-	-	32.94	38.07	31.23	34.08	D
NSC-0%-15M-S	NSC	-	-	40.75	38.41	31.48	36.88	D
NSC-0%-20M-S	NSC	-	-	40.75	38.41	31.48	36.88	D
NSC-0%-15M-S	NSC	-	-	32.94	38.07	31.23	34.08	S
NSC-0%-20M-S	NSC	-	-	40.75	38.41	31.48	36.88	S
CRC-2%A-#4-0	UHPC	A	2%	145.84	152.2	156.47	151.50	D
CRC-2%A-#4-0	UHPC	A	2%	123.02	138.69	142.12	134.61	S
CRC-2%A-15M-S	UHPC	A	2%	140.19	130.44	146.61	139.08	D
CRC-1%B-15M-0	UHPC	B	1%	108.52	116.94	119.21	114.89	D
CRC-2%B-15M-0	UHPC	B	2%	134.83	122.644	121.74	126.40	D
CRC-3%B-15M-0	UHPC	B	3%	144.74	156.54	147.52	149.60	D
CRC-2%A-15M-0	UHPC	A	2%	117.87	144.89	147.39	136.72	S
CRC-2%A-20M-S	UHPC	A	2%	155.76	159.38	146.81	153.98	S
CRC-2%A-20M-0	UHPC	A	2%	143.36	161.11	142.28	139.08	D
CRC-2%A-20M-S	UHPC	A	2%	159.4	141.01	147.95	149.45	D
CRC-2%A-#4-HSR-S	UHPC	A	2%	132.54	139.21	134.17	135.31	S
CRC-2%A-#4-HSR-S	UHPC	A	2%	162.78	162.45	153.67	159.63	D
CRC-2%A-#5-HSR-S	UHPC	A	2%	163.39	141	141.13	148.51	D
CRC-2%A-#6-HSR-S	UHPC	A	2%	145	141.2	137.78	141.33	D
CRC-2%A-#6-HSR-S	UHPC	A	2%	160.2	148.91	151.24	153.45	S

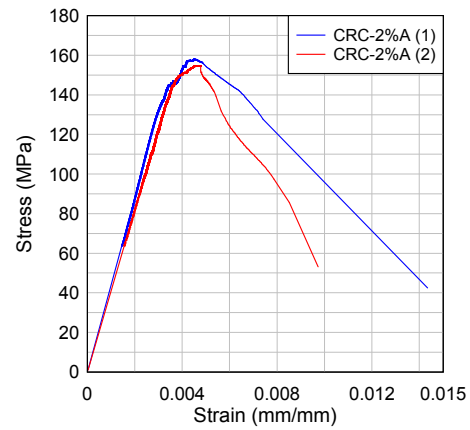
D: dynamic beam specimen; S: static beam specimen

Concrete Stress-Strain Relationship (NSC)



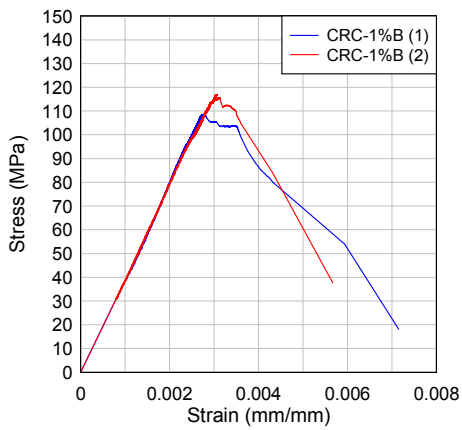
a) Normal Strength Concrete, NSC-0%

Concrete Stress-Strain Relationship (CRC-2%A)



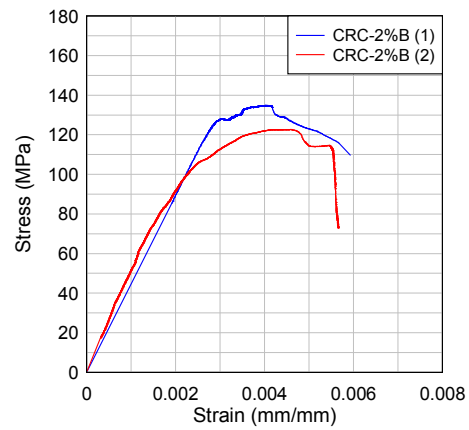
b) CRC-2%A

Concrete Stress-Strain Relationship (CRC-1%B)



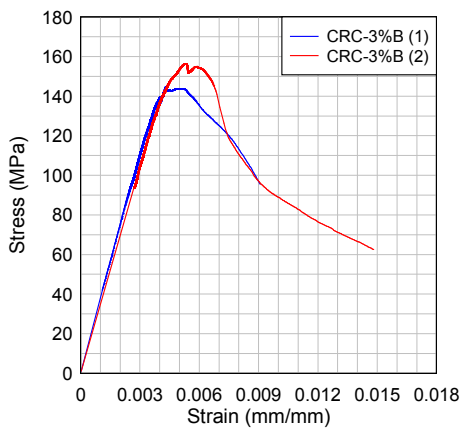
c) CRC-1%B

Concrete Stress-Strain Relationship (CRC-2%B)



d) CRC-2%B

Concrete Stress-Strain Relationship (CRC-3%A)



e) CRC-3%B

Figure 3-16 Concrete Stress-Strain Relationships

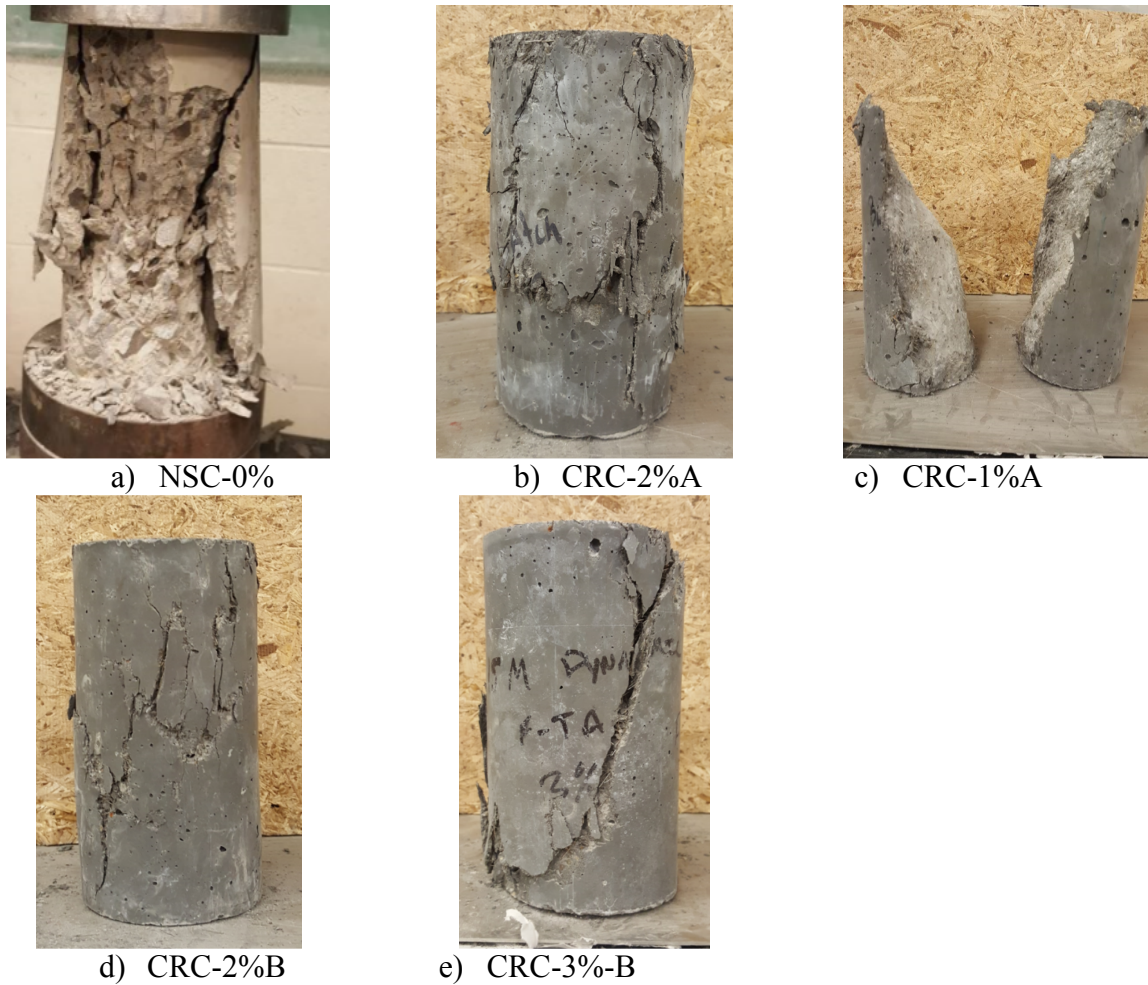
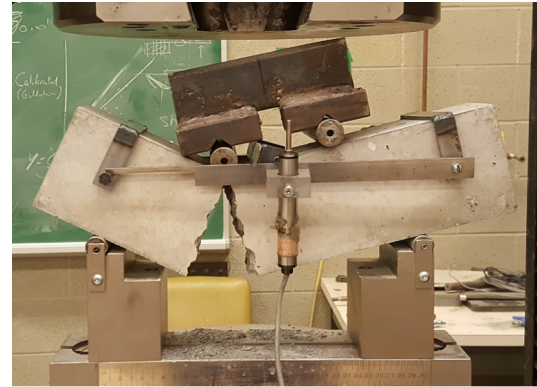


Figure 3-17 Sample Concrete Cylinder Testing Failures

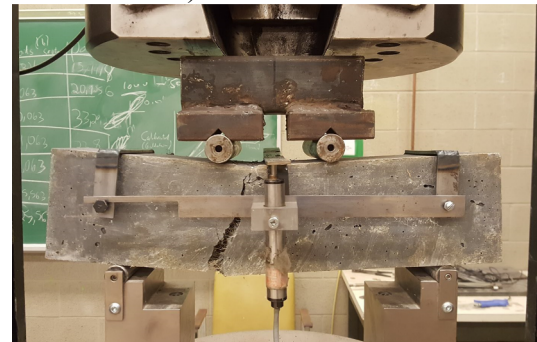
The flexural toughness of the concrete mixes utilized throughout the experimental research program were tested in accordance to ASTM C1609 “Standard Testing Method for Flexural Performance of Fiber-Reinforced Concrete (Using Beam with Third-Point Loading)”. Testing was conducted on prisms having dimensions of 100 mm x 100 mm x 400 mm, with a 300 mm test span. A photograph of the test setup is displayed in Figure 3-18, alongside a summary of the flexural toughness test results presented in Table 3-8 and Figure 3-19. All UHPC mixes experienced deflection stiffening following an initial peak load due to the provision of steel fibers. The noteworthy peak flexural strength of the CRC-2%A and CRC-3%B are 26.51MPa and 23.78MPa, respectively. The flexural behaviour of the fiber reinforced UHPC mixes are characterized by the formation of one principle crack which experiences deformation concentrations until fiber pullout occurs and ultimately failure. Conversely, the normal strength concrete experience minimal flexural strength (6.22MPa), followed by a very brittle failure soon after first cracking.



a) Flexural Prism Test Setup



i) NSC Prism



ii) UHPC Prism

b) Prism Failure Photographs

Figure 3-18 ASTM C1609 Flexural Prism Toughness Test Photographs

Table 3-8 ASTM C1609 Flexural Toughness Test

Concrete Mix	ASTM C1609										
	P_1	δ_1	P_p	δ_p	f_1	f_p	P_{600}	f_{600}	P_{150}	f_{150}	T_{150}
NSC-0%	-	-	20.75	0.058	-	6.22	-	-	-	-	-
CRC-2%A	54.03	0.14	88.37	1.43	16.20	26.51	71.39	21.41	70.03	21.00	150.98
CRC-1%B	23.86	0.026	50.13	0.89	7.15	15.03	41.51	12.45	40.74	12.22	84.88
CRC-2%B	55.87	0.169	62.18	0.49	16.76	18.65	62.08	18.62	17.07	5.11	84.69
CRC-3%B	65.96	0.185	79.29	0.6	19.78	23.78	73.91	22.17	42.48	12.74	121.5

L = Span Length (300 mm), ($L/600 = 0.5$, $L/150 = 2$) P_1 = First-Peak Load (kN) δ_1 = Net Deflection at First-Peak Load (mm) P_p = Peak Load (kN) δ_p = Net Deflection at Peak Load (mm) f_1 = First-Peak Strength (MPa) f_p = Peak Strength (MPa) P_{600} = Residual Load at net deflection of $L/600$ (kN) f_{600} = Residual Strength at net deflection of $L/600$ (MPa) P_{150} = Residual Load at net deflection of $L/150$ (kN) f_{150} = Residual Strength at net deflection of $L/150$ (MPa) T_{150} = Area under load vs. net deflection curve (0 to $L/150$), (kN*mm) FT = Flexural Toughness Factor = $(T_{150} * L) / (L/150 * b * d^2)$	
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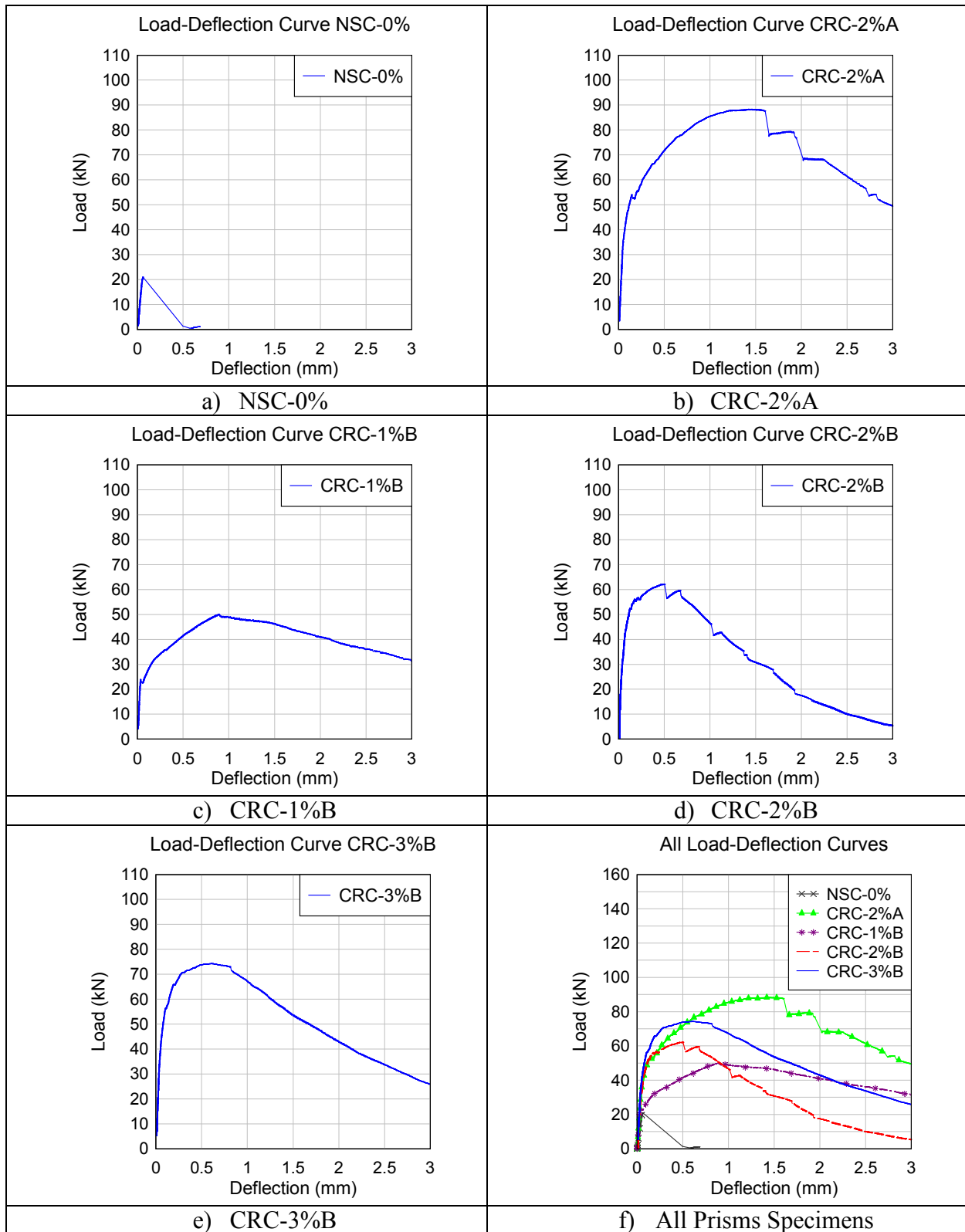


Figure 3-19 Flexural Beam Load-Deflection Curve Samples

3.5. Experimental Setup

A detailed overview of the dynamic and static test setup will be presented in this section. Starting with the blast setup, information is provided on the shock tube, the load transfer device (LTD), support conditions, as well as the instrumentation utilized to measure, displacement, longitudinal reinforcement strain, support reactions in addition to the induced shockwave properties. Moreover, details on the static loading setup, instrumentation and procedure and will be presented.

3.5.1. Dynamic Blast Testing

3.5.1.1. Shock-Tube

The dynamic blast testing was conducted at the University of Ottawa's Blast Laboratory using a shock-tube capable of simulating far-range explosion shockwaves. The shock-tube is operated through the manipulation of pressure differentials in various sections which are illustrated in Figure 3-20 through Figure 3-22. As depicted in Figure 3-20, the variable length driver section is used to accumulate compressed air until the desired pressure is achieved; it is the accumulated pressure in the driver section which determines the magnitude of reflected pressure imposed on the specimens. Meanwhile two sets of aluminum diaphragms separate the driver section from the spool section, and the spool section from the expansion chamber as seen in Figure 3-21, where pressure is also accumulated to maintain a prescribed pressure differential until the shockwave is ready to be detonated. Once the laboratory is cleared of all personnel, and the desired driver pressure is reached, the compressed air located in the spool section is drained; the resulting differential pressure exceeds the capacity of the aluminum foils causing both sets to fail. This in turn results in the rapid release of compressed air from the driver section through the spool section and into the expansion chamber in the form of a shockwave. Once the shockwave reaches the rigid end test frame depicted in Figure 3-22, the shockwave is collected by a load transfer device (LTD) which covers the entire shock tube opening and transfers the blast pressures onto the specimen. The pressure experienced at the rigid end frame section is referred to as the "reflected pressure" and is measured through the use of two pressure sensors located at the bottom and side of the shock-tube LTD. The shock-tube is also equipped with a total of 12 pressure relief vents which help dissipate any additional pressure contained within the shock-tube.

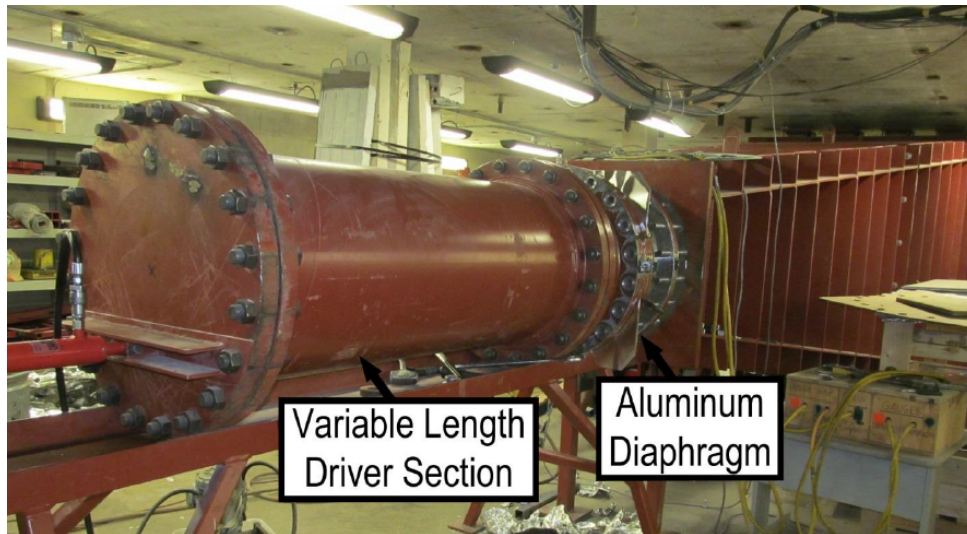


Figure 3-20 Shock Tube Driver Section (Adapted from (Melancon, 2015))

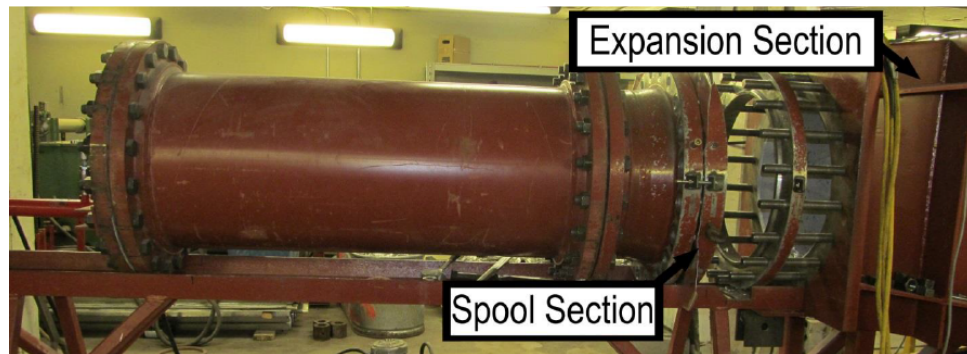


Figure 3-21 Shock Tube Spool Section (Adapted from (Melancon, 2015))

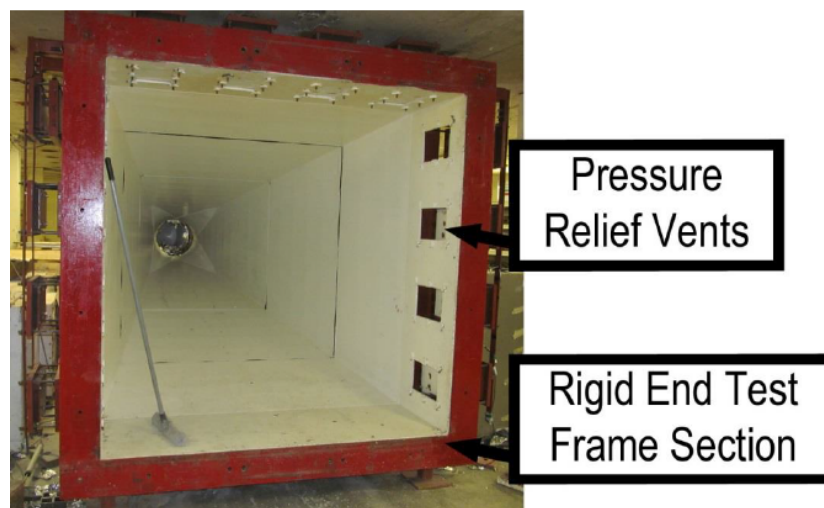


Figure 3-22 Shock Tube's Rigid End Test Frame Section (Adapted from (Melancon, 2015))

3.5.1.2. Lateral Load Transfer Device

The lateral load transfer device (LTD) is designed to collect and transfer the imposed shockwave from the expansion chamber to the specimen being tested. The shock-tube LTD was designed to replicate the quasi-static loading and support conditions to test the specimens under third-point bending. The load transfer system, consists of two individual load transfer devices which were designed to operate independently of each other, however, stiffened W150x30 steel beams are used to connect the two LTDs together at the load application points (Jaques & Saatcioglu, Effect of High Strain Rate on Reinforced Concrete Structures, 2011). The load transfer device system measures 2032mm by 2030mm which effectively covers the entirety of the test frame section. Frictionless hinges are provided at the top and bottom of the load transfer device to effectively transfer the lateral load without incurring any moment reactions at the ends as illustrated in Figure 3-23. The entirety of the system has a tributary area of 3.4m^2 and a mass of 283.6 kg (Jaques & Saatcioglu, Effect of High Strain Rate on Reinforced Concrete Structures, 2011).

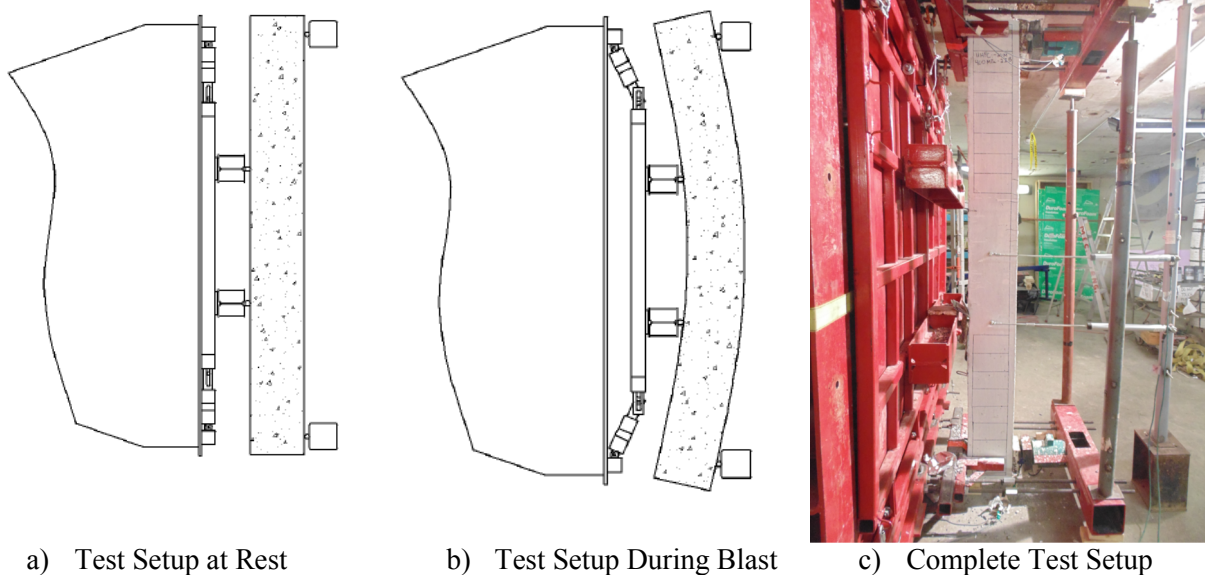


Figure 3-23 Dynamic Load Transfer Device and Setup. (Adapted from (Jaques & Saatcioglu , 2011))

3.5.1.3. Supports

The supports provided in the dynamic setup replicates that of the quasi static tests. The setup results in an unsupported beam span of 2232mm which can be sub-divided into two shear spans of 741mm measured from the top and bottom supports to the load application points, followed by a constant moment region of 750mm. The load is transferred at the point load locations using two stiffened I-beam sections. The beam specimen is supported up against the load transfer device through support assemblies which connect the beams using 4 -19mm threaded rods at the top and bottom. The outer support at each end consists of a 1000mm long 152mm x 152mm hollow square section equipped with an integral roller and a load-cell which is used to record support reactions such as

depicted in Figure 3-24. It is important to note that the top and bottom supports were only lightly tightened to adequately hold the beam in place and provide simply supported boundary conditions.

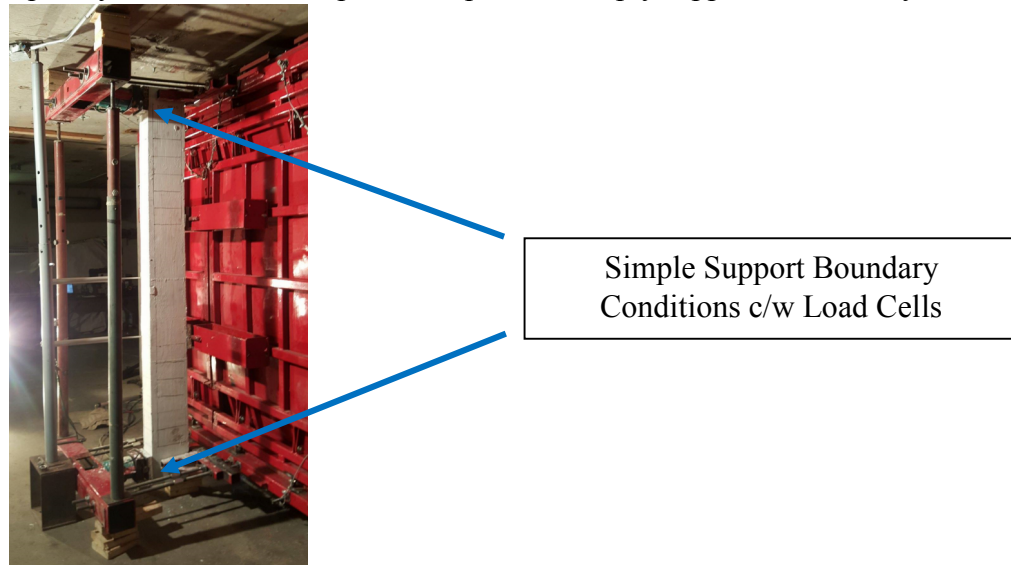


Figure 3-24 Simply Supported Boundary Conditions and Support Assemblies

3.5.1.4. Data Acquisition System

The data from the dynamic testing was collected using a Yokogawa SL1000 High Speed Data Acquisition Unit, having a recording capacity of 100, 000 samples of data per second. The system begins recording once the trigger is activated through a pressure sensor located at the rigid end frame opening, and is capable of recording strain data, displacements, pressure readings and support reactions for the duration of the experiment. The data can then be exported through software XViewer in a .csv format for full analysis.

3.5.1.5. High Speed Video Camera

All shock-tube tests were recorded using a high-speed video camera (AOS Technology X-PRI) in order to better visualise and understand the specimens' behaviour under extreme dynamic loading. The 800x600 pixel resolution camera can record up to 1000 frames per second in full colour images for optimized analysis. The camera was installed perpendicular to the side of the beam specimens, this also enabled lateral displacement monitoring in the event of data acquisition malfunction.

3.5.1.6. Pressure Sensors

The PCB Piezotronics Model #112A22 piezoelectric pressure sensors are key components of the experimental test setup as they record the reflected pressure time history for each blast imposed on a specimen. The sensors are also used to trigger the high-speed video camera and data acquisition system. The setup is equipped with two pressure sensors; one located at the bottom and one located on the side of the rigid end frame section of the shock tube.

3.5.1.7. Linear Variable Displacement Transducers (LVDT)

A total of two CLWG-300 linear variable displacement transducers (LVDT) were installed at mid-span and third span of each beam specimen to record the in-plane lateral displacements during the

experiments, such as illustrated in Figure 3-25; the LVDTs allow for extraction of the maximum and residual displacements data. A third LVDT was fixed to the shock-tube and laboratory slab in order to measure any relative displacements which could then be subtracted from the actual specimen displacement reading. The maximum LVDT displacement measurement is 300mm.

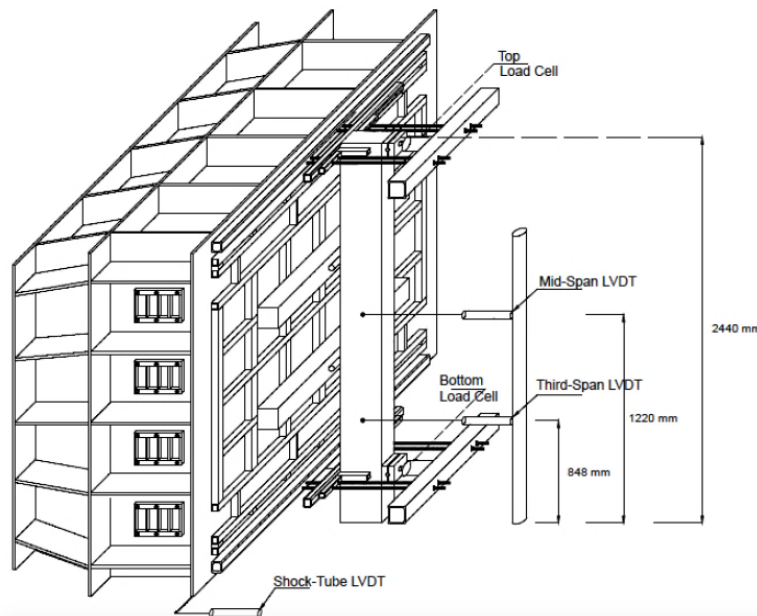


Figure 3-25 Linear Variable Displacement Transducers Setup (Adapted from (Algassem, Parameters, 2016))

3.5.1.8. Strain Gauges

The strain experienced in the longitudinal reinforcement steel bars was recorded through the use of FLA-6-350-11 strain gauges having a nominal length of 6mm and an electrical resistance of 350 ± 1.0 ohms attached at the rebar mid-span such as presented in Figure 3-12 . The acquired data indicated the yielding of the reinforcement and enabled further detailed analysis of each specimen.

3.5.1.9. Experimental Procedure

Each beam specimen was subjected to a series of gradual increasing blast loads until failure. All specimens followed the same test sequence using a 2743mm (9 ft) driver section such as presented in Figure 3-26, while the driver pressure was gradually increased in 10-20 psi increments. Driver properties and representative reflected impulses, pressures and positive phase duration values can be found in Table 3-9. It is important to note that although the failure blast of each specimen is not entirely representative of its actual ultimate blast capacity due to the use of repeated loading, the use of multiple blasts allowed for testing under elastic, yield and ultimate conditions, while also allowing for comparative test data for specimens having different reinforcement ratios and materials, while taking into account limited time and material availability.

Table 3-9 Incremental Shockwave Properties

Test #	Driver Pressure <i>KPa (psi)</i>	Driver Length <i>mm (ft)</i>	Avg. reflected impulse, I_r <i>kPa-ms</i>	Avg. Reflected Pressure, P_r <i>kPa</i>	Avg. positive phase duration, t_p <i>ms</i>
Blast 1	117 (17)	2743 (9)	246	22.3	23.85
Blast 2	207 (30)	2743 (9)	362	35.7	20.32
Blast 3	345 (50)	2743 (9)	533	48.8	20.76
Blast 4	483 (70)	2743 (9)	721	62.2	22.08
Blast 5	621 (90)	2743 (9)	906	82.0	22.52
Blast 6	759 (110)	2743 (9)	1078	94.2	23.41
Blast 7	828 (120)	2743 (9)	1085	97.7	23.41

Sample Pressure Time-Histories

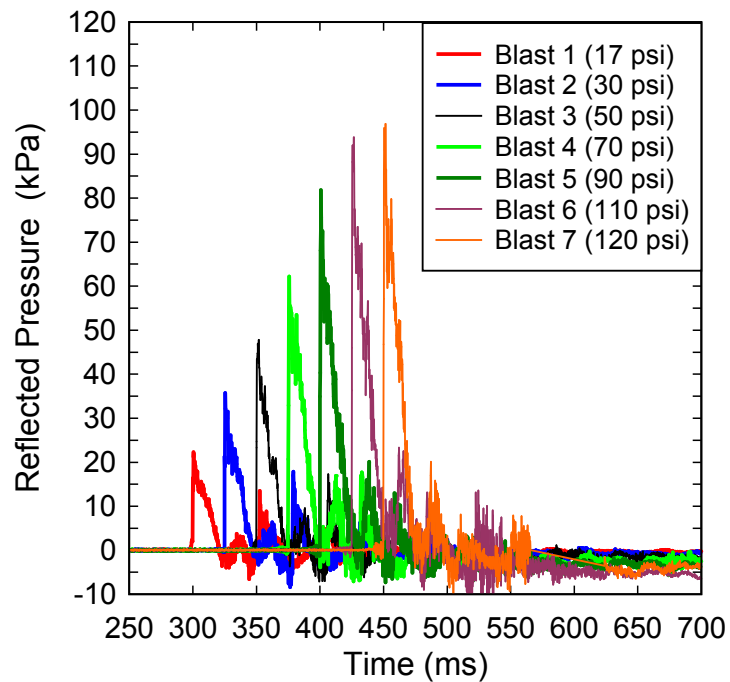


Figure 3-26 Sample Pressure Time Histories for Sequential Blasts (Arrival Time Shifted)

3.5.2. Static Testing

3.5.2.1. Load Setup and Instrumentation

The quasi-static tests conducted on a total of 7 beam specimens were completed using a third-point bending setup with simply supported boundary conditions as illustrated in Figure 3-27. The setup consists of a 2232mm unsupported span, alongside a constant moment region of 750mm, and two shear spans of 741 mm, resulting in a shear-span to depth ratio (a/d) of 2.96. The imposed load was applied using a hydraulic jack connected to a manual pump, and then transferred from the jack using an I-beam which concentrated the loading as two point loads located at 741mm from the specimen supports. The imposed load was monitored using two load cells placed at the support locations (with total load being the sum of the two), while displacements were recorded using of two cable displacement transducers (DCTs) attached to the beam at midspan. Strains in the tension steel were captured using strain gauges located at the mid-span of the longitudinal reinforcing bars. The loading, strain readings and displacement data was recorded using a data acquisition system connected to a computer using strain gauge wires.

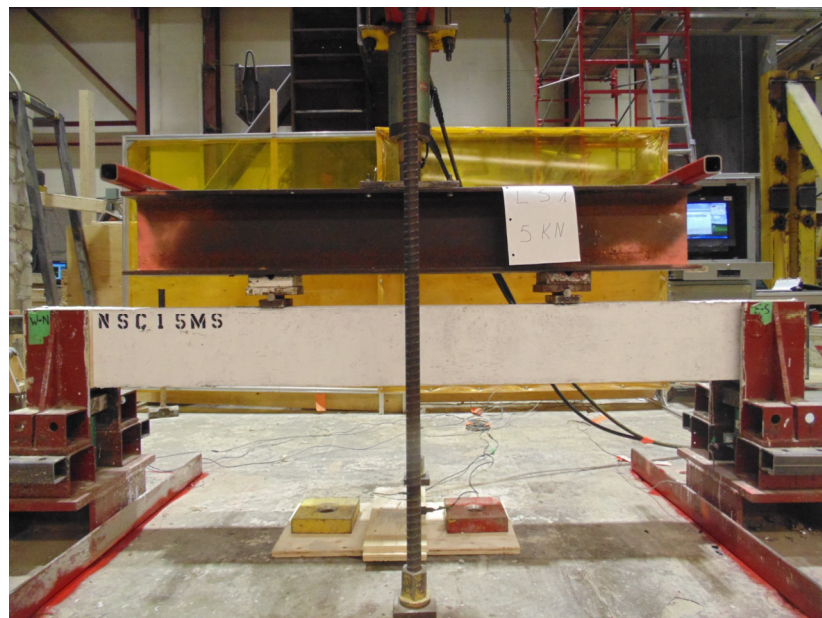


Figure 3-27 Simply Supported Quasi-Static Test Setup

3.5.2.2. Loading Sequence

The quasi-static testing was first carried out using a load-controlled loading scheme, where the load was incrementally augmented by 10kN at a time; at which point manual crack measurements were taken. Once the specimen reached yielding of longitudinal reinforcement, the loading scheme was altered to displacement-controlled, where at every additional 5mm mid-span displacement crack measurements were taken. Photographs of the displacement progression were taken at each load stage such as illustrated in Figure 3-27 a). Specimens were loaded until ultimate failure such as; shear failure, concrete crushing or rebar rupture.

Chapter 4. Results – Static Tests

4.1. Chapter Overview

This chapter presents the detailed experimental results for the static four-point bending tests conducted on a total of seven reinforced concrete beams. Individual beam results are presented in this chapter, while the comparison of results is presented in Chapter 5.

Among the results provided for each beam is: a load-displacement curve, a strain vs. load curve, a plot of the flexural and shear crack widths vs. load, as well as a pictorial description of loading from beginning to failure alongside the major events that occurred.

In addition, tables are provided for each beam to summarize: Peak (maximum) load resisted ($P_{\max}$), load and displacement at yield (P_y , Δ_{yield}) and displacement at failure $\Delta_{\max}$, defined as the displacement corresponding to a reduction of 15% in peak load capacity or rupture of tension steel reinforcement. Since the high-strength ASTM A1035 (MMFX) reinforcement does not show a well-defined yield plateau, the method presented in Figure 4-1 was used to approximate yield displacement for specimens CRC-2%A-#4-HSR-S and CRC-2%A-#6-HSR-S.

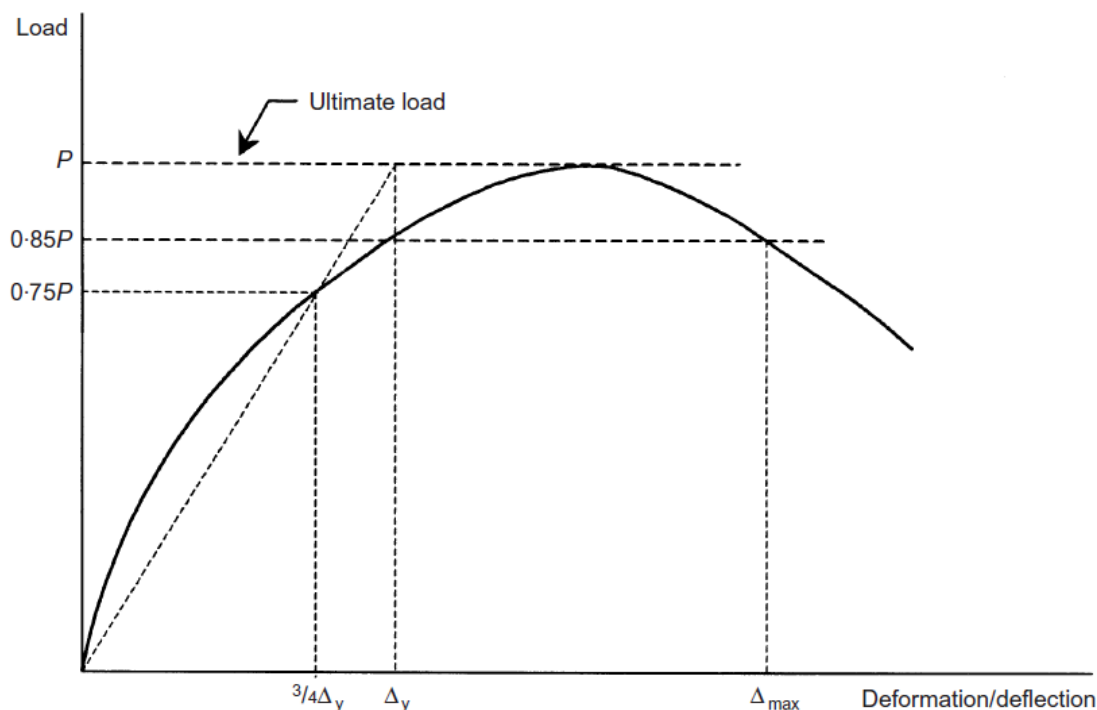


Figure 4-1: Yield Displacement of High Strength Steel Reinforced Beam. (Pam, Kwan, & Islam, 2001)

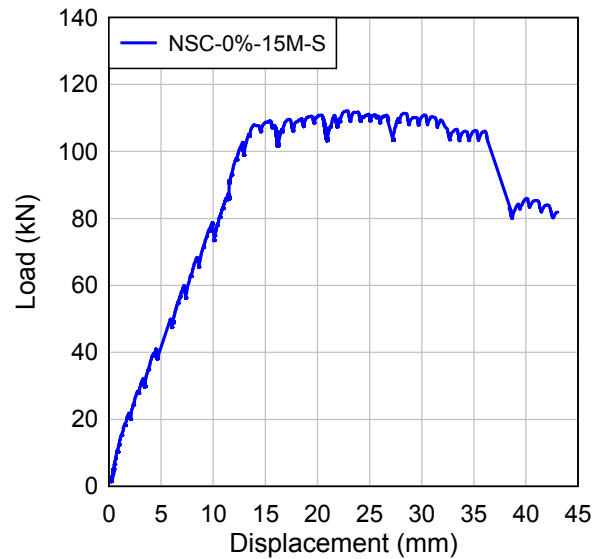
4.2. Description of Experimental Results - Static Beams

4.2.1. NSC-0%-15M-S

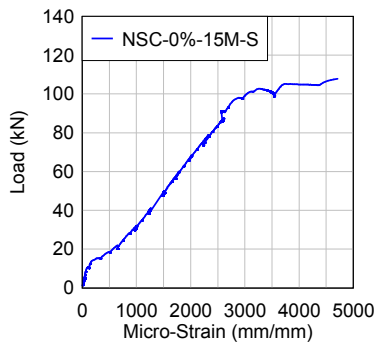
The first static specimen of the control series is designed with normal strength concrete and is reinforced with 15M normal strength longitudinal rebar ($\rho = 1.50\%$) and transverse reinforcement spaced at 100mm in the shear spans. The complete load-deflection curve, load vs. tensile reinforcement strain relationship and the flexural and shear crack-width relationships are presented in Figure 4-2, followed by detailed pictorial description of major events throughout loading in Figure 4-3. Yielding for this specimen occurred at a displacement of 12.6 mm. The maximum load resisted by the specimen is of 112 kN found in the post-yielding plateau at a corresponding displacement of 22 mm. The specimen experienced a flexural failure mode, with yielding of the longitudinal steel, followed by concrete crushing resulting in a 75% drop in capacity ($\Delta_{\max} = 37.7$ mm), with a final recorded displacement of 43mm.

Table 4-1 Summary of results for Beam NSC-0%-15M-S

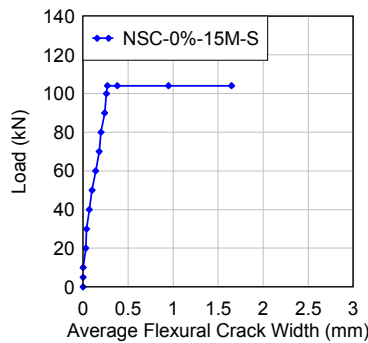
P_y	$P_{\max}$	Δ_{yield}	$\Delta_{\max}$	Failure Mechanism
102.4	112.0	12.6	37.7	Concrete Crushing



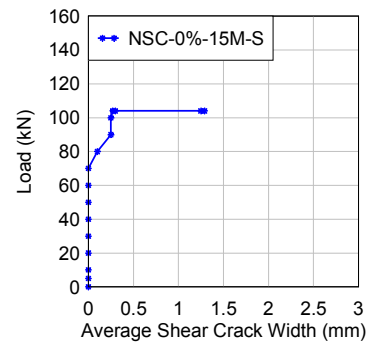
a) Load vs. Mid-Span Deflection



b) Tensile Strain in Long. Rebar



c) Max Crack-width (Flexure)



d) Max Crack-width (Shear)

Figure 4-2 Experimental Results for Beam NSC-0%-15M-S

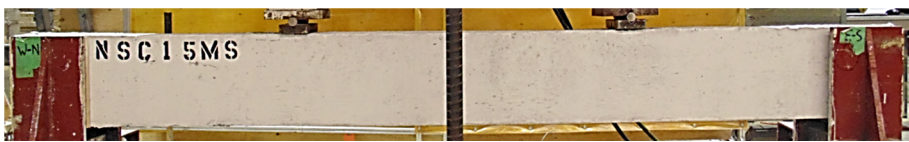
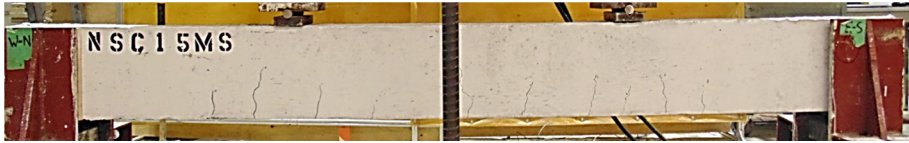
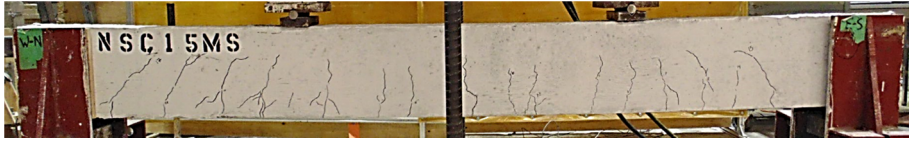
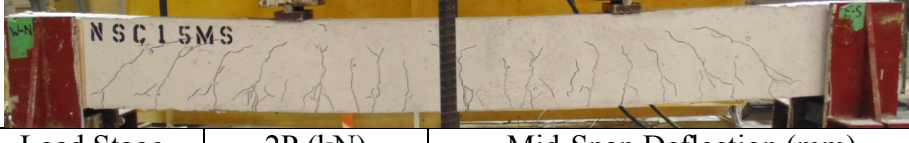
Load Stage	2P (kN)	Mid-Span Deflection (mm)	Comments
1	5.0	0.44	- First hairline crack appears in flexural zone
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- Cracks 1 through 6 further develop
4	30.0	3.35	
			- Crack 7 to 9 appear
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- Progression of diagonal cracks in both left and right shear spans
9	80.0	10.61	
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- Specimen yields
11	100.0	12.45	
			- Average flexural and shear cracks reach 0.25 mm and 0.27 mm, respectively
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- Plastic hinge forms and deflections is concentrated at mid-span while load remains in constant plateau
14	104.0	31.52	
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- Failure mode consists of concrete crushing
17	82.0	43.00	
			- Average flexural and shear cracks are recorded at 1.65 and 1.27 mm respectively

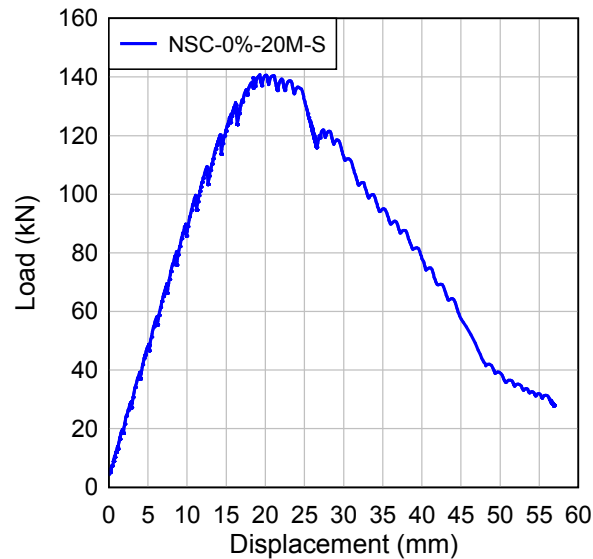
Figure 4-3 Major Events for Specimen NSC-0%-15M-NSR-S

4.2.2. NSC-0%-20M-S

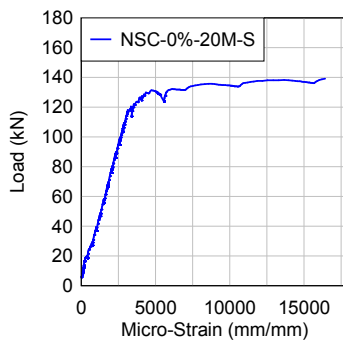
The second static specimen of the control series is designed with normal strength concrete and is reinforced with 20M normal strength longitudinal rebar ($\rho = 2.40\%$) and transverse reinforcement spaced at 100mm in the shear spans. The complete load-deflection curve, load vs. tensile reinforcement strain relationship and the flexural and shear crack-width relationships are presented in Figure 4-4, followed by detailed pictorial description of major events throughout loading in Figure 4-5. Yielding for this beam occurred at 16.2mm. The maximum load resisted by the specimen is of 140.7 kN found in the post-yielded peak at a corresponding displacement of 19.3mm. The specimen experienced a flexural failure mode, with yielding of the longitudinal steel, followed by gradual concrete crushing failure. The maximum displacement corresponding to 15% drop in capacity (Δ_{max}) was found to be 26.6 mm, with testing stopped at 56.8mm.

Table 4-2 Summary of results for Beam NSC-0%-20M-S

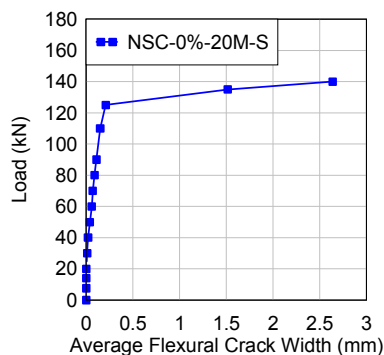
P_y	P_{max}	Δ_{yield}	Δ_{max}	Failure Mechanism
131.4	140.7	16.1	26.6	Concrete Crushing



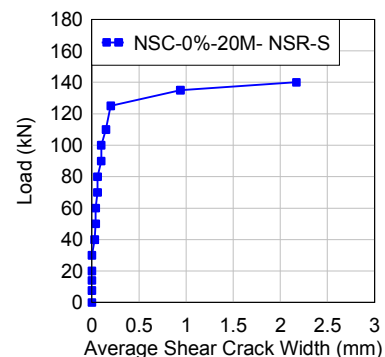
a) Load vs. Mid-Span Deflection



b) Tensile Strain in Long. Rebar



c) Max Crack-width (Flexure)



d) Max Crack-width (Shear)

Figure 4-4 Experimental Results for Beam NSC-0%-20M-S

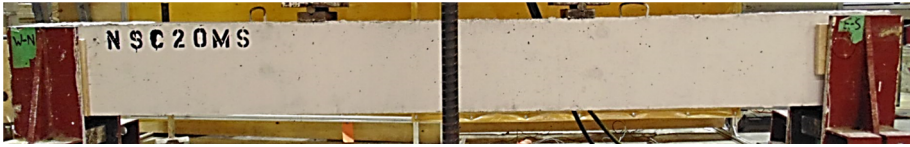
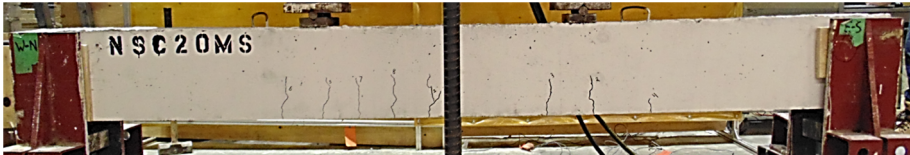
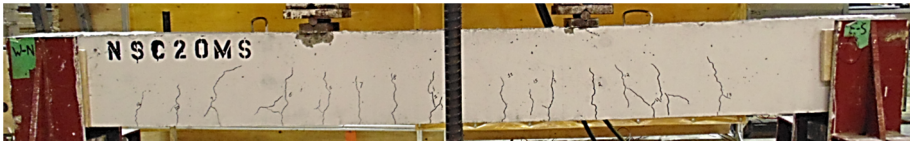
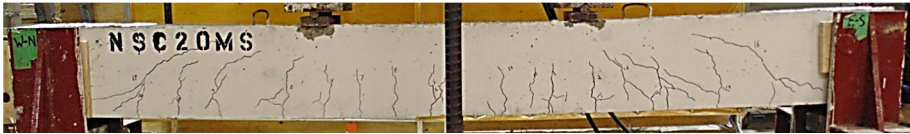
Load Stage	2P (kN)	Mid-Span Deflection (mm)	Comments
1	5.0	0.19	- First hairline crack appears in flexural zone
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- Cracks 1 through 8 further develop
4	30.0	2.91	- Cracks 9 and 10 appear
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- Cracks begin propagating in a diagonal fashion (flexural-shear cracks)
9	80.0	8.92	
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- Yielding occurs
12	120	15.07	- Well defined shear cracks occur near right and left supports
			- Average flexural and shear cracks are 0.21 mm and 0.2 mm respectively
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- Concrete crushing occurs
15	140.0	49.80	- Mid-span displacement at end of testing is 56 mm
			- Maximum flexural and shear cracks are recorded at 2.64 mm and 2.17 mm respectively

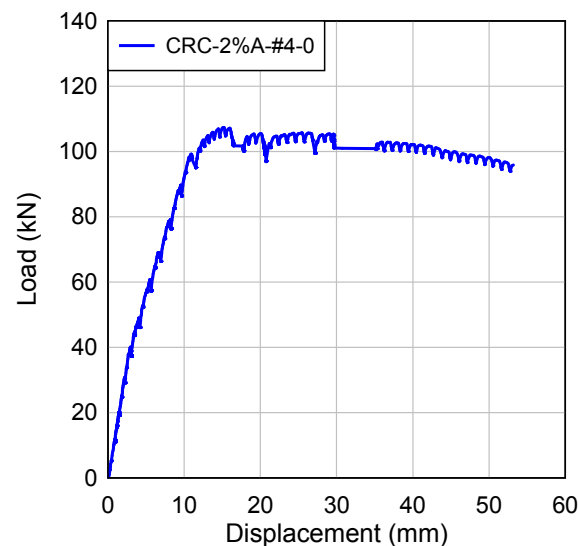
Figure 4-5 Major Events for Specimen NSC-0%-20M-S

4.2.3. CRC-2%A-#4-0

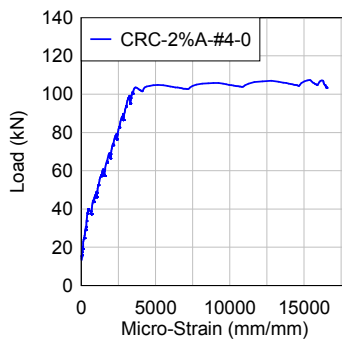
This first ultra-high performance concrete and normal strength reinforcement specimen is reinforced with #4 longitudinal rebar ($\rho = 1.0\%$) and no transverse reinforcement in the shear spans. The complete load-deflection curve, load vs. tensile reinforcement strain relationship and the flexural and shear crack-width relationships are presented in Figure 4-6, followed by detailed pictorial description of major events throughout loading in Figure 4-7. Despite the lack of shear reinforcement, yielding of the longitudinal steel occurred at 10.7mm. The maximum load resisted by the specimen is of 107.3 kN occurring shortly after longitudinal steel yielding at a displacement of 15.1 mm. The specimen experienced a flexural failure mode, with development of large deflections prior to abrupt rupture of the longitudinal reinforcement. The specimen demonstrated a well-defined yield plateau ranging from 11 mm to it's maximum deflection 53 mm at failure. It is noted that the specimen showed virtually no crushing in the compression and limited crack widths prior to rebar rupture.

Table 4-3 Summary of results for Beam CRC-2%A-#4-0

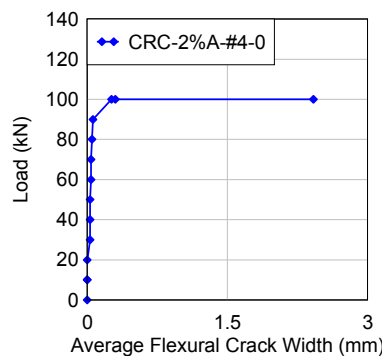
P_y	P_{max}	Δ_{yield}	Δ_{max}	Failure Mechanism
98.9	107.3	10.7	53.0	98.9



a) Load vs. Mid-Span Deflection



b) Tensile Strain in Long. Rebar



c) Max Crack-width (Flexure)

-No Shear Cracks Developed

d) Max Crack-width (Shear)

Figure 4-6 Experimental Results for Beam CRC-2%A-#4-0

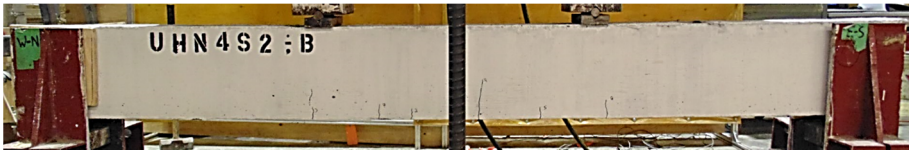
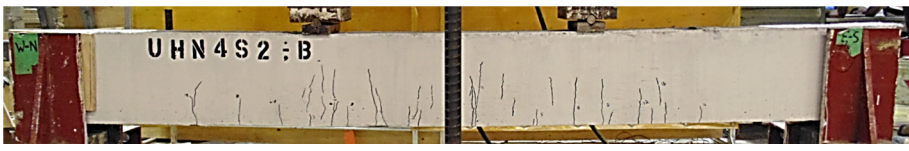
Load Stage	2P (kN)	Mid-Span Deflection (mm)	Comments
2	10.0	0.70	- No cracks are visible to the human eye
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- A total of 7 hairline cracks appear in the constant moment region
5	40.0	3.18	
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- Numerous flexural hairline cracks appear with an average width of 0.05 mm
10	90.0	9.18	
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- Yielding of longitudinal reinforcement occurs
12	100.0	11.66	
			
			- Plastic hinge forms at mid-span of beam
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- Specimen remains in yield plateau until rebar rupture occurs at a load of 95 kN at a displacement of 53 mm
14	107.0	53.42	
			
			- Maximum flexural and shear cracks are recorded at 2.64 mm and 2.17 mm respectively

Figure 4-7 Major Events for Specimen CRC-2%A-#4-0

4.2.4. CRC-2%A-15M-S

The second static specimen of the UHPC & NSR series is designed with CRC concrete and is reinforced with 15M normal strength longitudinal rebar ($\rho = 1.50\%$) and transverse reinforcement spaced at 100mm in the shear spans. The complete load-deflection curve, load vs. strain relationship and the flexural and shear crack-width relationships are presented in Figure 4-8, followed by detailed pictorial description of major events throughout loading in Figure 4-9. Yielding in this specimen occurred at 13.4mm. The maximum load resisted by the specimen is of 139.5 kN occurring shortly after yielding at a corresponding displacement of 14.5 mm. The specimen experienced a flexural failure mode with rupture of longitudinal reinforcement at 76% of peak load, with a maximum displacement of 47 mm. As with the previous specimen the beam showed large deflections but limited damage prior to bar rupture.

Table 4-4 Summary of results for Beam CRC-2%A-15M-S

P_y	P_{max}	Δ_{yield}	Δ_{max}	Failure Mechanism
138.8	139.5	13.4	47.0	Rebar Rupture

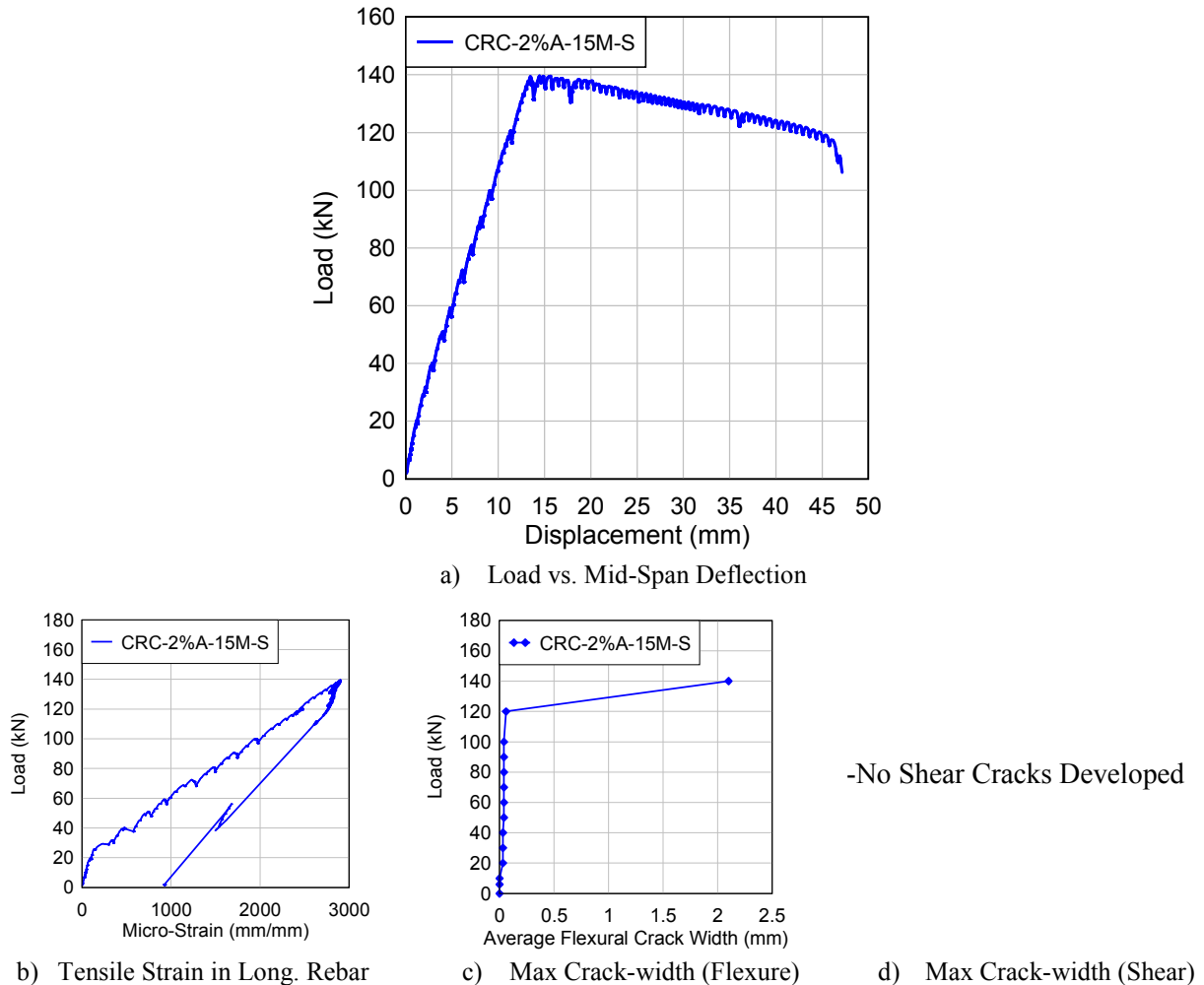


Figure 4-8 Experimental Results for Beam CRC-2%A-15M-S

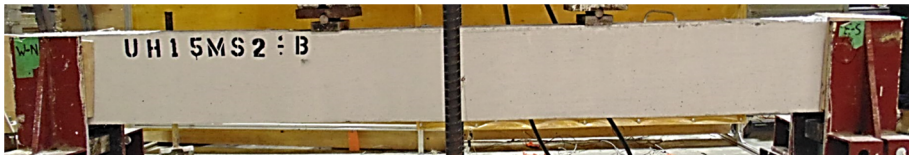
Load Stage	2P (kN)	Mid-Span Deflection (mm)	Comments
1	5.0	0.56	- No cracks are visible to the human eye
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- A total of 12 hairline cracks have developed in the flexural zone
6	50.0	4.24	
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- Total of 22 flexural cracks, with an average width of 0.04 mm
10	90.0	8.19	
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- Specimen yields and reached its peak load of 140 kN
14	140.0	14.50	
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- Plastic hinge develops at mid-span
17	107.0	47.0	
			
			- 15M longitudinal bars rupture under left hand side support at a displacement of 47.0 mm
			- Maximum flexural crack width prior to rupture is 8.3 mm
			- No shear cracks appear throughout loading

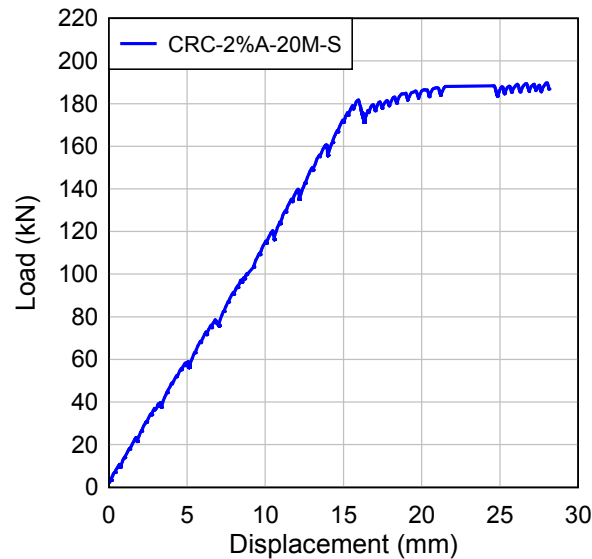
Figure 4-9 Major Events for Specimen CRC-2%A-15M-S

4.2.5. CRC-2%A-20M-S

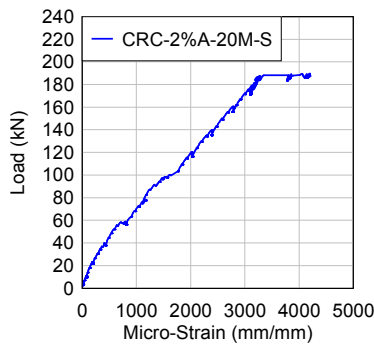
The third static specimen of the UHPC & NSR series was built with CRC and reinforced with 20M normal strength longitudinal rebar ($\rho = 2.40\%$) and transverse reinforcement spaced at 100mm in the shear spans. The complete load-deflection curve, load vs. tensile reinforcement strain relationship and the flexural and shear crack-width relationships are presented in Figure 4-10, followed by detailed pictorial description of major events throughout loading in Figure 4-11. This beam experienced yielding at $\Delta = 16\text{mm}$. The maximum load resisted by the specimen was 190 kN found in the post-yielding plateau at a corresponding displacement of 28.0 mm. The specimen experienced a flexural failure mode with rupture of longitudinal reinforcement upon reaching its peak load ($\Delta_{\text{max}} = 28\text{ mm}$). As with previous beams in this series, the specimen showed limited damage and relatively large deflections prior to the abrupt rebar failure.

Table 4-5 Summary of results for Beam CRC-2%A-20M-S

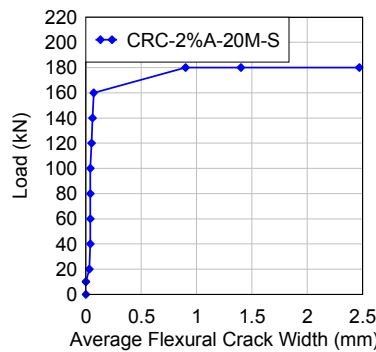
P_y	P_{max}	Δ_{yield}	Δ_{max}	Failure Mechanism
182.3	190.0	16.0	28.0	Rebar Rupture



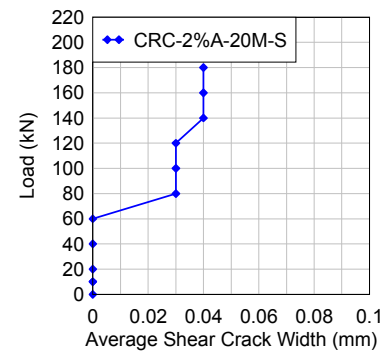
a) Load vs. Mid-Span Deflection



b) Tensile Strain in Long. Rebar



c) Max Crack-width (Flexure)



d) Max Crack-width (Shear)

Figure 4-10 Experimental Results for Beam CRC-2%A-20M-S

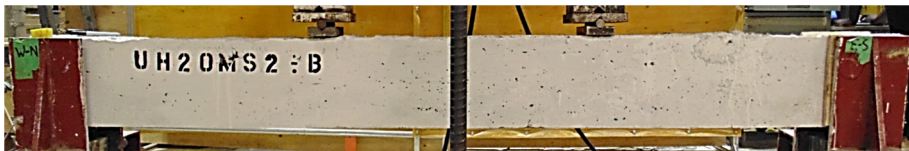
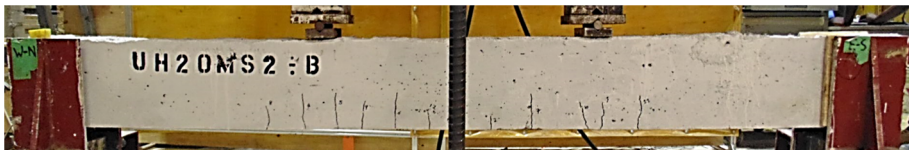
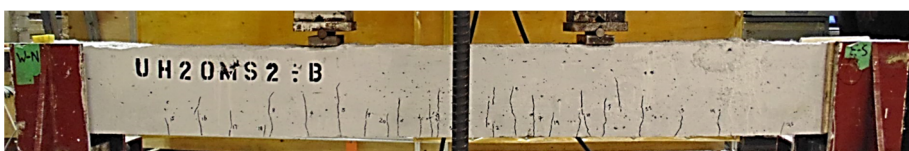
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- No cracks are visible to the human eye
2	10.0	0.79	
			- A total of 14 hairline cracks have developed over the beam span, concentrating mostly in the flexural zone
Load Stage	2P (kN)	Mid-Span Deflection (mm)	
5	60.0	5.35	- Total of 22 flexural cracks, with an average width of 0.05 mm
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- Specimen yields at a displacement of 16 mm
8	120.0	10.71	
			- First flexural-shear crack appears near the left-hand side support
Load Stage	2P (kN)	Mid-Span Deflection (mm)	
10	180.0	15.90	- 20M longitudinal bars rupture under left-hand load point at displacement of 28 mm
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- Maximum flexural crack width prior to rupture is 2.47 mm
11	190.0	28.00	
			

Figure 4-11 Major Events for Specimen CRC-2%A-20M-S

4.2.6. CRC-2%A-#4-HSR-S

The first static specimen of the UHPC & HSR series is designed with CRC concrete and is reinforced with #4 MMFX longitudinal rebar ($\rho = 1.0\%$) and transverse reinforcement spaced at 100mm in the shear span. The complete load-deflection curve, load vs. tensile reinforcement strain relationship and the flexural and shear crack-width relationships are presented in Figure 4-12, followed by detailed pictorial description of major events throughout loading in Figure 4-13. The use of high-strength bars in this beam resulted a round-house type load-deflection response which matches the general stress-strain form of the MMFX bars. Yielding was approximated to have occurred at a displacement of 19.46mm using the method discussed in Section 4.1. The maximum load resisted by the specimen is of 181.8 kN found in the “post-yield” region at a corresponding displacement of 30.2 mm. The specimen experienced a flexural failure mode with rupture of longitudinal reinforcement at 77% of P_{max} in the post-peak branch, with a maximum displacement of ~ 40 mm. Despite the lack of a well-defined yield plateau, the specimen experienced relatively high deflection prior to failure. Damage and cracking was well controlled throughout.

Table 4-6 Summary of results for Beam CRC-2%A-#4-HSR-S

P_y	P_{max}	Δ_{yield}	Δ_{max}	Failure Mechanism
160.9	181.8	19.46	39.1	Rebar Rupture

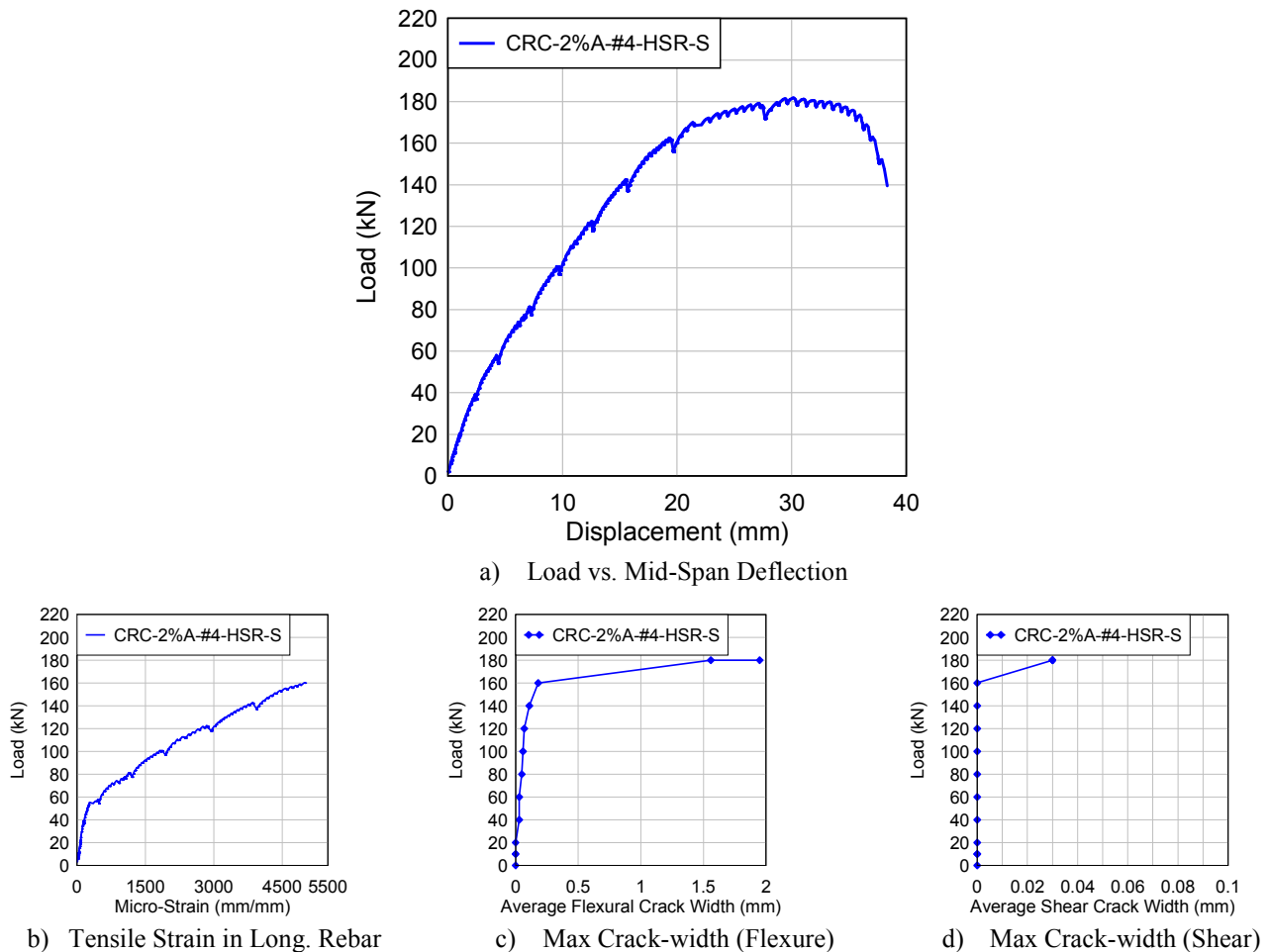


Figure 4-12 Experimental Results for Beam CRC-2%A-#4-HSR-S

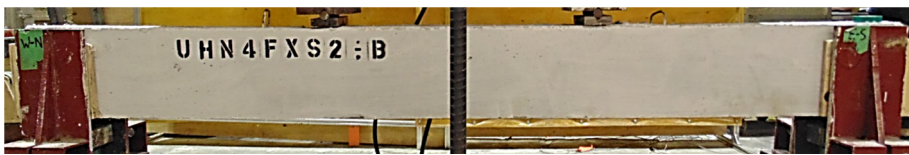
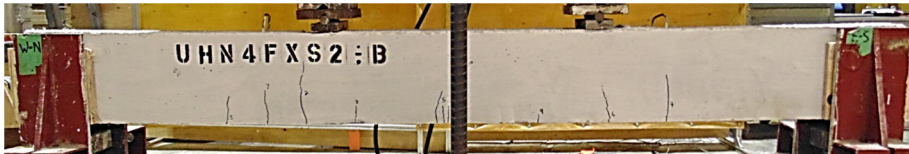
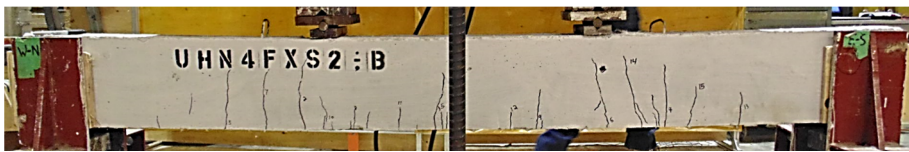
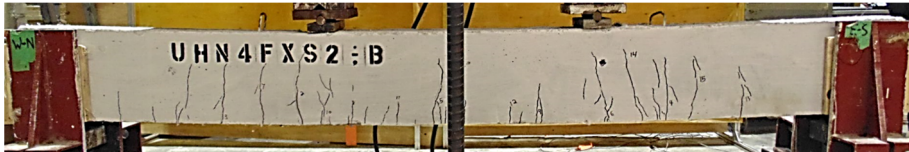
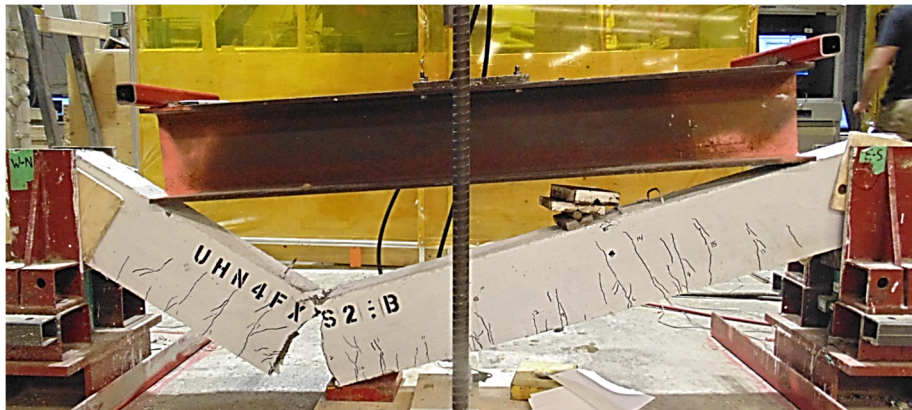
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- No cracks are visible to the human eye
2	10.0	1.85	
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- A total of 10 hairline cracks have developed in the flexural zone
5	60.0	4.51	
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- Total of 19 flexural cracks, with an average width of 0.07 mm
6	120.0		
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- Specimen yields at a displacement of @ 20 mm and an average flexural crack width of 0.18 mm
10	160.0	19.61	
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- #4 MMFX bars rupture under left-hand side load point at a displacement of 39.64 mm
11	180.0	39.64	
			
			- Maximum flexural and shear crack widths prior to rupture are 1.95 mm and 0.03 mm respectively

Figure 4-13 Major Events for Specimen CRC-2%A-#4-HSR-S

4.2.7. CRC-2%A-#6-HSR-S

The final static specimen of the UHPC & HSR series is designed with CRC concrete and is reinforced with #6 high strength MMFX longitudinal rebar ($\rho=2.3\%$) and transverse reinforcement spaced at 100mm in the shear span. The complete load-deflection curve, load vs. tensile reinforcement strain relationship and the flexural and shear crack-width relationships are presented in Figure 4-14, followed by detailed pictorial description of major events throughout loading in Figure 4-15. As with the previous HSR specimen, the beam shows a round-house type load-deflection response with no clear yield point. While the strain gauge failed during testing, yielding was approximated to have occurred at a displacement of 19.4 mm using the method discussed in Section 4.1. The maximum load resisted by the specimen is of 318 kN occurring in the post-yielding segment at a corresponding displacement of 45 mm. The specimen experienced a sudden shear type failure, without any rupture of the longitudinal bars. This failure could have possibly resulted due to honey-combing observed after construction near the left-hand load application point. The companion specimen tested under dynamic loading did not experience a shear failure.

Table 4-7 Summary of results for Beam CRC-2%A-#6-HSR-S

P_y	P_{max}	Δ_{yield}	Δ_{max}	Failure Mechanism
215.6	318.0	28.4	45.6	Shear Failure

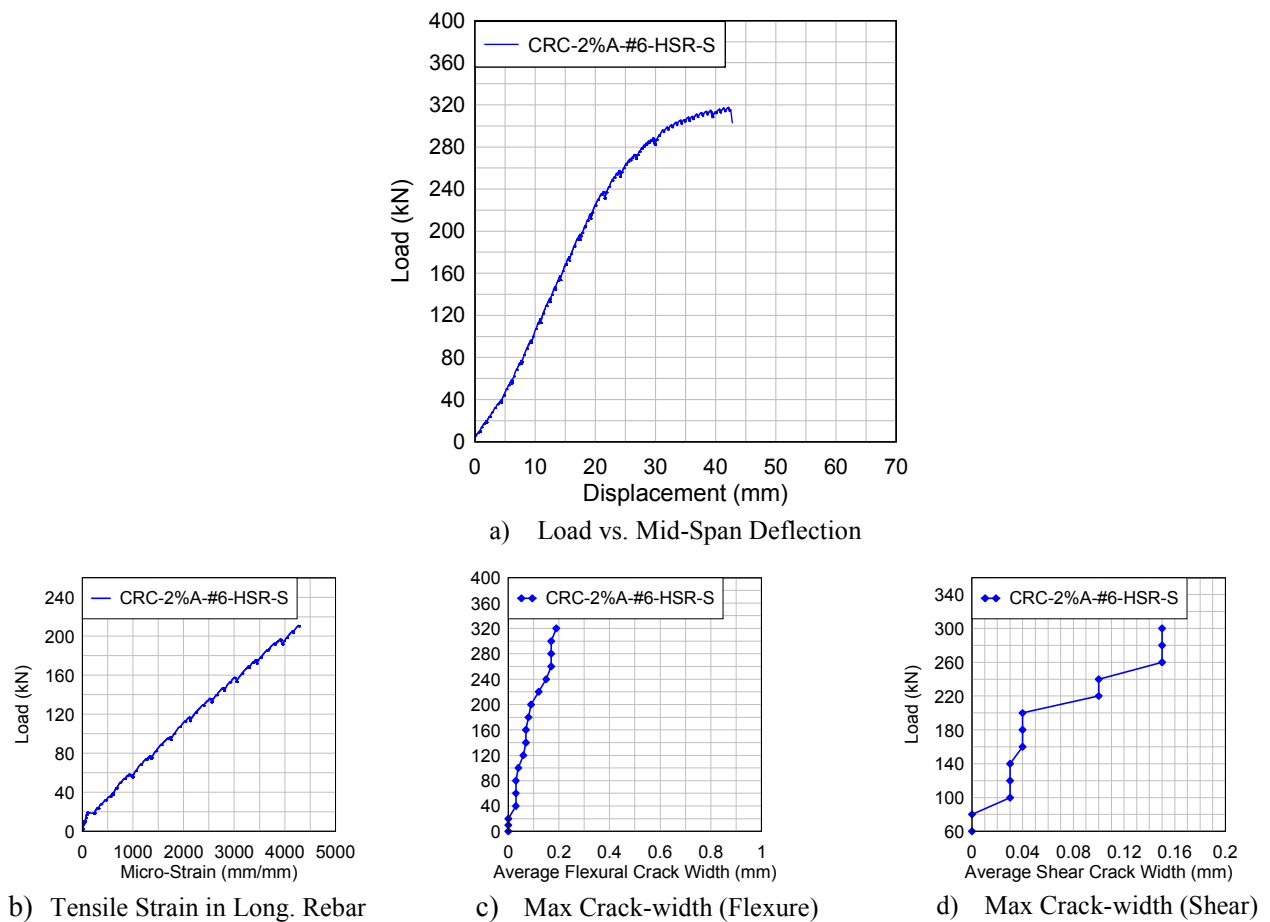


Figure 4-14 Experimental Results for Beam CRC-2%A-#6-HSR-S

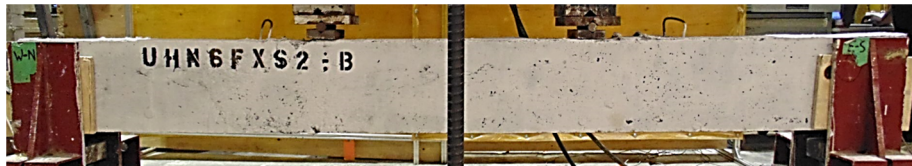
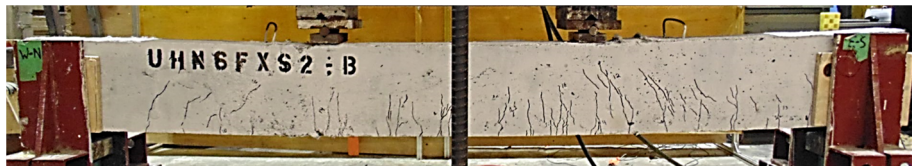
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- No cracks are visible to the human eye
2	10.0	0.58	
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- A total of 19 hairline cracks have developed in the flexural zone
6	80.0	8.06	
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- Specimen yields at a displacement of ~ 28.4 mm, with average flexural crack width of 0.12 mm - Flexural shear cracks begin to form near supports
12	280	28.4	
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- Prominent shear cracks develop in shear span - Displacement concentrations occur at mid-span
17	300	35.24	
			
Load Stage	2P (kN)	Mid-Span Deflection (mm)	- Sudden shear failure below left hand side point load - Pronounced shear cracks in both shear spans - Experimental casting error may have contributed to premature shear failure as minor honeycombing was present in failure trajectory
19	320	45.58	
			

Figure 4-15 Major Events for Specimen CRC-2%A-#6-HSR-S

Chapter 5. Discussion of Static Beam Results

5.1. Chapter Overview

Chapter 5 compares and discusses the experimental results obtained for the 7 reinforced concrete specimens tested under static loading. Several parameters were investigated including the effect of: concrete type, longitudinal reinforcement ratio and type. The response of the beam specimens is compared in terms of peak load resistance, beam stiffness, maximum displacement, ductility, toughness, crack widths, damage progression and failure mechanism.

5.2. Summary of Results and General Observations

This subsection presents a summary of the static test results. Figure 5-1 provides a comparison of the load-displacement curves for all 7 statically tested beams. Further details on specific parameters extracted from the load-displacement curves such as: yield load (P_{yield}) and corresponding displacements (Δ_{yield}), peak load (P_{max}), maximum displacement at failure (Δ_{max}) (where failure corresponds to 15% reduction in capacity), and beam stiffness (K) are presented in Table 5-1. Moreover, the ductility of each specimen defined as the ratio of maximum and yield displacement ($\frac{\Delta_{max}}{\Delta_{yield}}$) and the toughness at multiples of 2, 3 and at failure displacement (area under the load-displacement curve at: $2\Delta_y$, $3\Delta_y$ and Δ_{max}) are presented in Table 5-2. Moreover, these parameters are presented in the form of bar charts in Figure 5-2, Figure 5-2 and Figure 5-3.

A review of the results indicates that design parameters such as reinforcement ratio, concrete type and longitudinal reinforcement type had a major impact on the peak load resisted, maximum displacement at failure and ductility of the specimens, as will be discussed in the sections that follow.

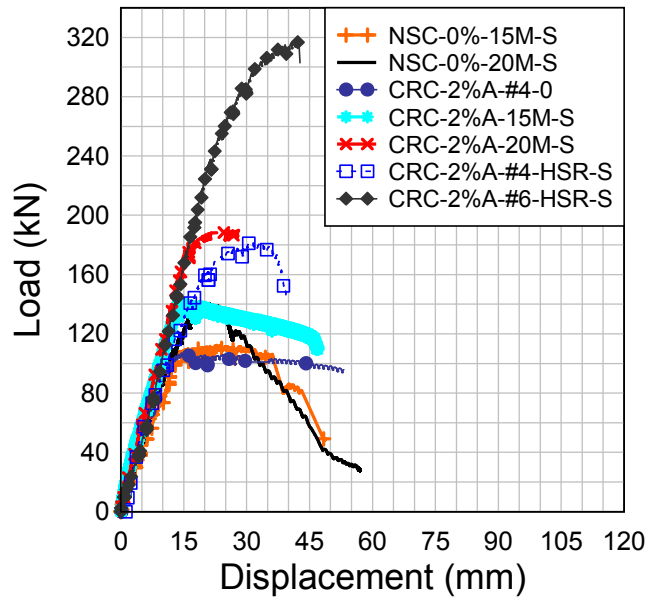


Figure 5-1 Load Displacement Curves for All Specimens

Table 5-1 Summary of Static Test Results (1/2)

Specimen	Load		Displacement		Stiffness
	P_{yield} (kN)	P_{max} (kN)	Δ_{yield} (mm)	Δ_{max} (mm)	K (N/mm)
NSC-0%-15M-S	102.4	112.0	12.6	37.7	7353
NSC-0%-20M-S	131.4	140.7	16.1	26.6	7686
CRC-2%A-#4-0	98.9	107.3	15.1	53.0	7660
CRC-2%A-15M-S	138.8	139.5	13.4	47.0	9244
CRC-2%A-20M-S	182.3	190.0	16.0	28.0	11023
CRC-2%A-#4-HSR-S	160.9	181.8	19.46	39.1	9374
CRC-2%A-#6-HSR-S	280.0	318.0	28.4	45.6	12537

Table 5-2 Summary of Static Test Results (2/2)

Specimen	Ductility	Toughness up to			Failure mode
	$\frac{\Delta_{max}}{\Delta_{yield}}$	$2\Delta_y$	$3\Delta_y$	Δ_{max}	
NSC-0%-15M-S	2.98	2050	3350	3795	Concrete Crushing
NSC-0%-20M-S	1.65	-	-	2570	Concrete Crushing
CRC-2%A-#4-0	3.51	2588	4107	4920	Rebar Rupture
CRC-2%A-15M-S	3.50	2847	4508	5344	Rebar Rupture
CRC-2%A-20M-S	1.75	-	-	3710	Rebar Rupture
CRC-2%A-#4-HSR-S	1.94	-	-	5155	Rebar Rupture
CRC-2%A-#6-HSR-S	1.61	-	-	9275	Shear Failure

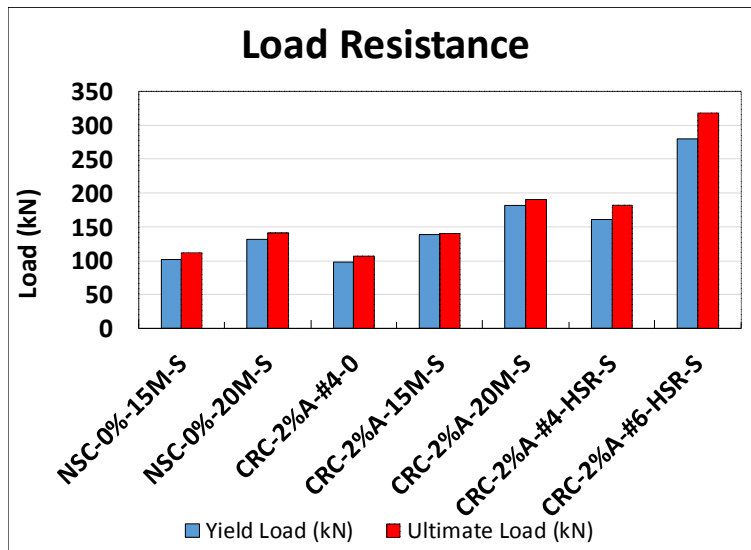


Figure 5-2 Specimen Maximum and Yield Load Comparison

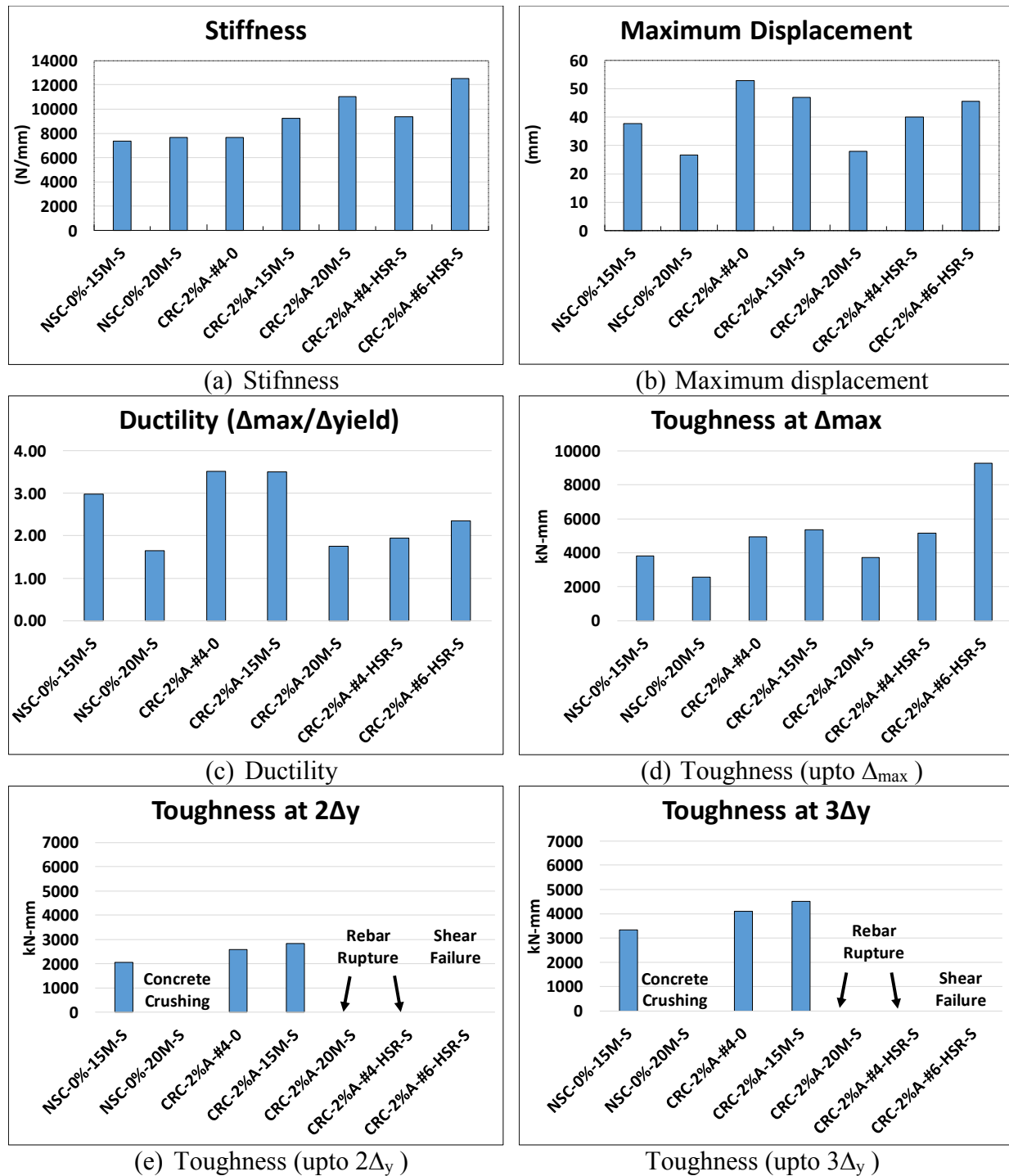


Figure 5-3 Specimen Stiffness, Maximum Displacement, Ductility and Toughness Comparison

5.3. Effect of Reinforcement Ratio

Throughout the experimental program the reinforcement ratio was found to have a great impact not only on beam capacity, but also on the failure mode. The effects of reinforcement ratio are best assessed by examining the response of the following specimens in three series:

- Control Specimens
 - NSC-0%-15M-S vs. NSC-0%-20M-S
- UHPC & NSR Specimens
 - CRC-2%A-#4-0 vs. CRC-2%A-15M-S vs. CRC-2%A-20M-S
- UHPC & HSR Specimens
 - CRC-2%A-#4-HSR-S vs. CRC-2%A-#6-HSR-S

5.3.1. Effect of Reinforcement Ratio: Control Series Comparison

Two Specimens from the Control Series; NSC-0%-15M-S and NSC-0%-20M-S illustrate the effect of longitudinal steel reinforcement ratio in conventional reinforced concrete beams. Both specimens are designed with normal strength concrete and transverse reinforcement in the shear span spaced at 100 mm o/c, however they have 15M and 20M bars, resulting in varying reinforcement ratios of $\rho = 1.5\%$ and 2.4% , respectively.

Table 5-3 presents a summary of results from both specimens. In addition, Figure 5-4 through Figure 5-6 presents the respective: load-displacement, load-strain, toughness, load resistance, beam stiffness, maximum displacements and ductility of the beams, complimented by photographs illustrating failure modes in Figure 5-5.

The load resistance curves obtained for both specimens are similar in terms or shape, in that they both show flexural failures with yielding regions, however the response of specimen NSC-0%-20M-S at peak was more rounded and parabolic followed by a shorter yield plateau and a linear decrease in capacity until failure at 56.8 mm. In comparison, the NSC-0%-15M-S specimen portrayed a well-defined plateau until experiencing a steep decrease in capacity after the initiation of concrete crushing, and finally failure at a displacement of 48.4 mm. Due to the increase in tension steel area, the peak load resisted by NSC-0%-20M-S ($P_{\max} = 140.7$ kN) is 26% greater then that of NSC-0%-15M-S ($P_{\max} = 112.1$ kN). When examining specimen stiffness, beam NSC-0%-20M-S ($k = 7686$ N/mm) is only slightly stiffer by 4.5% when compared to NSC-0%-15M-S ($k = 7353$ mm); as depicted in Figure 5-4 both specimens follow a similar initial ascending branch which correlated nicely to the longitudinal tension strain values recorded.

As expected, increasing the reinforcement ratio affected failure displacement and ductility. The maximum displacement experienced at failure was increased by 29% from 26.2 mm for the control specimen with 20M reinforcement to 37.7 mm for the control specimen having 15M reinforcement (where failure is considered to occur at a displacement equivalent to 85% of $P_{\max}$). Notably, the beam with smaller reinforcement ratio is found to have a 10% greater ductility ($\mu = 3.83$) than the specimen with larger reinforcement ratio ($\mu = 3.48$). However, the increase in reinforcement ratio from $\rho = 1.5\%$ (NSC-0%-15M-S) to $\rho = 2.4\%$ (NSC-0%-20M-S) proved to have the reverse

impact on specimen energy absorption capacity (toughness), as the beam with larger reinforcement ratio (20M bars) recorded increases of 57%, 32% and 28% at $2\Delta_y$, $3\Delta_y$ and Δ_{max} , respectively.

Both beams in the control series experienced flexural failure modes resulting in concrete crushing at mid-span as depicted in Figure 5-5. Furthermore, it is important to notice that both specimens experienced significant cracking and damage before failure.

Table 5-3 Control Series Overview of Results; Effect of Reinforcement Ratio Comparison

Specimen I.D	Long. ratio ρ (%)	Peak Load P_{max} (kN)	Load and Displ. at yield		Deflection at failure and end of test		Stiffness K (N/mm)
			P_y (kN)	Δ_y (mm)	Δ_{max}^* (mm)	Δ_{end} (mm)	
NSC-0%-15M-S	1.5%	112.1	102.4	12.6	37.7	48.4	7353
NSC-0%-20M-S	2.4%	140.7	131.4	16.1	26.2	56.8	7686

*Displacement at 15% drop in capacity after peak

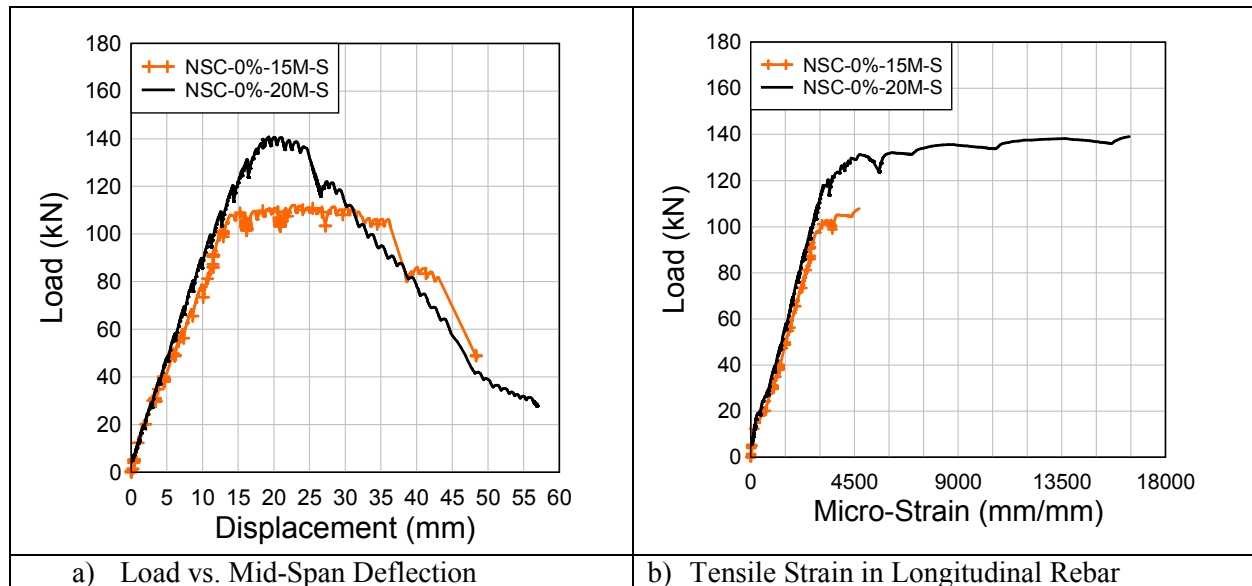


Figure 5-4 Load-Displacement and Load-Strain Curves: Effect of Reinforcement Ratio in Control Series

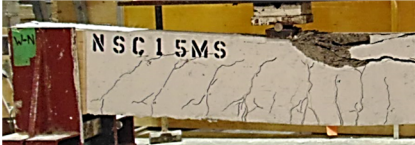
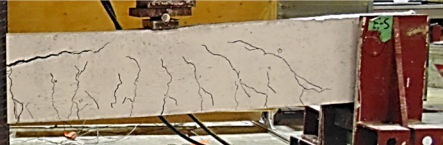
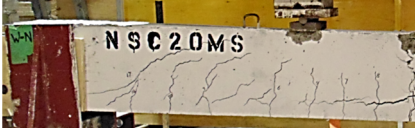
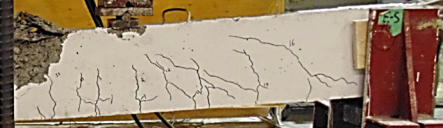
	<u>Failure Mode</u>	<u>Mid-span Δ (mm)</u>
	Concrete Crushing	43.0
NSC-0%-15M-S		
NSC-0%-20M-S	Concrete Crushing	56.0
		

Figure 5-5 Photographs: Effect of Reinforcement Ratio on Control Series

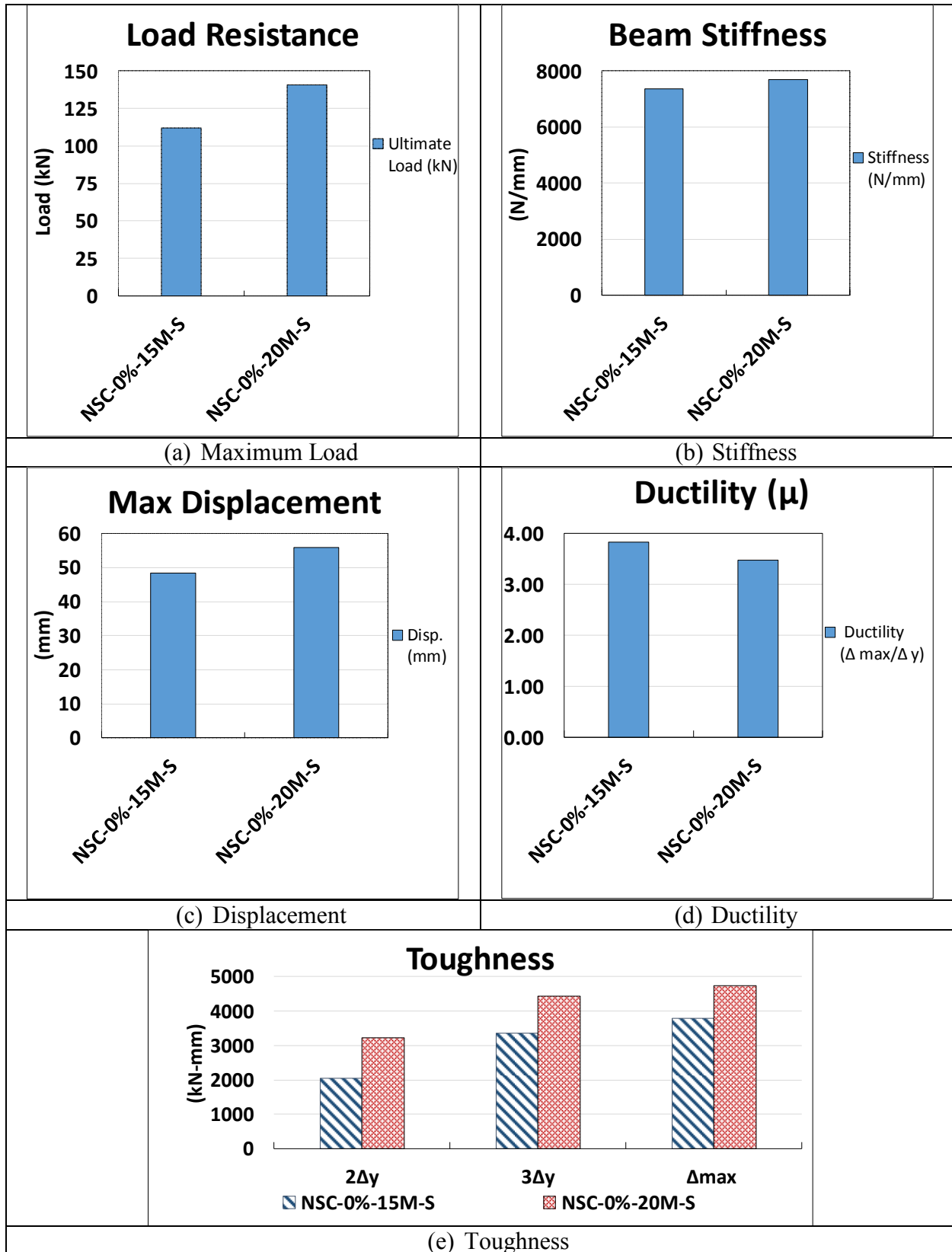


Figure 5-6 Load deflection curve parameters: Effect of Reinforcement Ratio in Control Series

5.3.2. Effect of Reinforcement Ratio: UHPC & NSR Series Comparison

Three Specimens from the UHPC & NSR series, CRC-2%A-#4-0, CRC-2%A-15M-S and CRC-2%A-20M-S, were selected to illustrate the effect of longitudinal steel reinforcement ratio in the UHPC & NSR series. All three specimens are designed with ultra-high performance concrete but had #4, 15M and 20M normal-strength bars, resulting in ratios of 1.0%, 1.5% and 2.4%. While the specimen with #4 reinforcement has no transverse reinforcement, the two companion beams have transverse reinforcement spaced at 100 mm o/c in the shear span. A summary of results for this series is presented in Table 5-4. In addition, Figure 5-7 through Figure 5-9 present the: load-displacement, load-strain, load resistance, beam stiffness, maximum displacements, ductility and toughness, complimented by photographs illustrating specimen failure modes in Figure 5-8.

As expected the increase in reinforcement ratio led to an increase in load-carrying capacity in the UHPC beams; the peak load resisted by CRC-2%A-20M-S ($P_{\max} = 189.7$ kN) is greater than that of CRC-2%A-15M-S ($P_{\max} = 139.5$ kN) and CRC-2%A-#4-0 ($P_{\max} = 107.4$ kN) by factors of 177% and 136%, respectively. When examining specimen stiffness, results show that the 20M beam with $\rho = 2.4\%$ has a stiffness ($k = 11023$ N/mm) which is 19% and 44% greater than the beams with 15M and #4 bars which had reinforcement ratios of 1.5% ($k = 9244$ N/mm) and 1.0% ($k = 7660$ N/mm), respectively. The enhanced stiffness with respect to increasing reinforcement ratio is also illustrated in the load-strain relationship for the longitudinal tension reinforcement in Figure 5-7 b). The load resistance curves obtained for the three specimens prove to have some differences as CRC-2%A-20M-S's response at peak is parabolic with a more sudden failure at a displacement of 28.3 mm, whereas CRC-2%A-15M-S and CRC-2%A-#4-0 had responses with well defined plateaus until failure at corresponding displacements of 47.15 mm and 53.18 mm. As a result of these displacements, it is found that beam ductility is decreased with increasing reinforcement ratio, with the ductility of specimen CRC-2%A-#4-0 ($\mu = 3.51$) being 100% greater than CRC-2%A-20M-S ($\mu = 1.75$), although the ductility of the #4 and 15M specimens are similar ($\mu = 3.50$ for CRC-2%A-15M-S).

The increase in longitudinal reinforcement ratio from CRC-2%A-#4-0 ($\rho = 1.0\%$) to CRC-2%A-15M-S ($\rho = 1.5\%$) proved to have an impact on specimen energy absorption capacity (toughness) as the specimen having 15M reinforcement recorded near constant increases of 10%, 10% and 9% at $2\Delta y$, $3\Delta y$ and $\Delta_{\max}$ when compared to the specimen detailed with #4 reinforcement. However, it is to be noted that the toughness of the specimen having the largest reinforcement ratio (CRC-2%A-20M-S 0 ($\rho = 2.4\%$), did not exhibit a similar trend because of its failure at 28.3 mm. In fact, the specimen recorded a toughness at maximum displacement of 3710 kN-mm², which is inferior to both the 15M and #4 specimen by margins of 31% and 26%, respectively.

One clear trend illustrated in the UHPC and NSR series is the failure mechanism. As depicted in Figure 5-8 all three specimens, regardless of reinforcement ratio, experienced flexural failure governed by rebar rupture. Further assessment of the cracks generating throughout the loading scheme confirms that specimens reinforced with #4 and 15M rebar did not have any diagonal shear cracks, however, the specimen with 20M showed diagonal cracking near the left-hand side support such as presented in Figure 5-11, indicating the increased shear demand in this specimen.

Table 5-4 UHPC & NSR Series Overview of Results: Effect of Reinforcement Ratio Comparison

Specimen I.D	Long. ratio ρ (%)	Peak Load P_{max} (kN)	Load and Displ. at yield		Deflection at failure Δ_{max} (mm)	Stiffness K (N/mm)
			P_y (kN)	Δ_y (mm)		
CRC-2%A-#4-0	1.0%	107.4	98.9	15.1	53.2	7660
CRC-2%A-15M-S	1.5%	139.5	138.8	13.4	47.2	9244
CRC-2%A-20M-S	2.4%	189.7	182.3	16.0	28.3	11023

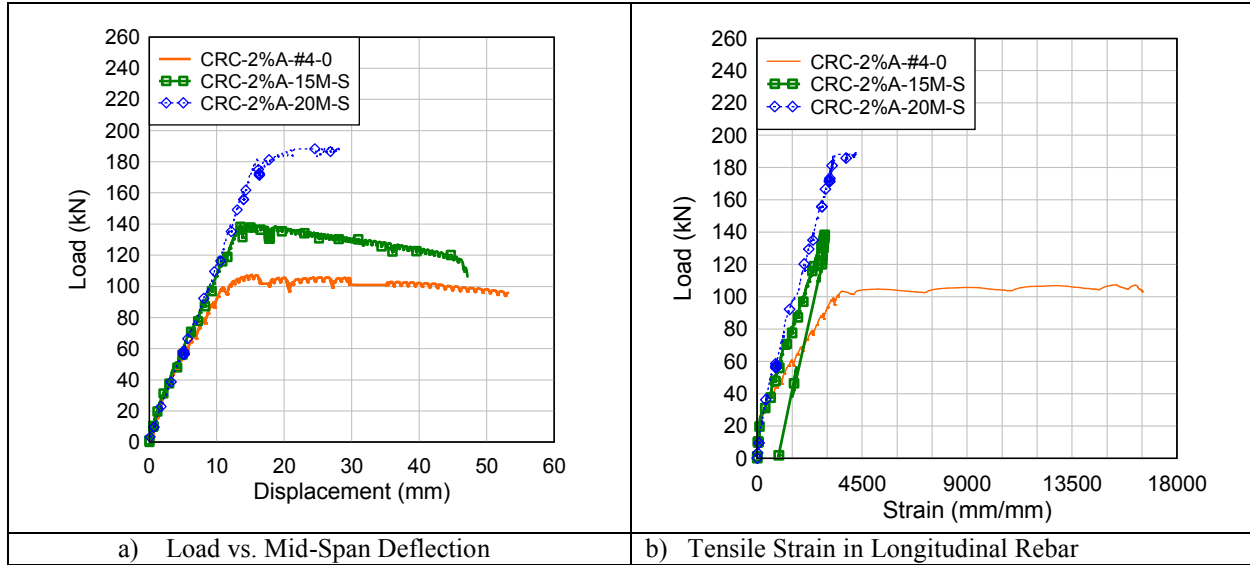


Figure 5-7 Load-Displacement and Load-Strain Curve: Effect of Reinforcement Ratio in UHPC& NSR Series

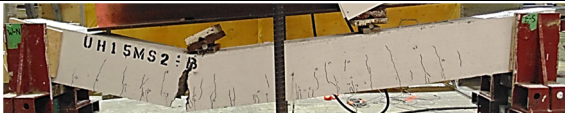
	Failure Mode	Mid-span Δ (mm)
	Rebar Rupture	53.0
CRC-2%A-#4-0		
CRC-2%A-15M-S	Rebar Rupture	47.0
		
CRC-2%A-20M-S	Rebar Rupture	28.0
		

Figure 5-8 Photographs: Effect of Reinforcement Ratio on UHPC & NSR Series

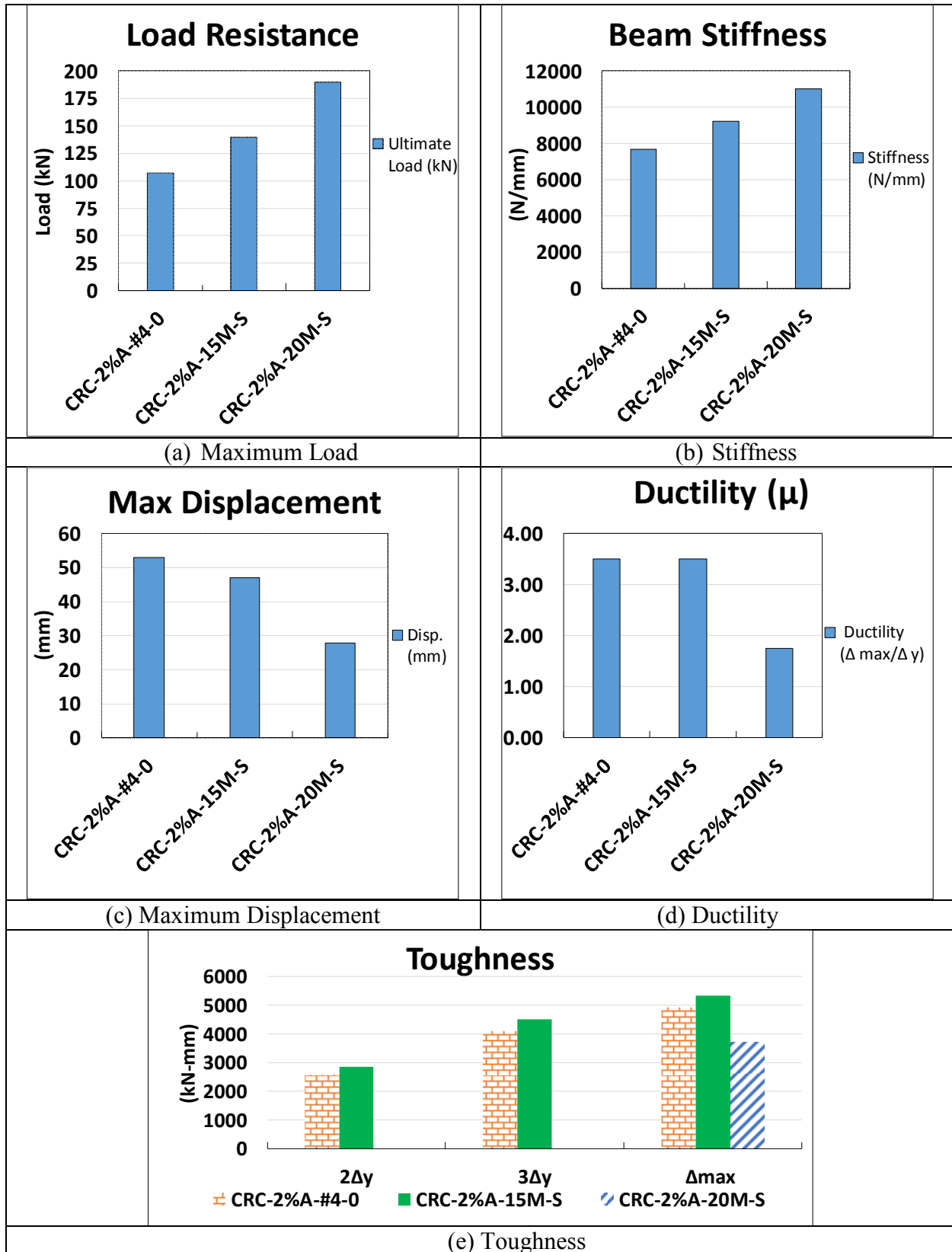


Figure 5-9 Load deflection curve parameters: Effect of Reinforcement Ratio in UHPC & NSR Series

5.3.1. Effect of Reinforcement Ratio: UHPC & HSR Series Comparison

The specimens considered from the UHPC and HSR series with varying longitudinal reinforcement ratios are: CRC-2%A-#4-HSR-S ($\rho = 1.0\%$) and CRC-2%A-#6-HSR-S ($\rho = 2.3\%$). Both specimens are designed with ultra-high performance concrete and high strength longitudinal steel, with transverse reinforcement spaced at 100 mm o/c in the shear span. A summary of results is presented in Table 5-5. In addition, Figure 5-10 through Figure 5-12 present the respective: load-displacement, load-strain, toughness, load resistance, beam stiffness, maximum displacements and ductility, complimented by photographs illustrating specimen failure modes in Figure 5-11.

The peak load resisted by CRC-2%A-#6-HSR-S ($P_{ult} = 317.6$ kN) is greater than that of CRC-2%A-#4-HSR-S ($P_{ult} = 181.8$ kN) by a factor of 175%. The shape of the load resistance curves obtained are relatively similar, with a parabolic ascension to peak load, followed shortly thereafter by specimen failure. The parabolic response can be attributed to the nature of the high strength reinforcement stress-strain relationship. There is a clear increase in stiffness with increase of reinforcement ratio; when comparing CRC-2%A-#6-HSR-S ($k = 12537$ N/mm) to CRC-2%A-#4-HSR-S ($k = 9374$ N/mm), where the specimen detailed with #6 reinforcement is 34% stiffer than the specimen equipped with #4 bars. The enhanced stiffness with respect to increasing reinforcement ratio is also observed in the load-strain relationship in Figure 5-10.

Comparing the maximum displacement resisted by each specimen, CRC-2%A-#6-HSR-S sustained a higher displacement compared to CRC-2%A-#4-HSR-S (45.6 mm vs. 40 mm, resulting in a 14% difference).

The increase in longitudinal reinforcement ratio from CRC-2%A-#4-HSR-S ($\rho = 1.0\%$) to CRC-2%A-#6-HSR-S ($\rho = 2.3\%$) proved to also have an impact on specimen energy absorption capacity (toughness). The specimen having #4 reinforcement shows a ductility ratio of 1.94 whereas the specimen detailed with #6 reinforcement achieved a ductility of 1.61. Nonetheless, overall toughness was larger for the beam with #6 bars (9275 vs. 5155 Nmm). Neither high strength specimens reached a multiple of two or three of yield displacement toughness, further confirming that high strength materials present less ductility.

Examining the failure mechanisms experienced by both specimen leads to the conclusion that CRC-2%A-#4-HSR-S failed due to longitudinal reinforcement rupture while CRC-2%A-#6-HSR-S failed in a more brittle fashion due to a major diagonal shear crack near the left-side support. It is important to note that by increasing the reinforcement ratio from $\rho = 1\%$ to 2.3% the flexural capacity is increased in the #6 specimen, which results in an increase in shear demand. Nonetheless, based on analytical predictions, CRC-2%A-#6-HSR-S was not supposed to fail in shear (due to provision of fibers and stirrups); one possible reason for this unexpected failure can be attributed to the honey-combing which was generated at the exact location of shear failure after casting of this specimen. This is further supported by the fact that its dynamic companion, failed in flexure (see Chapter 6). Figure 5-11 presents photographs of specimens at failure.

Table 5-5 UHPC & HSR Series Overview of Results: Effect of Reinforcement Ratio Comparison

Specimen I.D	Long. ratio ρ (%)	Peak Load P_{max} (kN)	Load and Displ. at yield		Deflection at failure Δ_{max} (mm)	Stiffness K (N/mm)
			P_y (kN)	Δ_y (mm)		
CRC-2%A-#4-HSR-S	1.0%	181.8	160.9	19.46	39.11	9374
CRC-2%A-#6-HSR-S	2.3%	317.6	280.0	28.4	45.6	12537

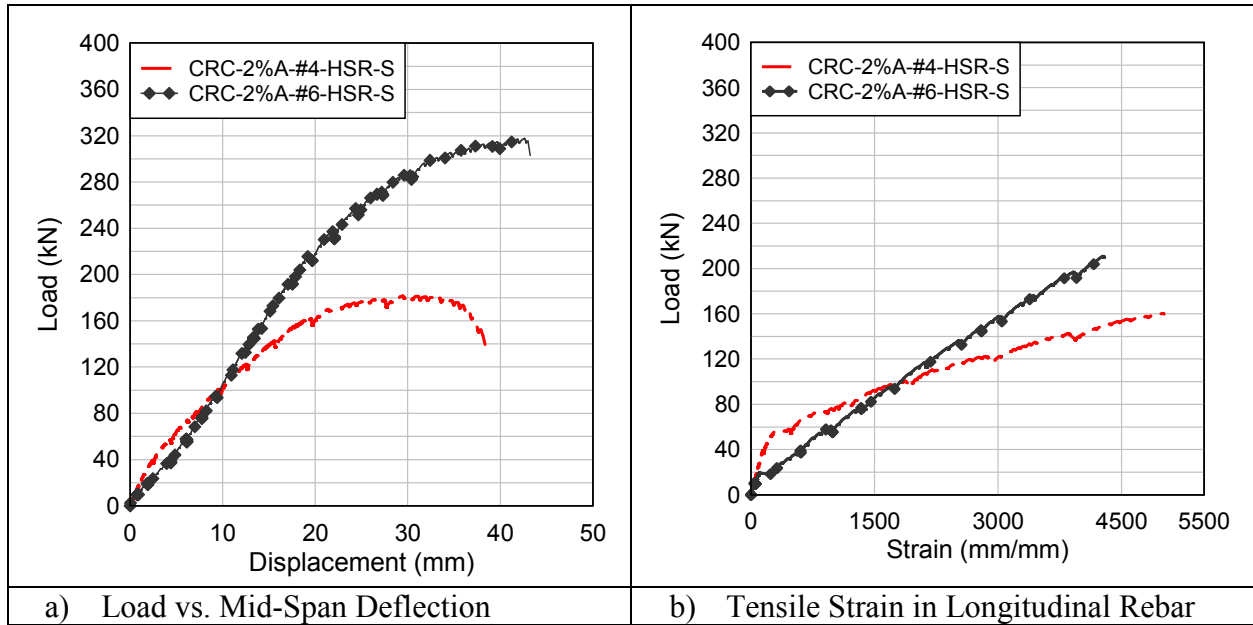


Figure 5-10 Load-Displacement and Load-Strain Curves: Effect of Reinf. Ratio in UHPC& HSR Series

	Failure Mode	Mid-span Δ (mm)
	Rebar Rupture	39.1
CRC-2%A-#4-HSR-S		
	Shear Failure	45.6
		
CRC-2%A-#6-HSR-S		

Figure 5-11 Photographs: Effect of Reinforcement Ratio on UHPC & HSR Series

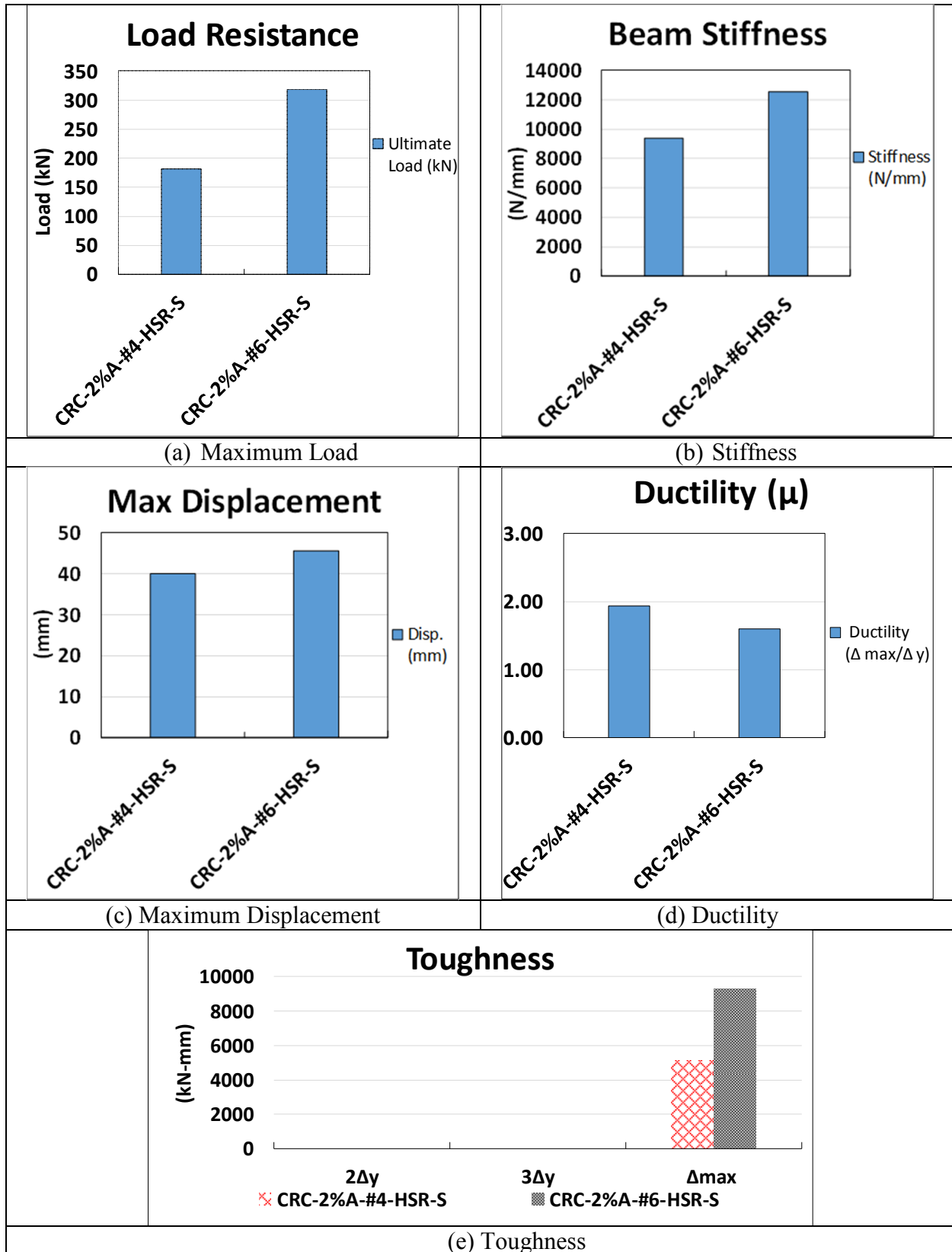


Figure 5-12 Load-Deflection Curve Parameters: Effect of Reinforcement Ratio in UHPC & HSR Series

5.4. Effect of Concrete Type

To examine the effects of concrete type, the response of four specimens is compared as follows:

- Control vs. UHPC beams with 15M bars:
 - NSC-0%-15M-S vs. CRC-2%A-15M-S
- Control vs. UHPC beams with 20M bars:
 - NSC-0%-20M-S vs. CRC-2%A-20M-S

Each set includes one beam built with UHPC and a companion with conventional concrete. The specimens are reinforced with normal-strength longitudinal bars (15M in the first set, 20M in the second set) and all beams have stirrups in the shear spans. Figure 5-13 compares the load-deflection response of the specimens. The parameters used to compare the effects of concrete type are summarized in Table 5-3 & Figure 5-15. Photos comparing failures are shown in Figure 5-14.

The peak loads resisted by the UHPC beams show significant improvements of 25% and 35% in the 15M and 20M sets when compared to their conventional concrete companions. The load resistance curve of NSC-0%-15M-S has a well-defined yield plateau, which is similar to that of CRC-2%A-15M-S. When examining stiffness, results show that the use of UHPC increases the stiffness by 26% in the 15M set and by 43% for the 20M set. Both the normal strength and UHPC 20M specimens (NSC-0%-20M-S and CRC-2%A-20M-S) reach their peak load in a more parabolic fashion. Failure of the 20M UHPC specimen occurs abruptly after rebar rupture, while the 20M normal strength specimen experiences major strength degradation as a result of concrete crushing before finally succumbing to failure.

Discussion related to the maximum displacement relates to the displacement at 15% decrease in capacity (Δ_{max}) in the case of the normal-strength concrete specimens. As portrayed in Table 5-3 and Figure 5-15 c), the use of UHPC reduces the maximum displacement by 3% and 50% in the 15M and 20M series. In direct correlation to this, the corresponding ductilities of the 15M and 20M conventional concrete beams are superior by 110% and 199%. More varied results were found when examining the effect of concrete type on toughness. As depicted in Figure 5-15 e), the use of UHPC in the 15M companion set increased the energy absorption capacity by 139%, 135% and 141% at $2\Delta_y$, $3\Delta_y$ and Δ_{max} , respectively. Conversely, the toughness of the conventional concrete 20M beam was greater than that of its UHPC companion, owing to the reduced ductility of the beam built with CRC. In fact, CRC-2%A-20M-S was not able to reach $2\Delta_y$ and $3\Delta_y$, hence, the only comparable data is at Δ_{max} for which the normal strength concrete specimen performed better by a factor of 130%. As noted before the 20M UHPC beam suffered a sudden rebar rupture, whereas the 15M UHPC beam experienced considerable bar elongation before bar rupture.

While all failure modes in this comparison were flexural in nature, the key difference in failure mechanisms between UHPC and normal strength concrete specimens is illustrated in Figure 5-14. Both the UHPC specimens failed due to longitudinal reinforcement rupture, whereas, the normal strength concrete specimens sustained concrete crushing modes of failure.

Table 5-6 Overview of Results: Effect of Concrete Type

Specimen I.D	ρ (Long.)	Peak Load P_{max} (kN)	Load and Displ. at yield		Deflection at failure (mm)		Stiffness K (N/mm)
			P_y (kN)	Δ_y (mm)	Δ_{max}	Δ_{end}	
NSC-0%-15M-S	1.5%	112.1	102.4	12.6	37.7*	48.4	7353
CRC-2%A-15M-S	1.5%	139.5	138.8	13.4	47.2	Same	9244
NSC-0%-20M-S	2.4%	140.7	131.4	16.1	26.2*	56.8	7686
CRC-2%A-20M-S	2.4%	189.7	182.3	16.0	28.3	Same	11023

* Displacement corresponding to a 15% drop in capacity

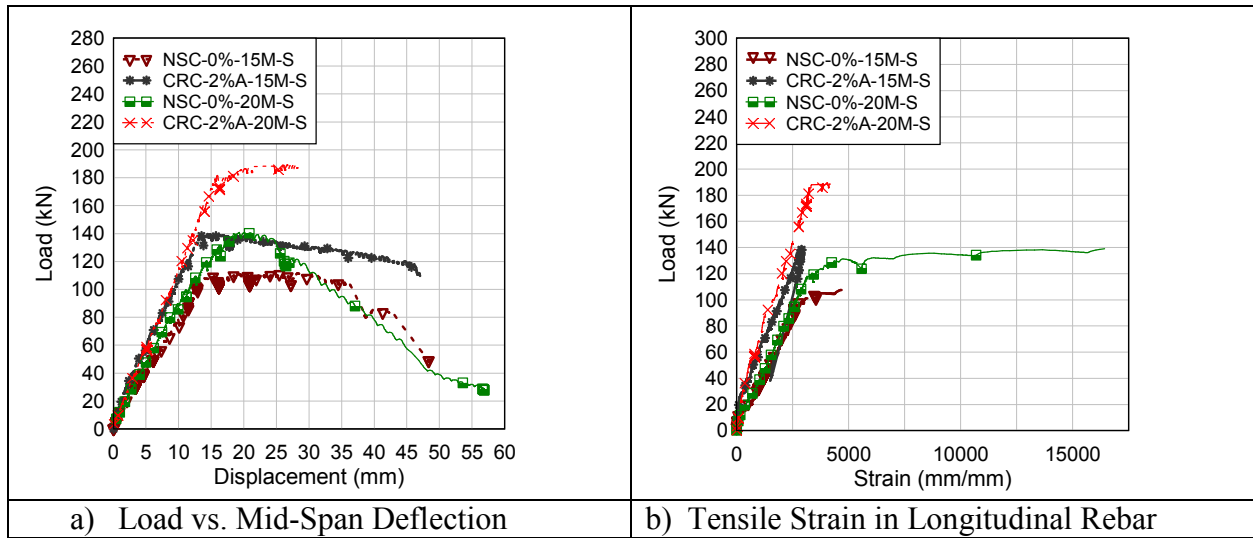


Figure 5-13 Load-Displacement and Load-Strain Curves: Effect of Concrete Type

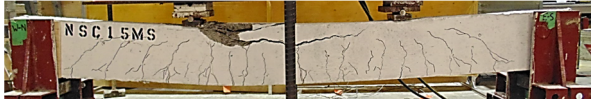
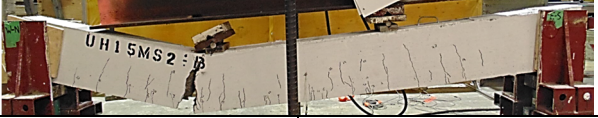
	<u>Failure Mode</u>	<u>Mid-span Δ</u>
	Concrete Crushing	43.0 mm
NSC-0%-15M-S		
CRC-2%A-15M-S	Rebar Rupture	47.0 mm
		
NSC-0%-20M-S	Concrete Crushing	56.0 mm
		
CRC-2%A-20M-S	Rebar Rupture	28.0 mm
		

Figure 5-14 Photographs: Effect of Concrete Type

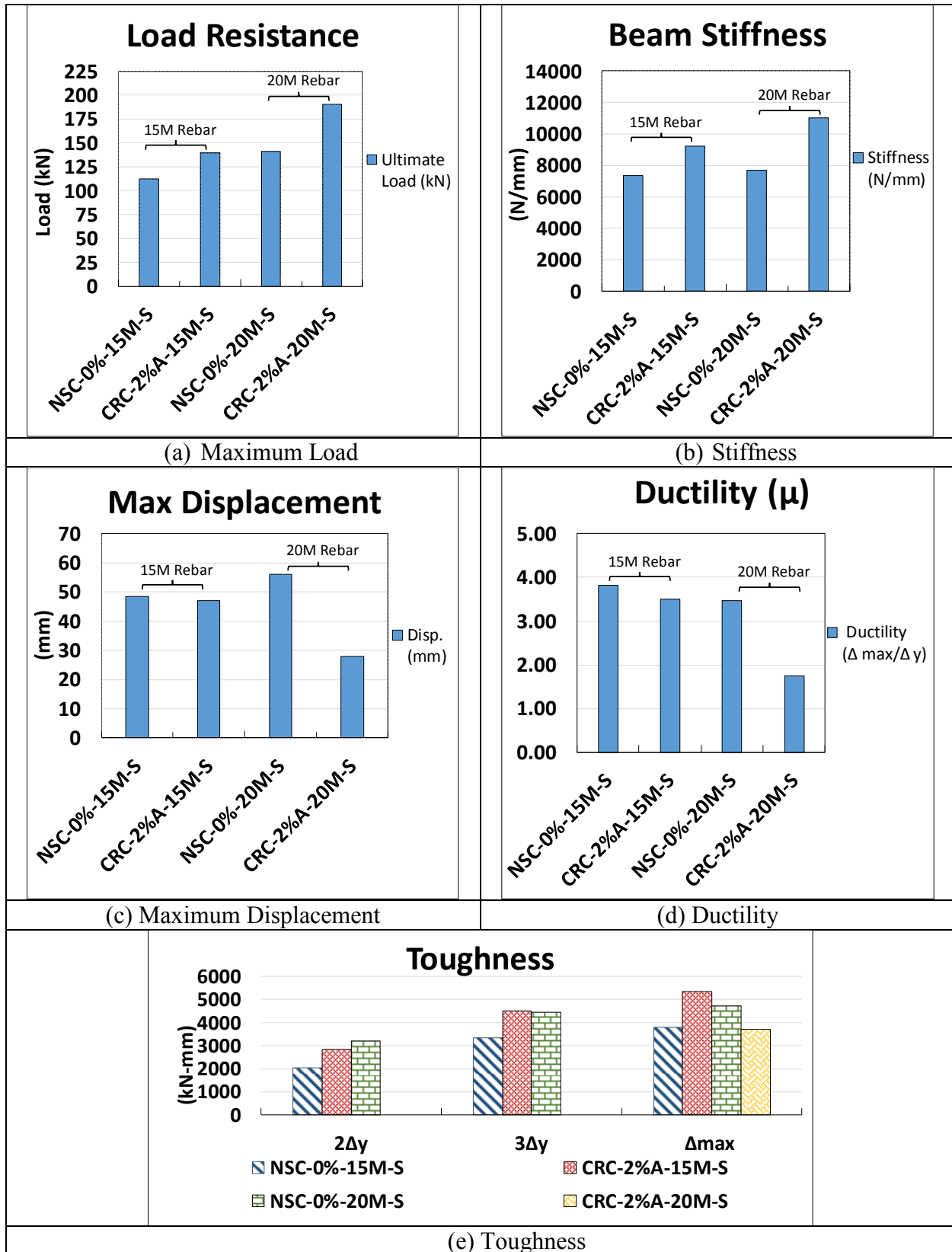


Figure 5-15 Load-Deflection Curve Parameters: Effect of Concrete Type

5.5. Effect of Longitudinal Steel Reinforcement Type

The effect of reinforcement type in ultra-high performance concrete is examined by comparing the response of companion specimens containing normal strength and high strength reinforcement:

- UHPC beams with #4 NSR vs. #4 HSR bars:
 - o CRC-2%A-#4-0 vs. CRC-2%A-#4-HSR-S
- UHPC beams with 20M NSR vs. #6 HSR bars:
 - o CRC-2%A-20M-S vs. CRC-2%A-#6-HSR-S

Both companion sets are identical in terms of design to the exception of the reinforcement type utilized. The first set of beams is designed with #4 normal strength and high strength bars, while the second set of beams are designed with 20M/#6 normal and high strength bars, respectively. Results of the aforementioned specimens are compared in Table 5-7, Figure 5-16 and Figure 5-18, complimented by photographs illustrating specimen failure modes in Figure 5-17.

The peak load resisted by the UHPC beams with #4 and #6 HSR (CRC-2%A-#4-HSR-S and CRC-2%A-#6-HSR-S) demonstrate significant improvements of 69% and 68%, when compared to the companions UHPC beams with #4 and 20M NSR (CRC-2%A-#4-0 and CRC-2%A-20M-S). Utilizing high strength reinforcement also increased the stiffness of the beams significantly, with increases of 25% and 64% recorded in the #4 and #6 companion sets. The load resistance curves for the two specimens detailed with normal strength bars have a well-defined yield plateau. Due to the nature of the high strength steel stress strain relationship, both specimens detailed with high strength bars exhibit a more parabolic ascending branch with failure shortly after reaching peak load as depicted in Figure 5-16 a.

The maximum displacements resisted by the specimens do not follow a well-defined trend. Comparing the #4 companion beams, the beam with normal strength bars experiences a maximum displacement 33% greater than the specimen with high strength bars. Conversely, for the 20/#6 companion set, the high strength steel specimen sustained a displacement 63% greater than the normal strength steel specimen, as depicted in Figure 5-16c. On the other, the ductility follows a more clear trend, with the normal strength steel specimens in the #4 and 20M/#6 sets experiencing 100% and 20% greater ductility. Further assessment of the toughness parameter indicates that only CRC-2%A-#4-0 is able to resist displacement multiples of $2\Delta_y$ and $3\Delta_y$. Comparing the toughness at Δ_{max} , it is found that the HSR specimens have greater toughness, with increases of 5% and 150% in the #4 and 20M/#6 companion sets respectively.

The failure modes of all beams were flexural in nature, to the exception of CRC-2%A-#6-HSR-S. Failure of the first three beams was governed by rebar rupture, while this failure mechanism is altered to shear in the specimen with #6 HSR. As noted earlier, failure in this beam may have been affected by construction issues (honey-combing), however the result points to the importance of ensuring that UHPC beams with high-strength reinforcement are designed with sufficient shear capacity due to the large shear demands in such beams.

Table 5-7 Overview of Results: Effect of Longitudinal Steel Reinforcement Type

Specimen I.D	Long. ratio ρ (%)	Peak Load P_{max} (kN)	Load and Displ. at yield		Deflection at failure Δ_{max} (mm)	Stiffness (N/mm)
			P_y (kN)	Δ_y (mm)		
CRC-2%A-#4-0	1.0%	107.4	98.9	15.1	53.2	7353
CRC-2%A-#4-HSR-S	1.0%	181.8	160.9	19.5	40.0	9374
CRC-2%A-20M-S	2.4%	189.7	182.3	16.0	28.3	7686
CRC-2%A-#6-HSR-S	2.3%	317.6	280.0	28.4	45.6	12537

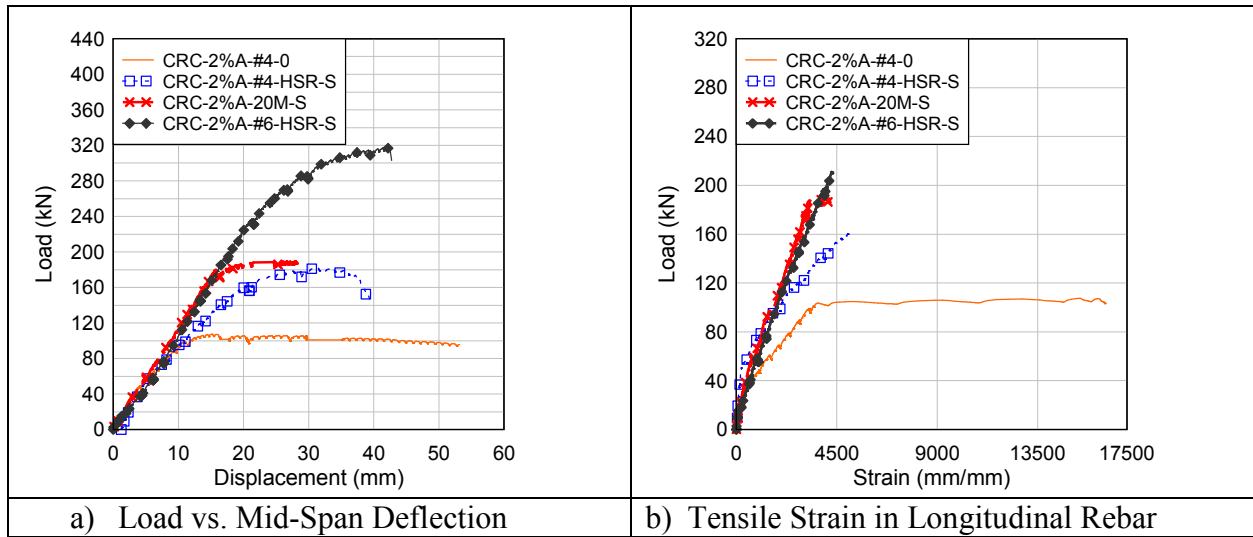


Figure 5-16 Load-Displacement and Load-Strain Curves: Effect of Longitudinal Steel Reinforcement Type

	Failure Mode	Mid-span Δ (mm)
	Rebar Rupture	53.0
CRC-2%A-#4-0		
CRC-2%A-#4-HSR-S	Rebar Rupture	39.6
		
CRC-2%A-20M-S	Rebar Rupture	28.0
		
CRC-2%A-#6-HSR-S	Shear Failure	45.6
		

Figure 5-17 Photographs: Effect of Longitudinal Steel Reinforcement Type

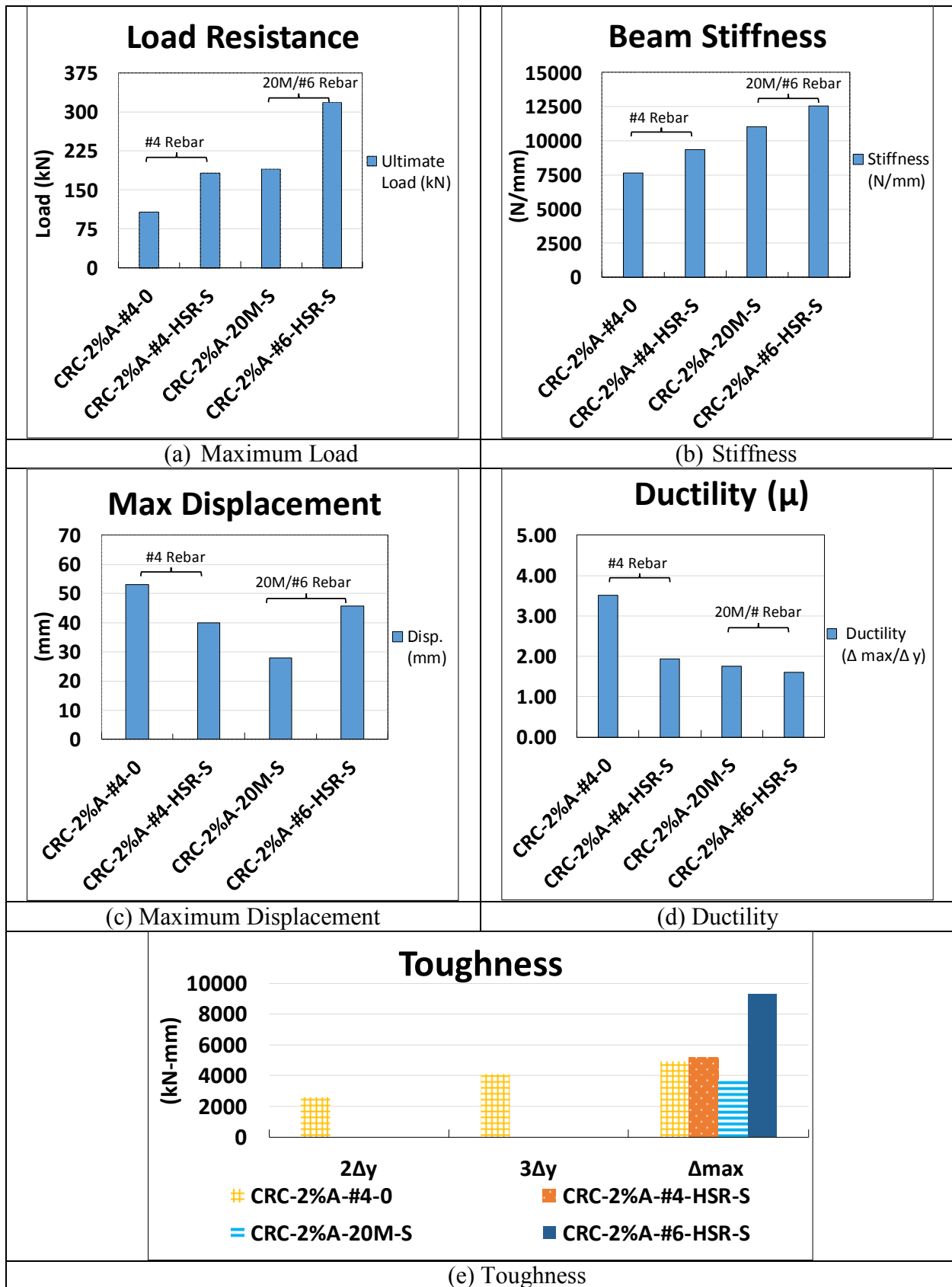


Figure 5-18 Load-Deflection Curve Parameters: Effect of Longitudinal Steel Reinforcement Type

Chapter 6. Experimental Results - Dynamic Tests

6.1. Chapter Overview

Chapter 6 presents the experimental results for the 14 beams tested under dynamic (blast) loading. Section 6.2 presents the tabulated results for each specimen at each incremental blast load including: the reflected pressure and impulse of each blast, the corresponding positive pressure duration, maximum and residual mid-span displacement as well as the corresponding support rotation. Section 6.3 provides the detailed results for each individual specimen, starting with a description of results after each shot, followed by a summary table presenting the key findings for each blast (maximum and residual mid-span displacements, support rotation, peak and residual steel reinforcement strains and comments regarding progression of damage). Additionally, each specimen's respective pressure and impulse time histories and displacement time histories are presented in the form of graphs. Sequential photographs showing progressive damage and failure mechanisms are summarized in at the end of each subsection.

6.2. Summary of Experimental Results

Tables 6-1 to 6-3 present the experimental results for all 14 dynamically loaded beam specimens. The table presents shockwaves properties: length of driver utilized (L_d), the driver pressure (P_d), the recorded reflected pressure on the load transfer device (P_r), the resulting impulse (I_r), positive shockwave duration (t_p), the maximum (δ_{max}) and residual (δ_{res}) mid-span displacements as well as the corresponding maximum support rotation (θ_{max}). It is to be noted that the displacement values presented are not cumulative.

Table 6-1 Dynamic Testing Data Summary (1/3)

Beams	Shockwave Properties						Displacements		θ_{max}
	Blast	L_d	P_d	P_r	I_r	t_p	δ_{max}	δ_{res}	
	ID	#	(mm)	(kPa)	(kPa-ms)	(ms)	(mm)	(mm)	degree
NSC-0%-#4-0		1	2745	117.3	21.1	240.1	22.5	11.8	0.6
		2	2745	207.0	35.4	364.7	20.7	27.3	1.4
		3	2745	345.0	46.9	653.7	21.2	67.7	3.4
NSC-0%-15M-0		1	2745	117.3	-	-	-	-	-
		2	2745	207.0	-	-	-	-	-
		3	2745	345.0	47.1	554.0	21.2	59.8	3.0
NSC-0%-15M-S		1	2745	117.3	19.8	232.5	24.1	10.0	0.5
		2	2745	207.0	35.3	354.0	22.0	19.3	0.9
		3	2745	345.0	51.5	486.2	21.6	42.0	2.1
NSC-0%-20M-S		1	2745	117.3	24.9	224.3	22.9	10.7	0.5
		2	2745	207.0	36.3	371.3	20.7	17.3	0.8
		3	2745	345.0	46.7	524.3	21.2	29.5	1.5
		4	2745	483.0	71.8	652.1	19.4	108.6	5.5

:- implies data acquisition malfunction.

Table 6-2 Dynamic Testing Data Summary (2/3)

Beams	Shockwave Properties						Displacements		$\theta_{\max}$
ID	Blast	L _d	P _d	P _r	I _r	t _p	$\delta_{\max}$	δ_{res}	
	#	(mm)	(kPa)	(kPa)	(kPa-ms)	(ms)	(mm)	(mm)	degree
CRC-2%A-#4-0	1	2745	117.3	28.2	244.8	22.9	5.7	1.5	0.2
	2	2745	207.0	35.6	377.5	21.2	12.6	3.3	0.6
	3	2745	345.0	53.7	534.9	22.5	26.3	12.3	1.3
	4	2745	483.0	63.4	683.3	20.7	118.0	72.2	6.0
CRC-2%A-20M-0	1	2745	117.3	20.7	274.5	21.6	7.7	2.3	0.4
	2	2745	207.0	36.0	365.1	21.2	11.4	1.0	0.5
	3	2745	345.0	47.7	547.7	21.6	21.4	2.6	1.1
	4	2745	483.0	61.4	710.4	22.9	34.0	14.5	1.7
	5	2745	621.0	83.1	859.01	20.7	56.6	28.9	2.9
CRC-2%A-15M-S	1	2745	117.3	23.6	230.7	22.9	10.2	4.2	0.9
	2	2745	207.0	34.5	346.6	21.6	12.8	0.4	0.6
	3	2745	345.0	47.1	530.7	22.0	22.0	6.2	1.1
	4	2745	483.0	60.4	673.4	21.2	41.9	19.0	2.1
CRC-2%A-20M-S	1	2745	117.3	23.5	234.2	23.8	4.5	1.3	0.2
	2	2745	207.0	36.6	367.6	20.7	10.7	1.3	0.5
	3	2745	345.0	54.9	549.8	21.1	16.8	2.4	0.8
	4	2745	483.0	65.6	708.1	22.5	25.8	7.7	1.3
	5	2745	621.0	77.2	831.8	23.4	48.4	25.3	2.4
CRC-1%B-15M-0	1	2745	117.3	21.8	236.3	22.5	8.0	1.8	0.4
	2	2745	207.0	34.5	366.4	20.3	14.2	0.4	0.7
	3	2745	345.0	48.8	509.5	19.8	21.0	8.3	1.0
	4	2745	483.0	61.6	668.5	21.2	40.8	18.5	2.1
	5	2745	621.0	75.5	824.3	22.0	247.1	-	12.4
CRC-2%B-15M-0	1	2745	117.3	23.1	229.3	21.6	8.9	2.1	0.4
	2	2745	207.0	-	-	-	18.5	1.4	0.9
	3	2745	345.0	53.0	566.7	20.3	29.5	14.4	1.5
	4	2745	483.0	68.5	695.7	20.7	62.0	38.3	3.1
	5	2745	621.0	68.8	718.0	21.6	-	-	-
CRC-3%B-15M-0	1	2745	117.3	23.4	223.6	22.9	6.2	1.3	0.3
	2	2745	207.0	37.1	372.6	19.4	13.2	1.3	0.6
	3	2745	345.0	55.4	530.7	20.7	20.0	4.9	1.0
	4	2745	483.0	70.2	706.7	21.2	45.6	23.8	2.3

:- implies data acquisition malfunction.

Table 6-3 Dynamic Testing Data Summary (3/3)

Beams	Shockwave Properties						Displacements		$\theta_{\max}$
ID	Blast	L _d	P _d	P _r	I _r	t _p	$\delta_{\max}$	δ_{res}	
	#	(mm)	(kPa)	(kPa)	(kPa-ms)	(ms)	(mm)	(mm)	degree
CRC-2%A-#4-HSR-S	1	2745	117.3	22.2	226.5	23.85	5.0	0.9	0.2
	2	2745	207.0	36.5	413.4	21.2	14.0	2.1	0.7
	3	2745	345.0	51.3	537.1	20.76	21.9	0.4	1.1
	4	2745	483.0	65.3	695.3	22.08	162.7	98.	8.2
CRC-2%A-#5-HSR-S	1	2745	117.3	20.0	215.1	27.39	6.67	2.2	0.3
	2	2745	207.0	30.0	330.5	21.2	11.7	0.5	0.6
	3	2745	345.0	47.5	532.8	22.38	19.1	0.3	0.9
	4	2745	483.0	60.4	698.2	21.64	28.1	1.5	1.4
	5	2745	621.0	73.2	863.9	24.73	40.0	6.3	2.0
	6	2745	759.0	87.7	1007.7	22.53	-	-	-
CRC-2%A-#6-HSR-S	1	2745	117.3	22.2	246.2	23.85	6.0	1.2	0.3
	2	2745	207.0	35.6	361.4	20.32	11.0	0.4	0.5
	3	2745	345.0	-	-	-	17.0	0.5	0.8
	4	2745	483.0	62.2	720.4	22.08	25.0	1.2	1.2
	5	2745	621.0	81.9	906.0	22.52	33.7	1.2	1.7
	6	2745	759.0	94.2	1085.1	23.41	49.8	16.6	2.5
	7	2745	828.4	97.6	1078.4	23.41	98.4	67.2	5.0

:- implies data acquisition malfunction.

6.3. Control Series

6.3.1. Beam with NSC and #4 NSR (NSC-0%-#4-0)

The first beam specimen tested was NSC-0%-#4-0; it is part of the normal strength concrete Control series. It is detailed with #4 normal strength longitudinal reinforcement without any shear reinforcement. The imposed shockwave properties and resulting displacements can be found in Table 6-1. The pressure-impulse and displacement time histories for this beam are presented in Figure 6-1, complimented by sequential blast photographs in Figure 6-2. Data corresponding to the following blast test descriptions can be found in Table 6-4.

Blast 1 (17 psi) was meant to test the specimen within the elastic range. As shown in Table 6-4, the strains in the longitudinal tension reinforcement remained elastic. The induced shockwave resulted in a maximum support rotation of 0.61°. A total of 9 hairline cracks were recorded on the tension side within the flexural zone.

Blast 2 (30 psi) strained the longitudinal reinforcement beyond its yield stress. The resulting maximum and residual displacements were of 27.39 mm and 7.21 mm, respectively, with support rotation of 1.41°. A total of 9 cracks having an average width of 0.23 mm were recorded in the constant moment region.

Blast 3 (50 psi) caused the beam specimen to fail. Failure was associated with concrete crushing and the formation of a splitting tensile cracks at mid-span. The maximum and residual strains were not recorded at this blast as the strain gauges were damaged. The beam suffered a maximum and residual displacement of 67.72 mm and 26.11 mm respectively. As depicted in Figure 6-2 c), the beam experienced a pronounced rotation of 3.47° about supports. The mean crack width is of 1.7 mm in the flexural zone while the largest crack reached 4 mm.

Table 6-4 NSC-0%-#4-0 Acquired Blast Data

Blast #	δ_{max} (mm)	Support Rotation (°)	δ_{res} (mm)	Peak Strain ($\mu\text{mm/mm}$)	Residual Strain ($\mu\text{mm/mm}$)	Comments
1	11.84	0.61	3.48	2077	410	- Longitudinal reinforcement elastic - 9 shallow and equally spaced hairline cracks on the tension side
2	27.39	1.41	7.21	9153	5492	- Yielding of longitudinal reinforcement occurs - Cracks propagate deeper into the section, with the development of flexural shear cracks near supports - Average crack width is 0.23 mm
3	67.72	3.47	26.22	N/A	N/A	- Specimen failure with concrete crushing and splitting at mid-span - Average crack width of 1.7 mm

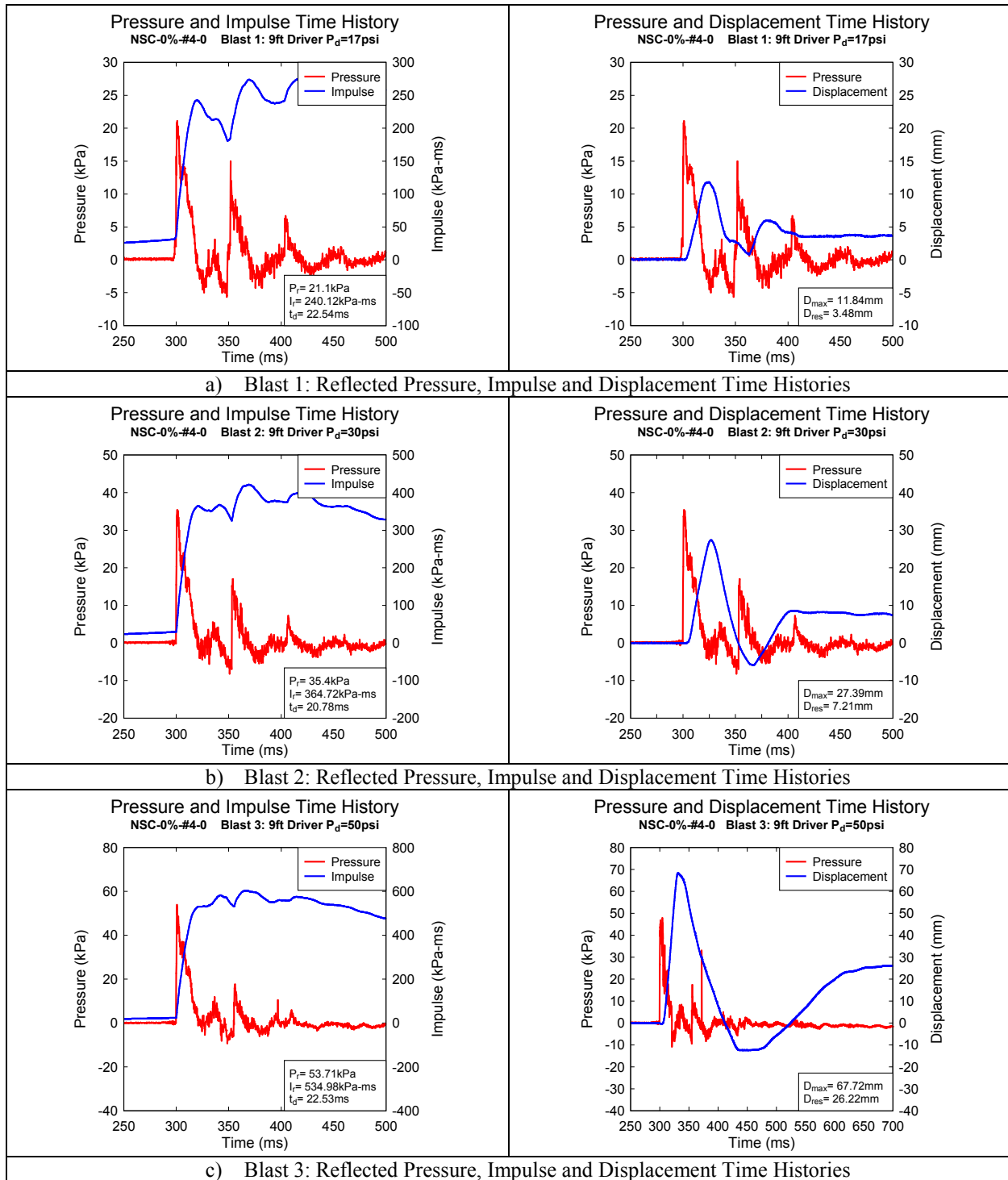


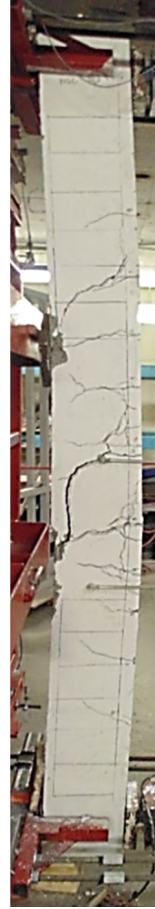
Figure 6-1 NSC-0%-#4-0, Recorded Reflected Pressure, Impulse and Displacements



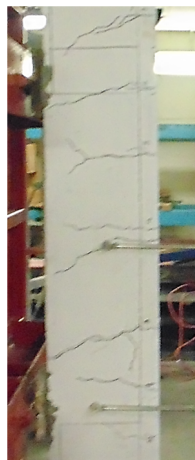
a) Blast 1
(17 psi)



b) Blast 2
(30 psi)



c) Blast 3
(50 psi)



d) Mid-span cracking
(Blast 2)



e) Mid-span cracking
(south face)
(Blast 3)



f) Mid-span cracking
(north face)
(Blast 3)

Figure 6-2 NSC-0%-#4-0, Photographs at the End of Blasts 1-3

6.3.2. Beam with NSC and 15M NSR (NSC-0%-15M-0)

This normal strength concrete beam specimen is the second beam in the Control series. This beam is detailed with 15M normal strength longitudinal bars without any shear reinforcement. It is noted that the data acquisition system experienced a malfunction and no blast data was recorded at Blasts 1 and 2 (17 psi and 30 psi). Blast 3 pressure-impulse and displacement time histories are presented in Figure 6-3, complimented by post-blast photographs in Figure 6-3. Data and comments for this blast can be found in Table 6-5.

While no data is available, the specimen showed limited damage after Blasts 1 and 2. As depicted in Figure 6-3, the beam experienced a shear failure between the top support and upper point load during Blast 3 (50 psi). A concrete segment approximately 130 mm long, 40 mm deep and 125 mm wide spalled off at the very top of the beam near the support. In consequence of the given dynamic loading the beam suffered a maximum and residual displacement of 59.85 mm and 29.75 mm respectively, with a support rotation of 3.1° . Strains recorded during this test indicate the reinforcement entered the plastic range prior to failure.

Table 6-5 NSC-0%-15M-0 Acquired Blast Data

Blast #	δ_{max} (mm)	Support Rotation ($^\circ$)	δ_{res} (mm)	Peak Strain ($\mu\text{mm/mm}$)	Residual Strain ($\mu\text{mm/mm}$)	Comments
3	59.85	3.07	29.75	15410	8677	- Beam specimen experienced a severe shear failure between the top support and upper point load - The average crack width not considering the shear failure is measured at 0.14 mm - Minor concrete crushing occurred at both upper and lower point load locations on the beam compression side

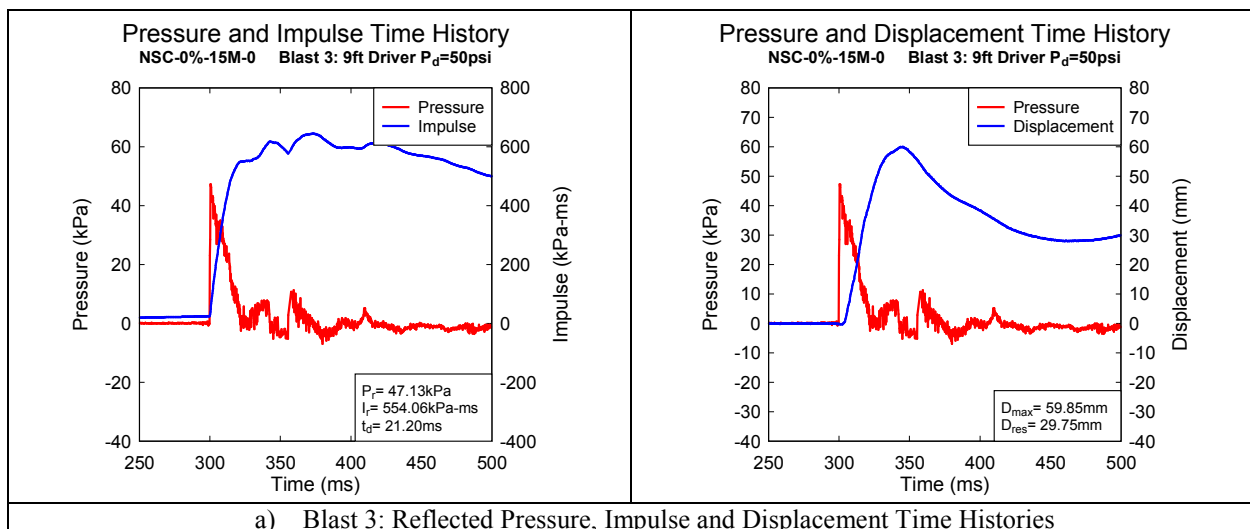
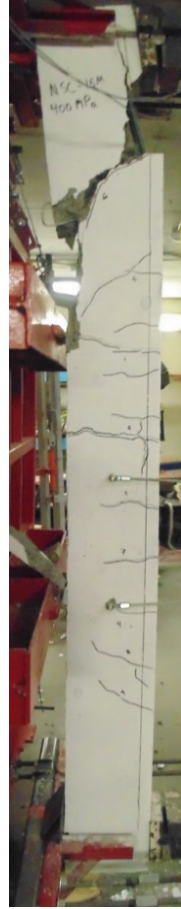


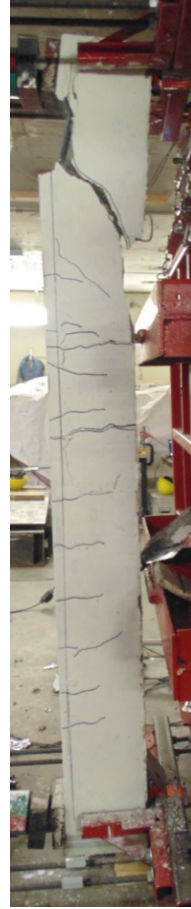
Figure 6-3 NSC-0%-15M-0, Recorded Reflected Pressure, Impulse and Displacement for Blast 3



a) Blast 1
(17 psi)



b) Blast 3
(50 psi)



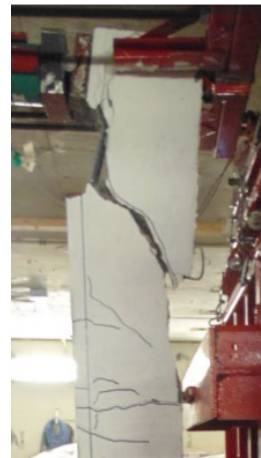
c) Blast 3
(50 psi)



d) Shear Failure
(left side)
(Blast 3)



e) Mid-span cracking
(tension side)
(Blast 3)



f) Mid-span cracking
(right side)
(Blast 3)

Figure 6-4 NSC-0%-15M-0, Photographs at the End of Blasts 1-3

6.3.3. Beam with NSC and 15M NSR (NSC-0%-15M-S)

The third normal strength concrete beam specimen is designated as NSC-0%-15M-S. This control specimen is detailed with 15M bars, but unlike the previous beam stirrups are provided in the shear spans, at 100 mm o/c. The imposed shockwave properties and resulting displacements can be found in Table 6-1. A complete description of each pressure-impulse and displacement time histories is presented in Figure 6-5, complimented by sequential blast photographs in Figure 6-6. Data and comments for each blast can be found in Table 6-6.

Blast 1 (17 psi) was meant to test the elastic response of this specimen, as confirmed by the strain readings in Table 6-1. The induced shockwave resulted in a support rotation of 0.52° with limited residual displacement. A total of six hairline cracks were recorded within the flexural zone.

Blast 2 (30 psi) strained the longitudinal reinforcement beyond its yielding stress. The resulting maximum and residual displacements were of 19.32 mm and 2.01 mm, with the support rotation equal to 1° . Flexural cracks between both point loads became more prominent, with a splitting crack across the section depth at mid-span.

Blast 3 (50 psi) caused failure of the beam specimen. The strain gauges on the tension reinforcement recorded large maximum strains before failing. In consequence of the given dynamic loading the beam suffered a maximum and residual displacement of 42.1 mm and 30.6 mm. As depicted in Table 6-6, the beam experienced a support rotation of 2.16° before failing due to concrete crushing. The maximum crack width is recorded as 0.4mm in the flexural zone.

Table 6-6 NSC-0%-15M-S Acquired Blast Data

Blast #	δ_{max} (mm)	Support Rotation ($^\circ$)	δ_{res} (mm)	Peak Strain ($\mu\text{mm/mm}$)	Residual Strain ($\mu\text{mm/mm}$)	Comments
1	10.04	0.52	0.69	1992	17	- Elastic behavior - Formation of 6 visible hairline cracks
2	19.32	0.99	2.01	3051	229	- Specimen yields - Propagation of existing cracks and formation of 6 new cracks in the flexural region - Average crack width is 0.15 mm
3	42.05	2.16	30.6	25205	N/A	- Concrete crushing occurs - Large concrete compression block is broken off the specimen at mid-span - Plastic hinge develops with post-blast cracks number 2 and 4 reaching 0.4 mm

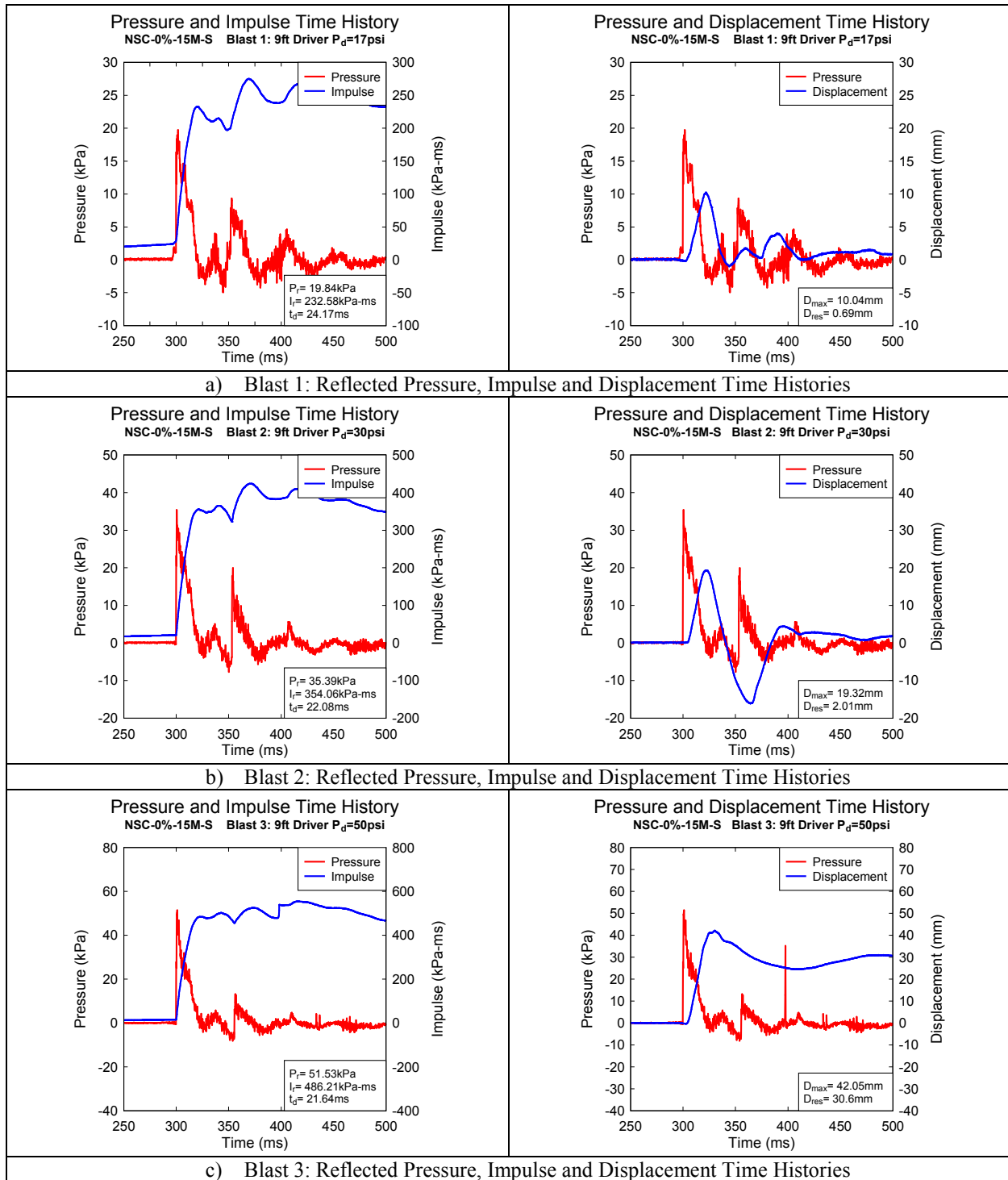
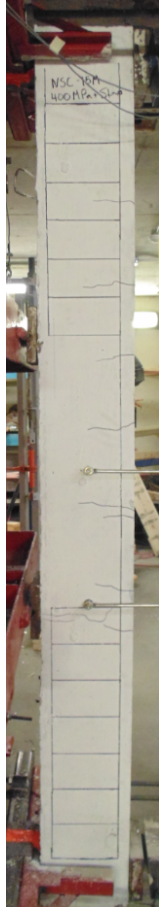
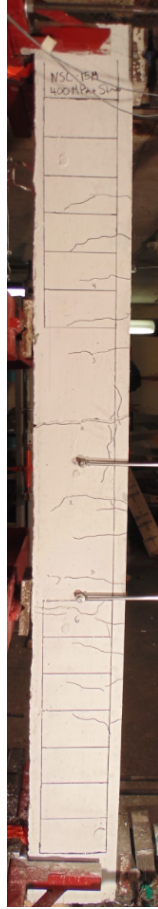


Figure 6-5 NSC-0%-15M-S, Recorded Reflected Pressure, Impulse and Displacements



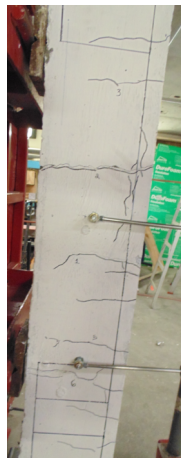
a) Blast 1
(17 psi)



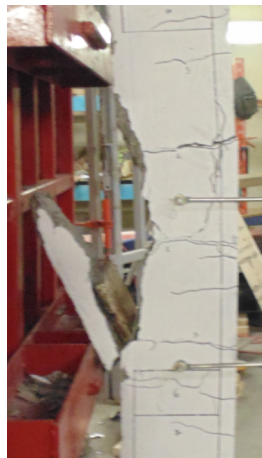
b) Blast 2
(30 psi)



c) Blast 3
(50 psi)



d) Mid-span cracking
(Blast 2)



e) Mid-span concrete
crushing (right side)
(Blast 3)



f) Mid-span concrete
crushing (left side)
(Blast 3)

Figure 6-6 NSC-0%-15M-S, Photographs at the End of Blasts 1-3

6.3.4. Beam with NSC and 20M NSR (NSC-0%-20M-S)

The last beam specimen from the normal strength concrete Control Series is referred to as NSC-0%-20M-S. It is detailed with 20M normal strength longitudinal bars, with shear reinforcement in the shear spans at 100 mm o/c. The imposed shockwave properties and resulting displacements can be found in Table 6-1. A complete description of each pressure-impulse and displacement time histories is presented in Figure 6-7, complimented by sequential blast photographs in Figure 6-8. Data corresponding to the blast tests can be found in Table 6-7.

Blast 1 (17 psi) and Blast 2 (30 psi) tested the elastic response of this specimen. The strain gauges failed to record data however the low residual displacements would confirm this. Flexural cracks remained hairline after Blast 1, with the formation of further flexural cracks and diagonal shear cracks after Blast 2.

Blast 3 (50 psi) resulted in maximum and residual displacement of 29.54 mm and 9.60 mm respectfully. As depicted in Figure 6-8, and confirmed looking at the support rotation of 1.52° the beam is approaching the heavily damaged state.

Blast 4 (70 psi) resulted in the failure of the specimen due to concrete crushing. The recorded maximum and residual displacement are 108.7 mm and 51.3 mm. The beam can be described as having suffered a near hazardous failure when considering the support rotation of (5.56°). As a result of blast 4, significant flexural shear cracks developed within the shear spans, the maximum post-blast crack width is recorded at 1.7 mm in the constant moment region.

Table 6-7 NSC-0%-20M-S Acquired Blast Data

Blast #	δ_{max} (mm)	Support Rotation ($^\circ$)	δ_{res} (mm)	Peak Strain ($\mu\text{mm/mm}$)	Residual Strain ($\mu\text{mm/mm}$)	Comments
1	10.77	0.55	2.36	N/A*	N/A*	- Two hairline cracks are recorded in the flexural region
2	17.34	0.89	0.35	N/A*	N/A*	- Existing cracks propagate while 4 new cracks including two flexural shear cracks in shear span - Average crack width is 0.095 mm
3	29.54	1.52	9.6	N/A*	N/A*	- Damage becomes prominent - A total of 11 flexural and flexural shear cracks reach an average crack width of 0.3 mm - Splitting crack at level of tension steel
4	108.69	5.56	51.3	N/A*	N/A*	- Concrete crushing occurs - Large concrete compression block is broken off the specimen at mid-span - Largest flexural crack is 1.7 mm

* Strain gauge damaged during first shot, no strain data recorded

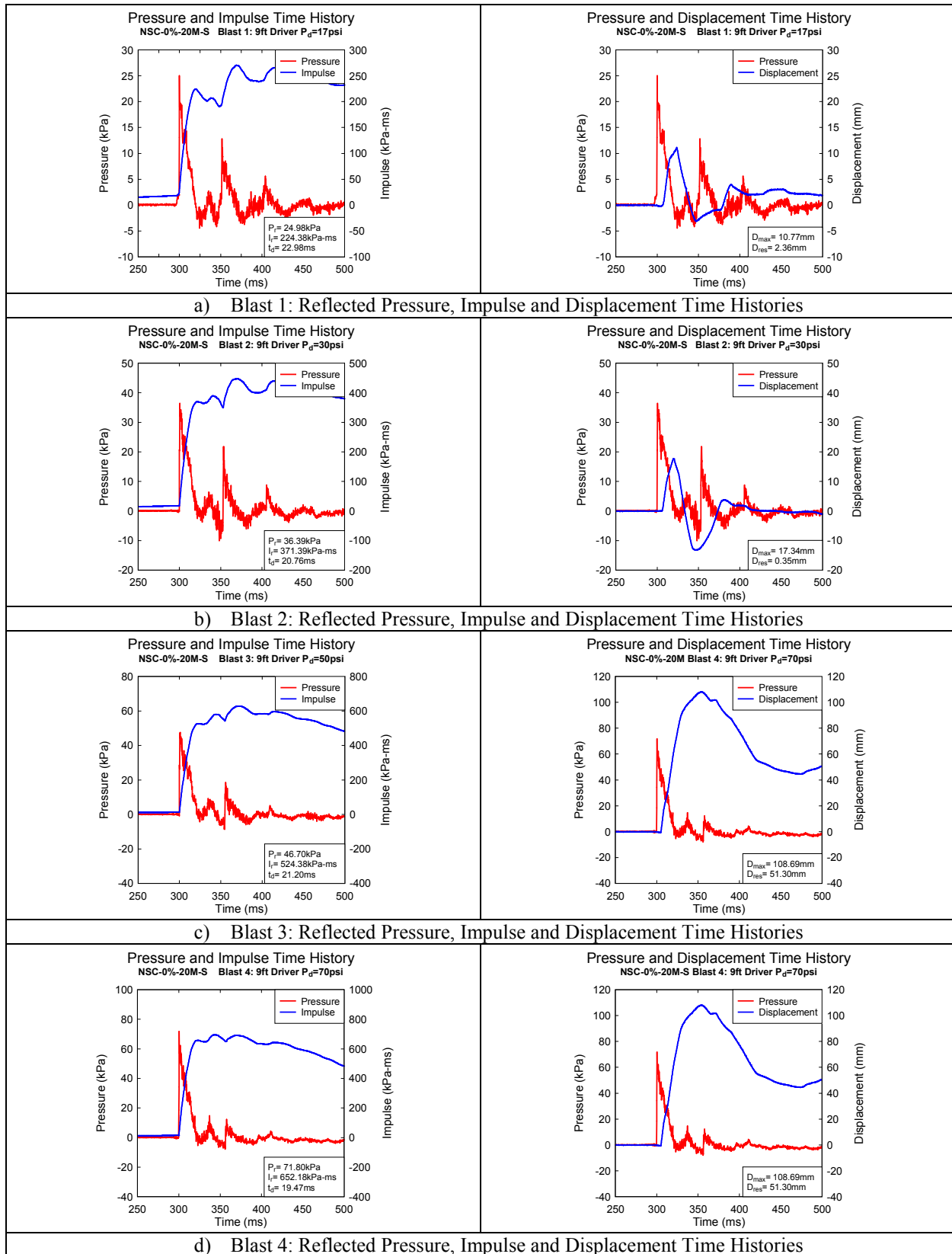


Figure 6-7 NSC-0%-20M-S, Recorded Reflected Pressure, Impulse and Displacement for Blast 1 to 4



a) Blast 1
(17 psi)



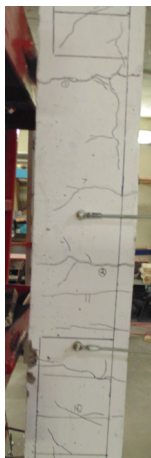
b) Blast 2
(30 psi)



c) Blast 3
(50 psi)



d) Blast 4
(70 psi)



e) Mid-span cracking
(Blast 3)



f) Mid-span concrete
Crushing (South face)
(Blast 4)



g) Mid-span cracking
(East face)
(Blast 4)

Figure 6-8 NSC-0%-20M-S, Photographs at the End of Blasts 1-4

6.4. UHPC & NSR Series (1 of 2)

6.4.1. Beam with UHPC and #4 NSR (CRC-2%A-#4-0)

CRC-2%A-#4-0 is part of the UHPC & NSR series. It is built with ultra-high performance concrete and detailed with #4 normal strength longitudinal reinforcement, without any stirrups. The CRC mix in this specimen has 2% of fiber type A. The imposed shockwave properties and resulting displacements can be found in Table 6-1. A complete description of each pressure-impulse and displacement time histories is presented in Figure 6-9, complimented by sequential blast photographs in Figure 6-10. Data and comments can be found in Table 6-8.

Blast 1 (17 psi) kept the longitudinal tension reinforcement within the elastic range. The induced shockwave resulted in a maximum displacement of 5.7 mm and a residual displacement of 1.51 mm. A total of two hairline cracks were recorded on the tension side within the flexural zone.

Blast 2 (30 psi) strained the longitudinal reinforcement beyond its yielding stress. The resulting maximum and residual displacements were of 12.65 mm and 0.65 mm. Support rotation is still below 1°. A total of 7 hairline cracks were recorded in the constant moment region.

Blast 3 (50 psi) resulted in maximum and residual displacements of 26.36 mm and 1.35 mm. Support rotation exceeds 1°. As depicted in Figure 6-10, the beam experienced some pronounced curvature in the flexural region with cracks propagating through the beam depth at both point load locations. The mean crack width is of 0.5 mm in the flexural zone. It is important to note that visible fiber pullout is initiated across some cracks.

Blast 4 (70 psi) was sufficient to cause the longitudinal #4 NSR bars to rupture and result in the complete collapse of the beam specimen. Maximum and residual displacements were recorded at 118.05 mm and 6.04 mm. Despite the severe failure, minimal secondary fragments were visible to the human eye as the fibers held the concrete together outside of the failure zone.

Table 6-8 CRC-2%A-#4-NSR-0 Acquired Blast Data

Blast #	δ_{max} (mm)	Support Rotation (°)	δ_{res} (mm)	Peak Strain ($\mu\text{mm/mm}$)	Residual Strain ($\mu\text{mm/mm}$)	Comments
1	5.7	0.29	1.51	1192	570	- Elastic response - Hairline cracks appear
2	12.65	0.65	3.37	3147	1341	- Reinforcement Yielding - Average flexural crack width: 0.03 mm
3	26.36	1.35	12.36	4112	1457	- Specimen experiences major cracks at point load locations - Average flexural crack width is 0.5 mm
4	118.05	6.04	72.26	N/A	N/A	- Rebar rupture in tensile region - Excessive cracking with widths of 39.5 and 10.46 mm at cracks 2 and 6 - Beam is completely severed

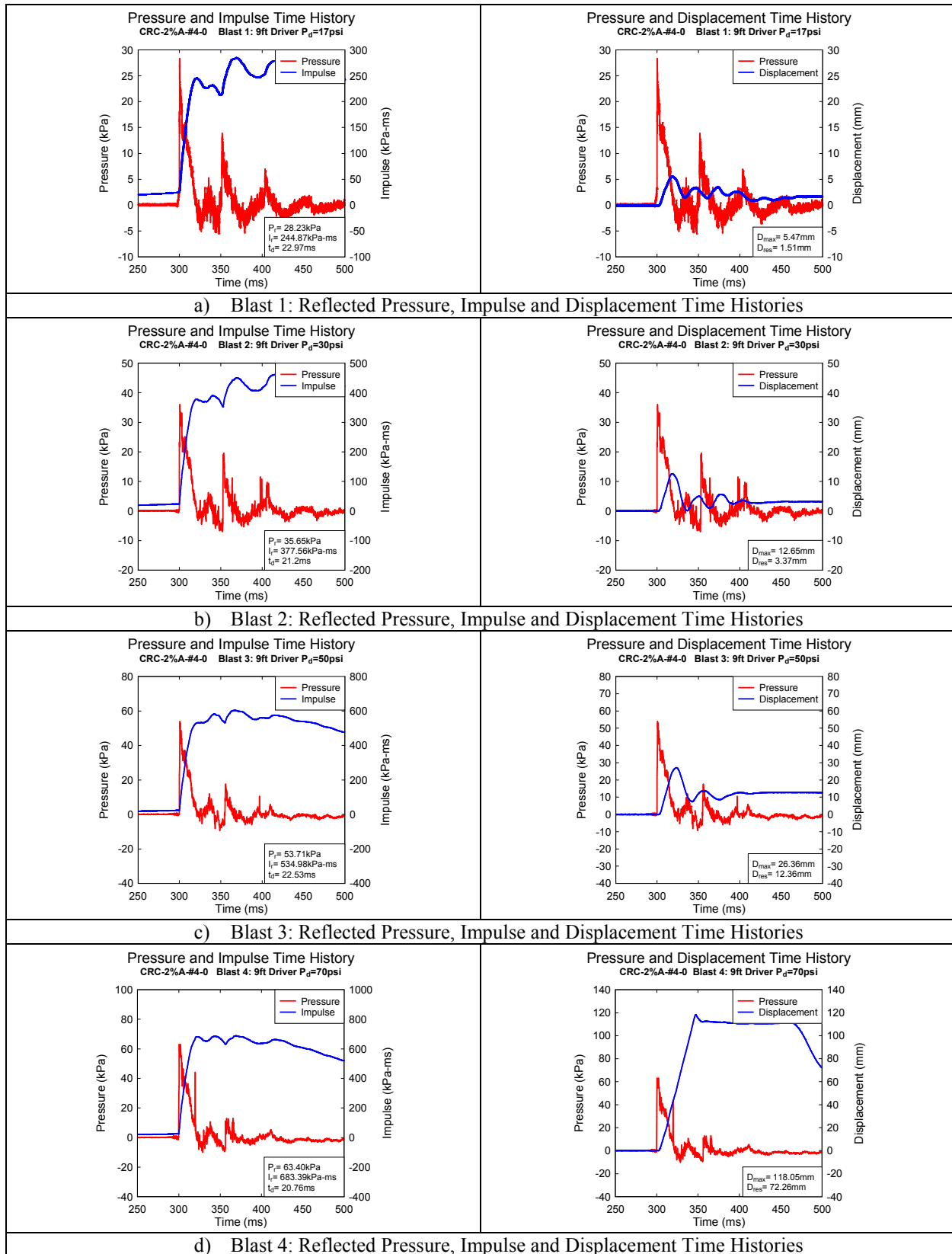


Figure 6-9 CRC-2%A-#4-0, Recorded Reflected Pressure, Impulse and Displacement for Blast 1 to 4



a) Blast 1
(17 psi)



b) Blast 2
(30 psi)



c) Blast 3
(50 psi)



d) Blast 4
(70 psi)



e) Mid-span cracking
(Blast 3)



f) Mid-span cracking
(Blast 4)



g) Mid-span cracking close up
(Blast 4)

Figure 6-10 CRC-2%A-#4-0, Photographs Showing Results of Blast 1-4

6.4.2. Beam with UHPC and 15M NSR (CRC-2%A-15M-S)

The second beam in the UHPC & NSR series is CRC-2%A-15M-S. It is detailed with 15M normal strength bars alongside shear reinforcement in the shear spans at 100 mm o/c. The CRC in the contains 2% of fiber type A. The imposed shockwave properties and resulting displacements can be found in Table 6-1. A complete description of each pressure-impulse and displacement time histories is presented in Figure 6-11 complimented by sequential blast photographs in Figure 6-12. Data and comments at each blast test can be found in Table 6-9.

Blast 1 (17 psi) strained the longitudinal tension reinforcement to ~ half-way to yield. The blast resulted in a maximum support rotation of 0.52° corresponding to a max displacement of 10 mm.

Blast 2 (30 psi) strained the longitudinal reinforcement beyond it's yielding strain. Given the minor support rotation of 0.66° , damage remains classified as superficial. The resulting maximum and residual displacements were of 12.89 mm and 0.41 mm. Six hairline cracks were recorded in both the constant moment region and shear spans.

Blast 3 (50 psi) pushed the steel reinforcement into the inelastic range. The beam approaches moderate damage status as maximum support rotation is recorded at 1.64° . Moreover, the considerable maximum mid-span displacement of 31.91 mm results in a critical crack just below the top point load application point.

Blast 4 (70 psi) imposed adequate force as to cause specimen failure through means of fiber pullout at two critical cracks (numbered 2 and 8). Limited visible debris was generated at failure as the fibers held the concrete together. The concrete on the compression face remained unscathed as there is no evidence of concrete crushing. Considering the maximum support rotation of 2.15° , the specimen is described as having damage quantified between moderate and heavy.

Table 6-9 CRC-2%A-15M-S Acquired Blast Data

Blast #	δ_{max} (mm)	Support Rotation ($^\circ$)	δ_{res} (mm)	Peak Strain ($\mu\text{mm/mm}$)	Residual Strain ($\mu\text{mm/mm}$)	Comments
1	10.22	0.52	0.91	1082	372	- Specimen in elastic range - 3 hairline cracks are visible in the constant moment region
2	12.89	0.66	0.41	3501	863	- A total of 6 cracks are found in the flexural zone - Strain gauge recording depicts steel reinforcement yielded
3	31.91	1.64	6.21	4556	1079	- Reinforcement has yielded - Critical crack at point load location - Cracks propagate deeper and have an average width of 0.5 mm - Flexural shear cracks in shear span
4	41.97	2.15	19.08	N/A*	N/A*	- Fiber pullout failure occurs in constant moment region near upper point load - Cracks 2 and 8 are critical, measured at 12.5 mm and 4 mm

* Strain Gauge Rupture, No Data Recorded

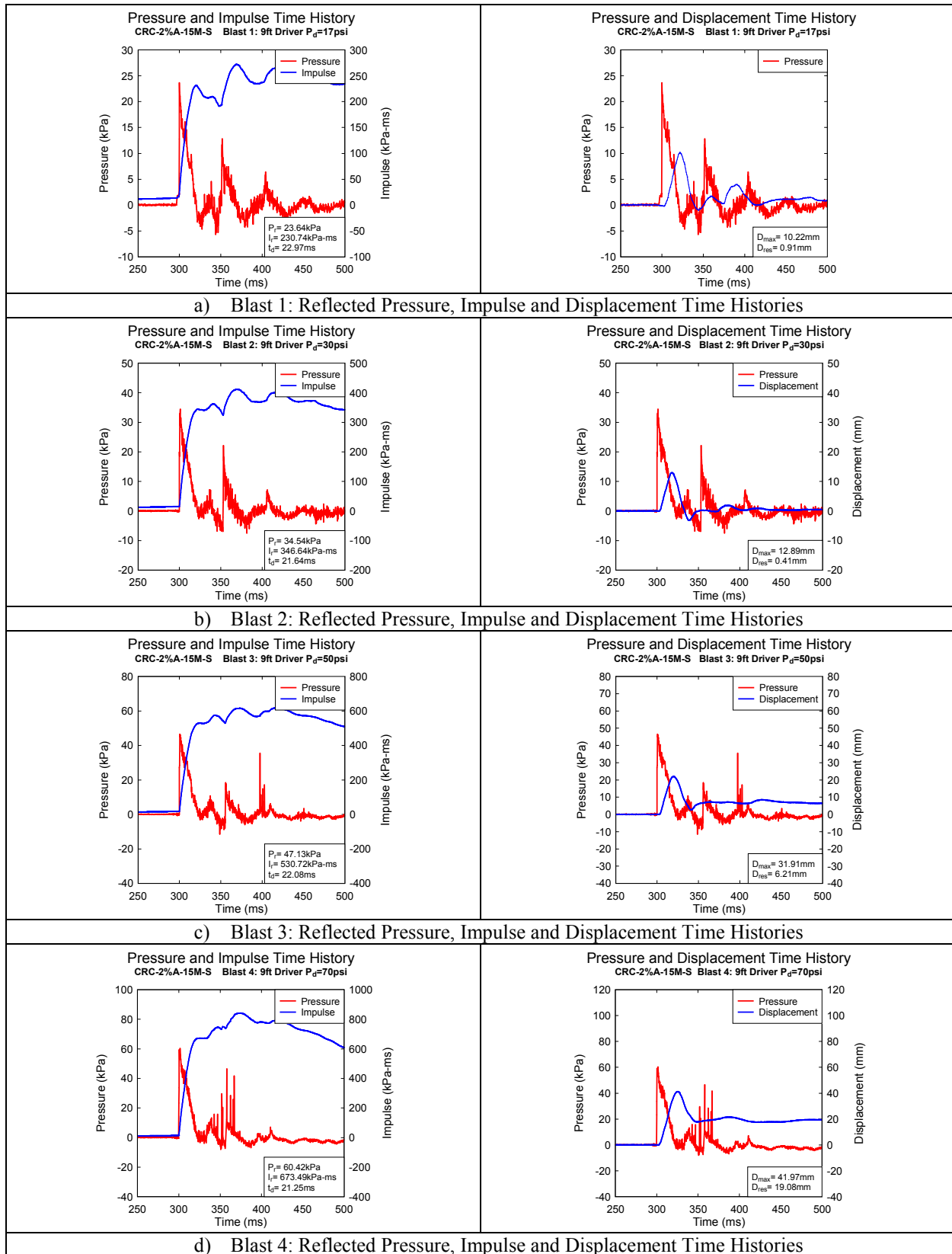
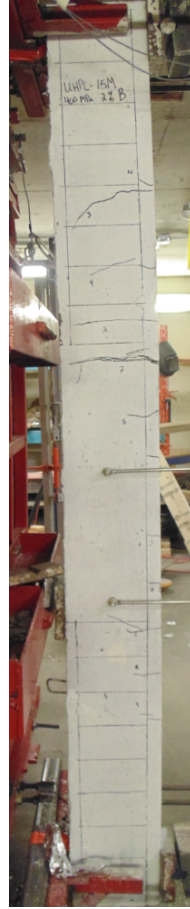


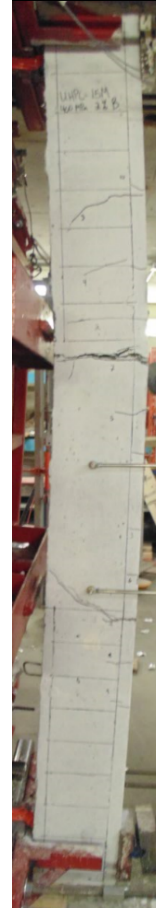
Figure 6-11 CRC-2%A-15M-S, Recorded Reflected Pressure, Impulse and Displacement for Blast 1 to 4



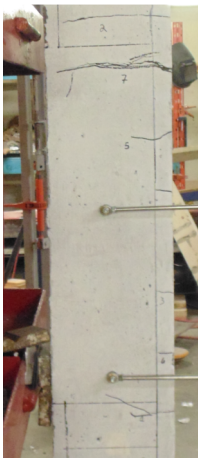
a) Blast 2
(30 psi)



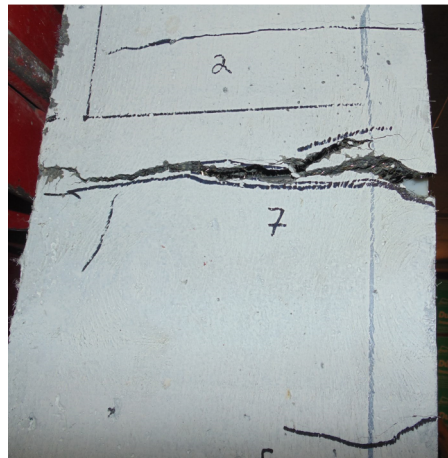
b) Blast 3
(50 psi)



c) Blast 4
(70 psi)



d) Mid-span cracking
(Blast 3)



e) Mid-span fiber pullout
(South face) (Blast 4)



f) Mid-span fiber pullout
(East face) (Blast 4)

Figure 6-12 CRC-2%A-15M-S, Photographs Showing Results of Blast 1-4

6.4.3. Beam with UHPC and 20M NSR (CRC-2%A-20M-0)

Beam CRC-2%A-20M-0 is the third beam which is part of the UHPC & NSR series. It is detailed with 20M normal strength longitudinal reinforcement without any shear reinforcement, the specimen is built with CRC having 2% of fiber type A. The imposed shockwave properties and resulting displacements can be found in Table 6-1. A complete description of each pressure-impulse and displacement time histories is presented in Figure 6-13, complimented by sequential blast photographs in Figure 6-14. Data and comments for the blast tests can be found in Table 6-10.

Blast 1 (17 psi) and Blast 2 (30 psi) kept the longitudinal tension reinforcement within the elastic range. Support rotations remained below 1° with limited residual displacements. A total of three hairline cracks were recorded within the flexural zone after Blast 1, with the number of cracks growing to 7 after Blast 2.

Blast 3 (50 psi) propelled the specimen into the inelastic zone as the longitudinal steel reinforcement yielded. The beam suffered a maximum and residual displacement of 21.48 mm and 2.61 mm.

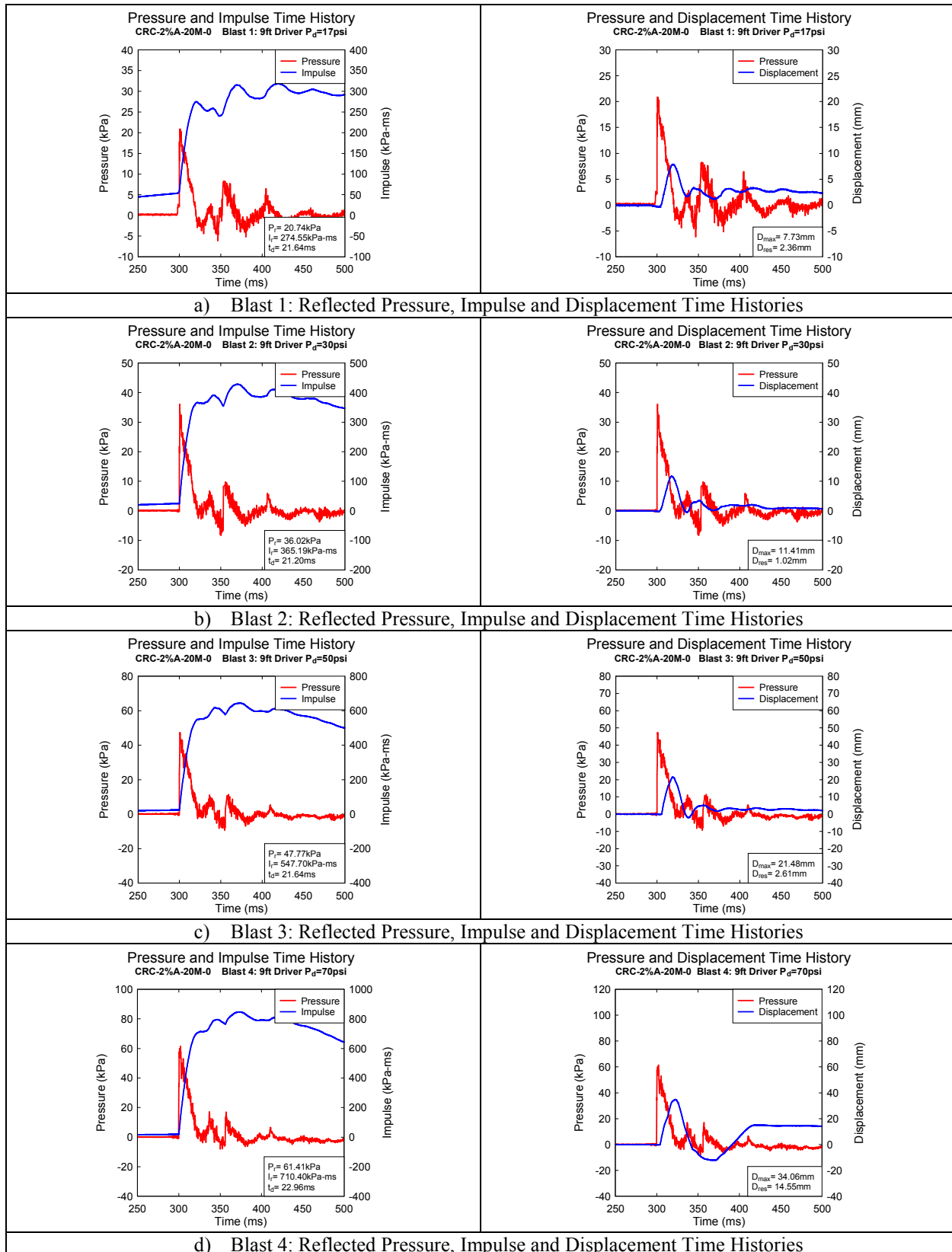
Blast 4 (70 psi) inflicted moderate damage as a support rotation of 1.75° was recorded with maximum and residual displacements of 34.06 mm and 14.55 mm. A plastic hinge develops at mid-span and the two principal cracks further widen to 1.5 mm and 1.2 mm.

Blast 5 (90 psi) led to the failure of the specimen due to fiber pullout at two principal crack locations resulting in crack openings of 10.2 mm and 7.4 mm. The remaining flexural cracks remained hairline. A resulting support rotation of 2.91° indicates that the beam is between moderately and heavily damaged. The failure blast maximum and residual displacements are of 56.67 mm and 28.97 mm, respectively.

Table 6-10 UHPC-2%A-20M-NSR-0 Acquired Blast Data

Blast #	δ_{max} (mm)	Support Rotation ($^\circ$)	δ_{res} (mm)	Peak Strain ($\mu\text{mm/mm}$)	Residual Strain ($\mu\text{mm/mm}$)	Comments
1	7.73	0.40	2.36	1153	431	- Remains elastic with minor damage
2	11.41	0.59	1.02	2116	676	- Approaching elasticity limit, with damage limited to 7 hairline cracks
3	21.48	1.10	2.61	3549	992	- Tensile reinforcement yields - Cracks propagate deeper into the beam cross-section
4	34.06	1.75	14.55	3777	1238	- Cracks numbered 2 and 3 widen the most having respective crack widths of 0.12 mm and 0.1 mm
5	56.67	2.91	28.97	N/A *	N/A *	- Specimen suffers failure. - Moderate damage, cracks limited by fibers to the exception of two cracks which experienced fiber pullout

* Strain Gauge Rupture, No Data Recorded



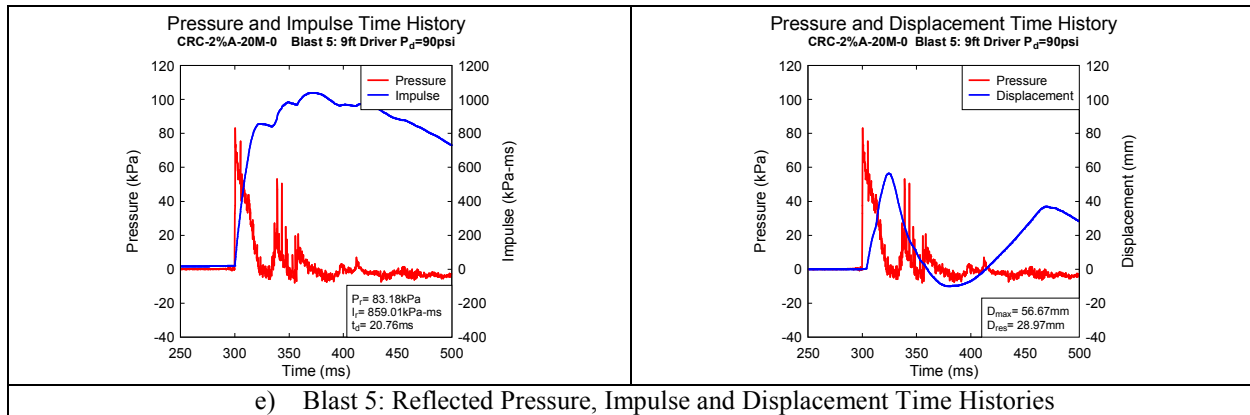


Figure 6-13 CRC-2%A-20M-0, Recorded Reflected Pressure, Impulse and Displacement for Blast 1 to 5

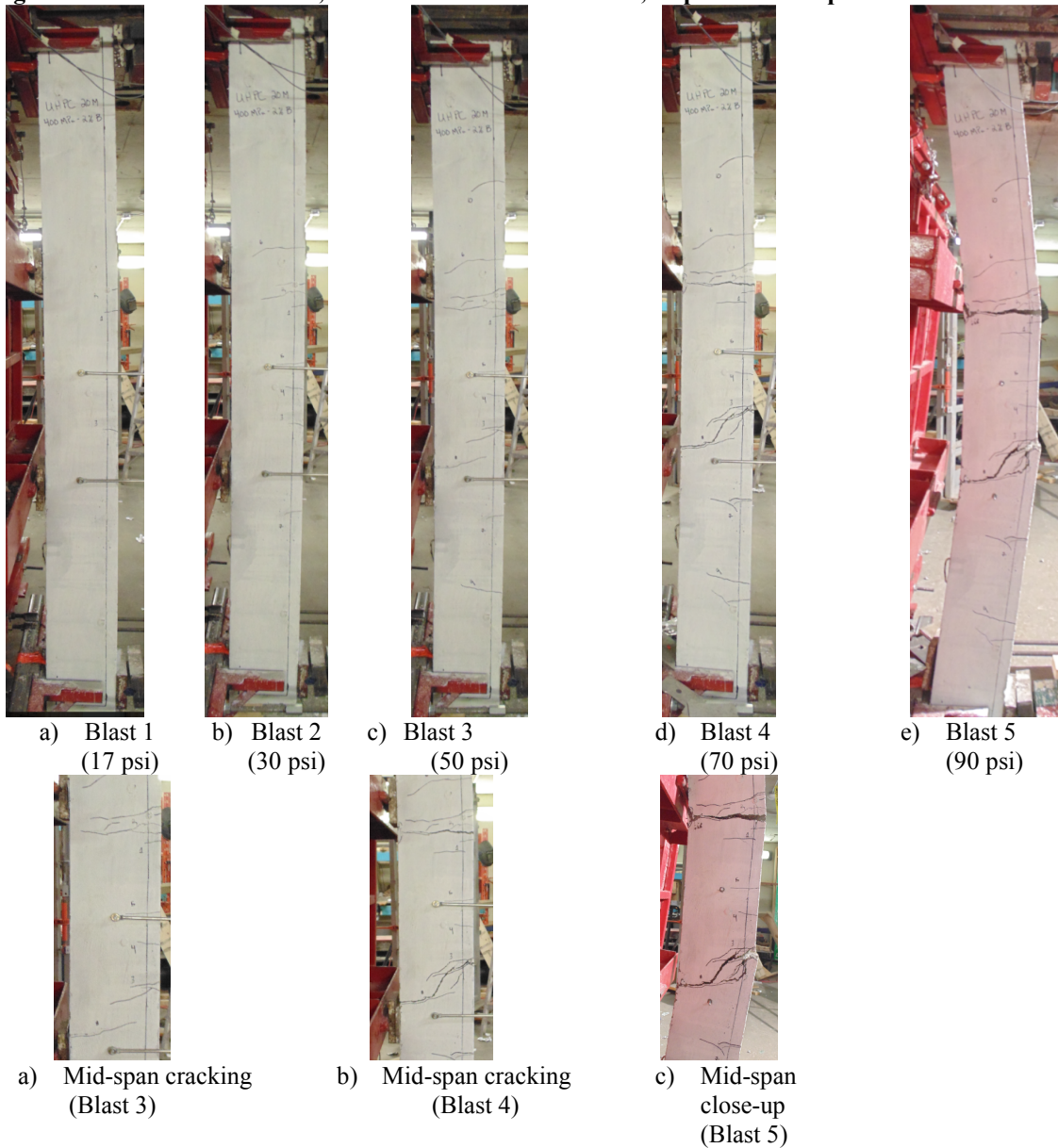


Figure 6-14 CRC-2%A-20M-0, Photographs Showing Results of Blast 1-5

6.4.4. Beam with UHPC and 20M NSR (CRC-2%A-20M-S)

Beam CRC-2%A-20M-S is part of the UHPC & NSR series. It is similar to the previous specimen, in that it contains CRC with 2% fibers (Type A) and 20M bars, however unlike the previous beam, shear reinforcement is provided in the shear spans at 100 mm o/c. The imposed shockwave properties and resulting displacements can be found in Table 6-1. A complete description of each pressure-impulse and displacement time histories is presented in Figure 6-15, complimented by sequential blast photographs in Figure 6-16. Data and comments corresponding to each blast test descriptions can be found in Table 6-11.

Blast 1 (17 psi) kept the longitudinal tension reinforcement within the elastic range. The shockwave resulted in a maximum support rotation 0.23° . No cracks were visible to the naked eye. Blast 2 (30 psi) imposed such force that resulted in the tension bars approaching but not exceeding yield strain. Six hairline cracks were recorded in the constant moment region.

Blast 3 (50 psi) pushed the tension steel into inelastic response. The beam experienced a support rotation of 0.86° which remains in the superficial damage category. Cracks also develop in the shear span resulting in a total of 13 cracks (all cracks remained hairline).

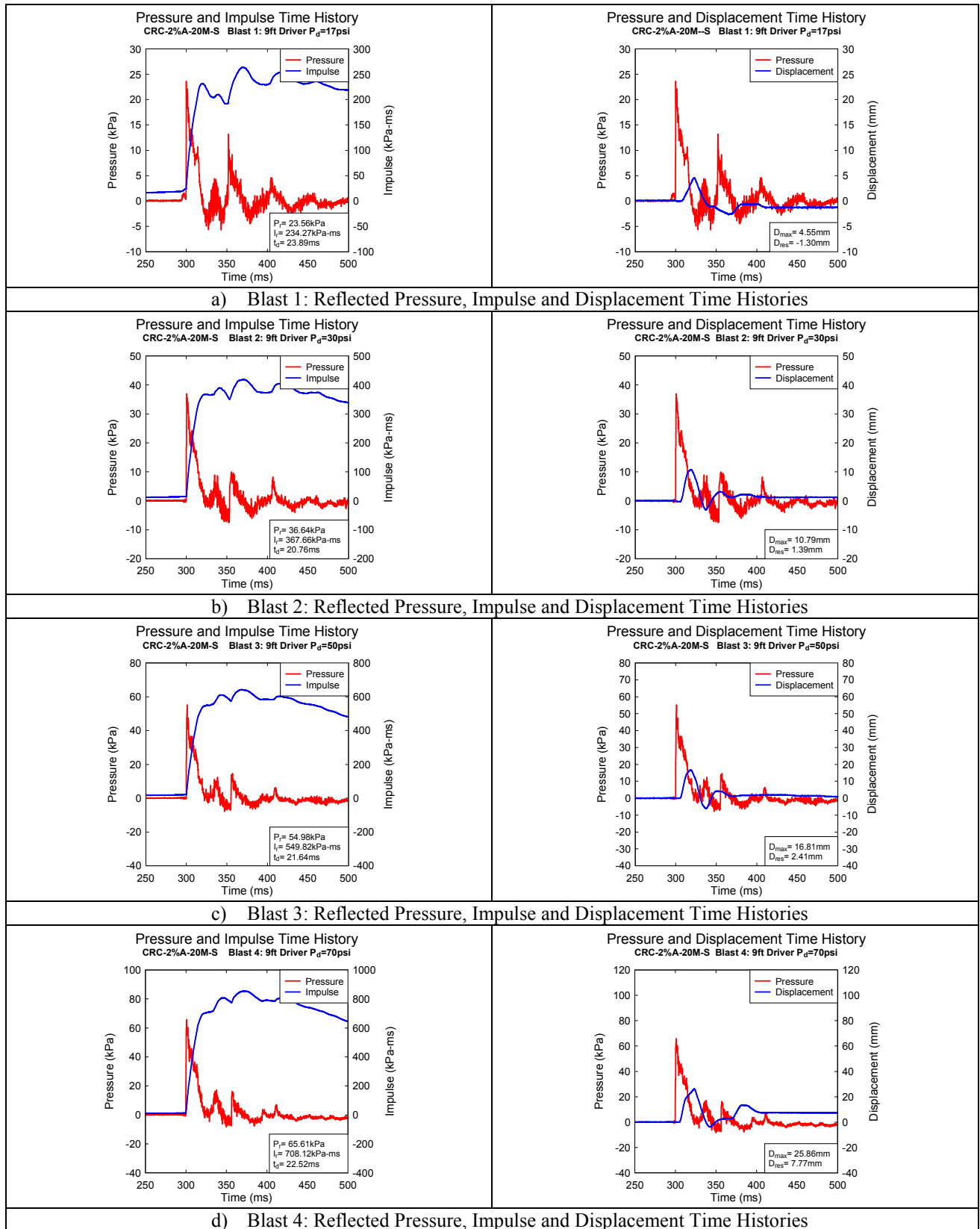
Blast 4 (70 psi) imposed substantial mid-span displacement and the support rotation of 1.33° approaches moderate damage as per current standards. As depicted in Figure 6-16, the fibers bridge the gap in all visible crack locations. The beam begins to experienced some pronounced curvature in the flexural region with cracks concentrating near both load application locations.

Blast 5 (90 psi) ultimately caused the specimen to fail through means of fiber pullout. Damage was considered moderate to heavy when assessing the recorded support rotation of 2.49° . No diagonal shear cracks or signs of concrete crushing were visible upon failure while crack widening was prominent at both the upper and lower point load locations.

Table 6-11 CRC-2%A-20M-S Acquired Blast Data

Blast #	δ_{max} (mm)	Support Rotation ($^\circ$)	δ_{res} (mm)	Peak Strain (mm/mm)	Residual Strain (mm/mm)	Comments
1	4.55	0.23	1.3	856	343	- Elastic response - No visible cracks
2	10.79	0.55	1.39	2250	670	- Formation of 6 hairline cracks in flexural region - Peak strain is slightly under yield
3	16.81	0.86	2.41	3625	906	- Existing cracks propagate deeper into cross-section - Bars are now in plastic range
4	25.86	1.33	7.77	4000	1090	- Crack number 9 width expands considerably to 0.95 mm at bottom point load
5	48.48	2.49	25.3	N/A*	N/A*	- Fiber pullout failure occurs. - Cracks 2 & 9 prove to be critical reaching widths of 3.5 & 8 mm.

* Strain Gauge Rupture, No Data Recorded



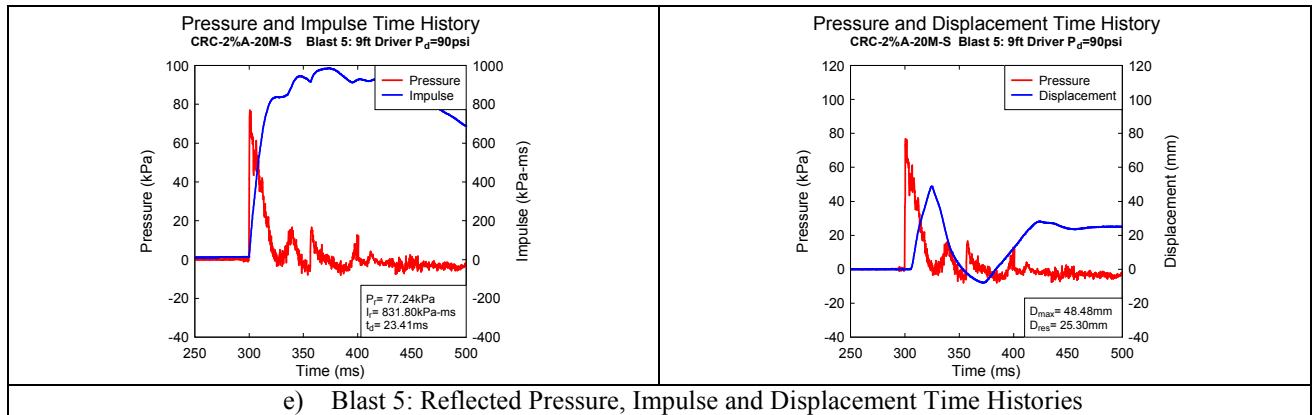


Figure 6-15 CRC-2%A-20M-S, Recorded Reflected Pressure, Impulse and Displacement for Blast 1 to 5

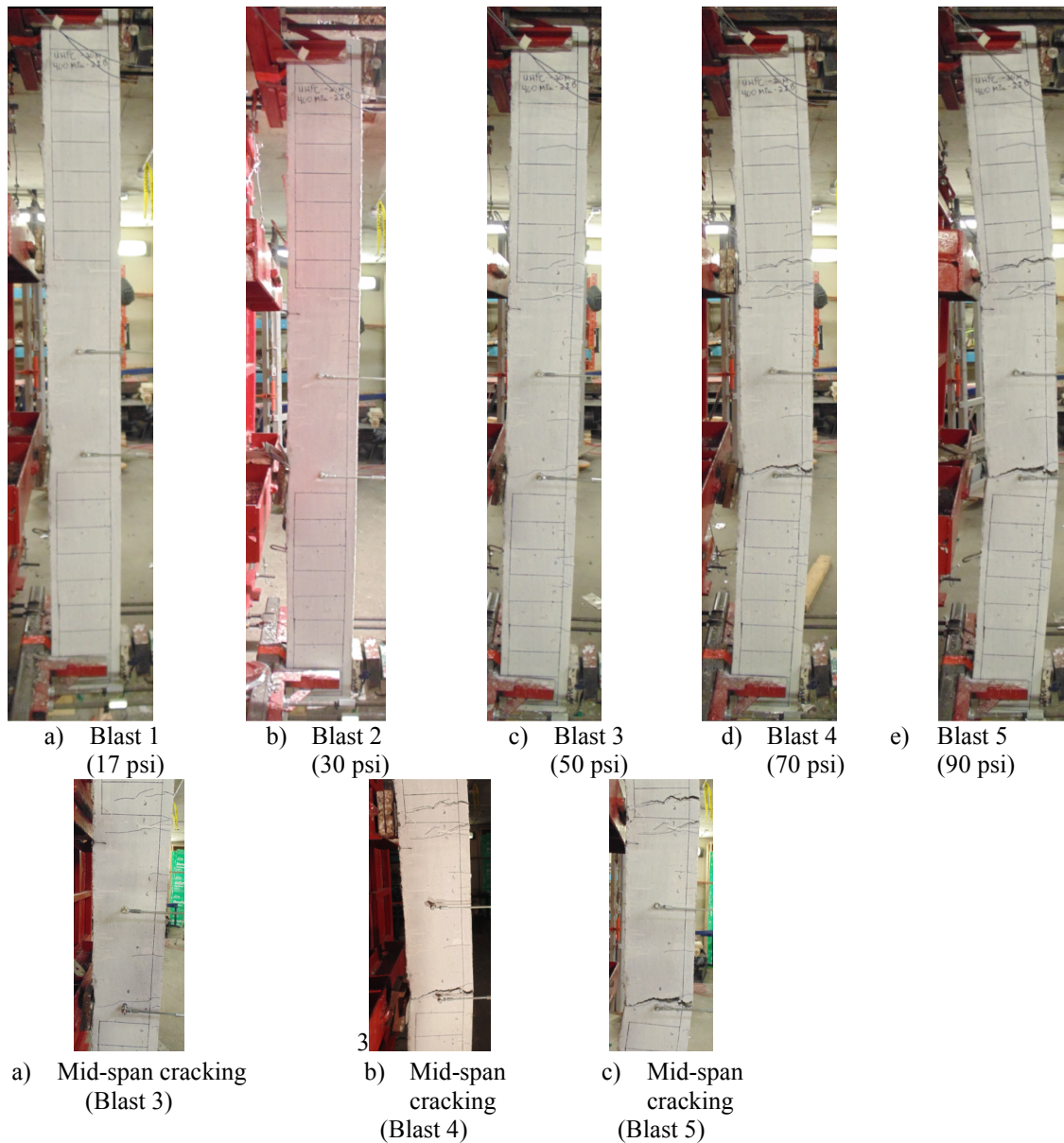


Figure 6-16 CRC-2%A-20M-S, Photographs Showing Results of Blast 1-5

6.5. UHPC & NSR Series (2 of 2)

6.5.1. Beam with UHPC and 1% Fiber B (CRC-1%B-15M-0)

The next set of beams in the UHPC & NSR series examine the effect of fiber type and content. Specimen CRC-1%B-15M-0 is detailed with 15M normal strength bars without any shear reinforcement. It is built with CRC having 1% of fiber type B. The shockwave properties and resulting displacements can be found in Table 6-1. Pressure-impulse and displacement time histories are presented in Figure 6-17, complimented by sequential blast photographs in Figure 6-18. Data and comments after each blast test can be found in Table 6-12.

Blast 1 (17 psi) kept the longitudinal tension reinforcement in the elastic range. The shockwave induced minimal superficial damage with a corresponding support rotation of 0.41° and a total of 6 hairline cracks. Blast 2 (30 psi) strained the longitudinal reinforcement slightly beyond its yielding stress. The resulting maximum support rotation was recorded at 0.73° . A total of 13 hairline cracks were recorded along the beam span.

Blast 3 (50 psi) pushed the steel reinforcement into the plastic range. In consequence of the given dynamic loading a total of 31 cracks have developed throughout the span, including diagonal cracks forming near the top and bottom load application points as depicted in Figure 6-18.

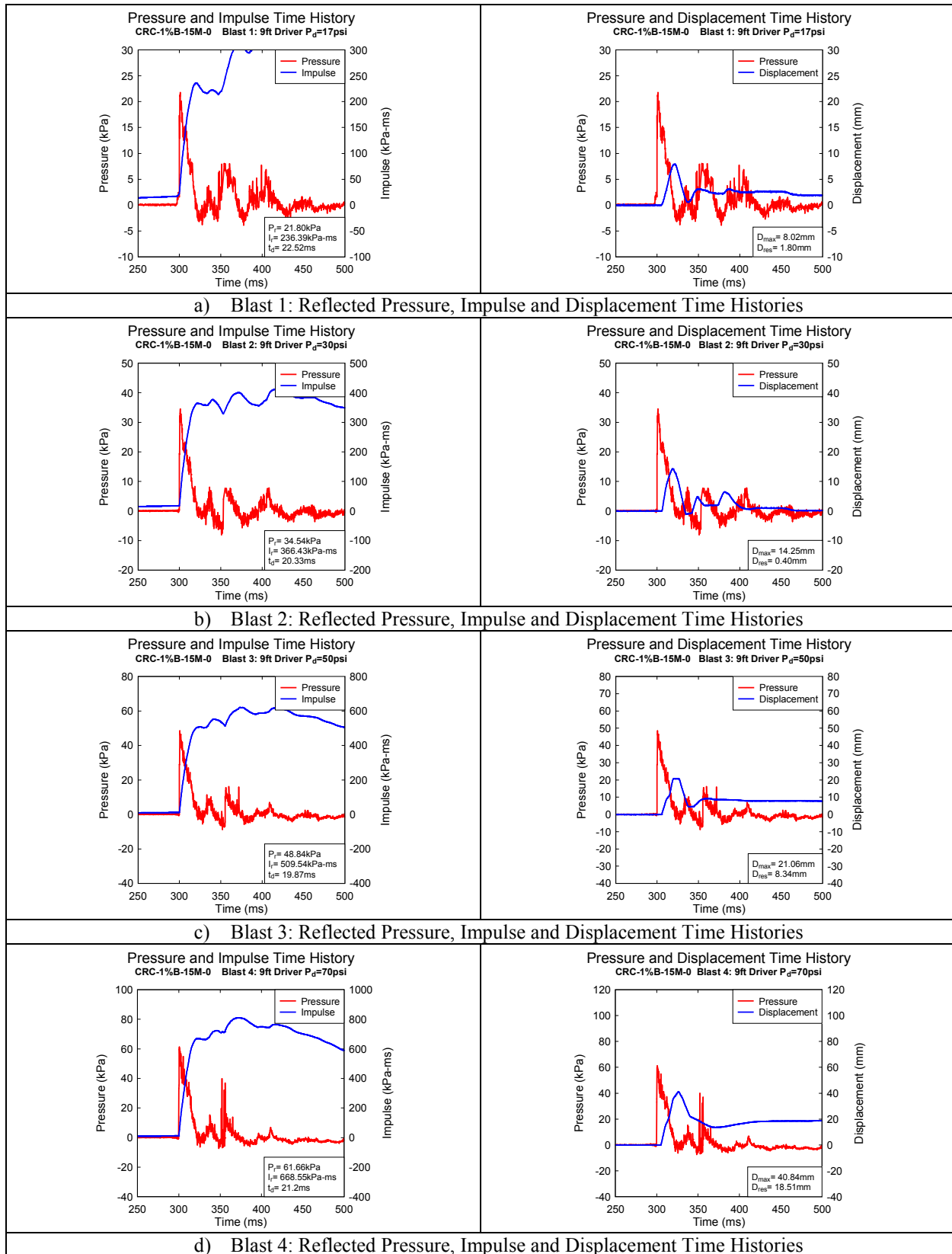
Blast 4 (70 psi) inflicted sufficient force as to initiate visible fiber bridging of the tension side concrete at the upper point load location. Maximum displacement reaches 41 mm. Damage can be quantified as moderate for a maximum support rotation value of 2.10° . The steel is well into the inelastic range. No fragmentation is generated upon slow motion video review of the blast.

Blast 5 (90 psi) caused complete blowout failure of the specimen; initiated by major crack widening at upper point load, leading to rupture of the 15M longitudinal reinforcement, followed by complete severance of the beam. With a maximum support rotation of 12.49° , the blast caused a hazardous failure projecting the top half of the beam into mid air.

Table 6-12 CRC-1%B-15M-0 Acquired Blast Data

Blast #	δ_{max} (mm)	Support Rotation ($^\circ$)	δ_{res} (mm)	Peak Strain (mm/mm)	Residual Strain (mm/mm)	Comments
1	8.02	0.41	1.8	1423	737	- Elastic range response - Formation of 6 hairline cracks
2	14.25	0.73	0.4	2940	1237	- Longitudinal reinforcement yields
3	21.06	1.08	8.34	3652	991	- Further development of cracks (mean crack width = 0.18mm)
4	40.84	2.10	18.51	N/A*	N/A*	- Specimen experiences significant rotation and crack propagation - Mean crack width of 0.55 mm, while crack 11 is measured at 6 mm
5	247.11	12.49	N/A	N/A*	N/A*	- Complete specimen blowout - Beam severed at upper point load

* Strain Gauge Rupture, No Data Recorded



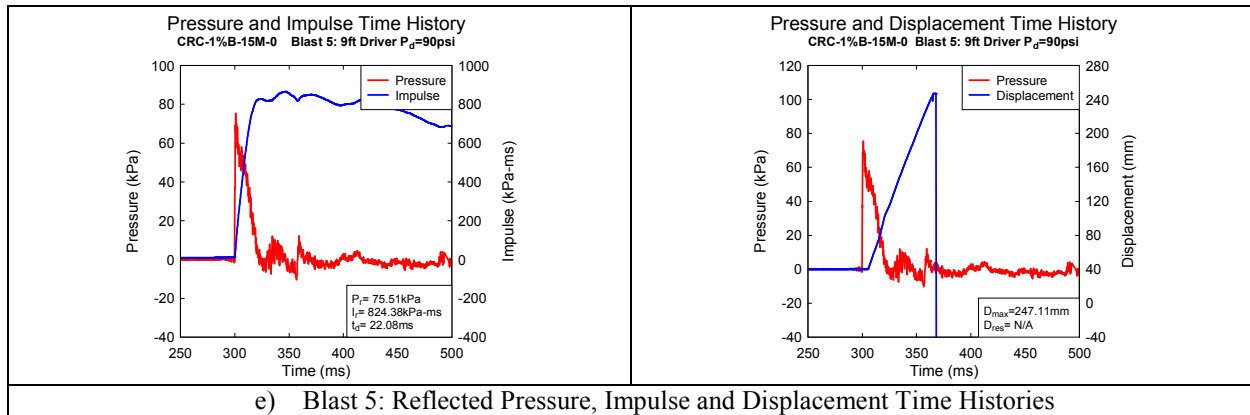


Figure 6-17 CRC-1%B-15M-0, Recorded Reflected Pressure, Impulse and Displacement for Blast 1 to 5

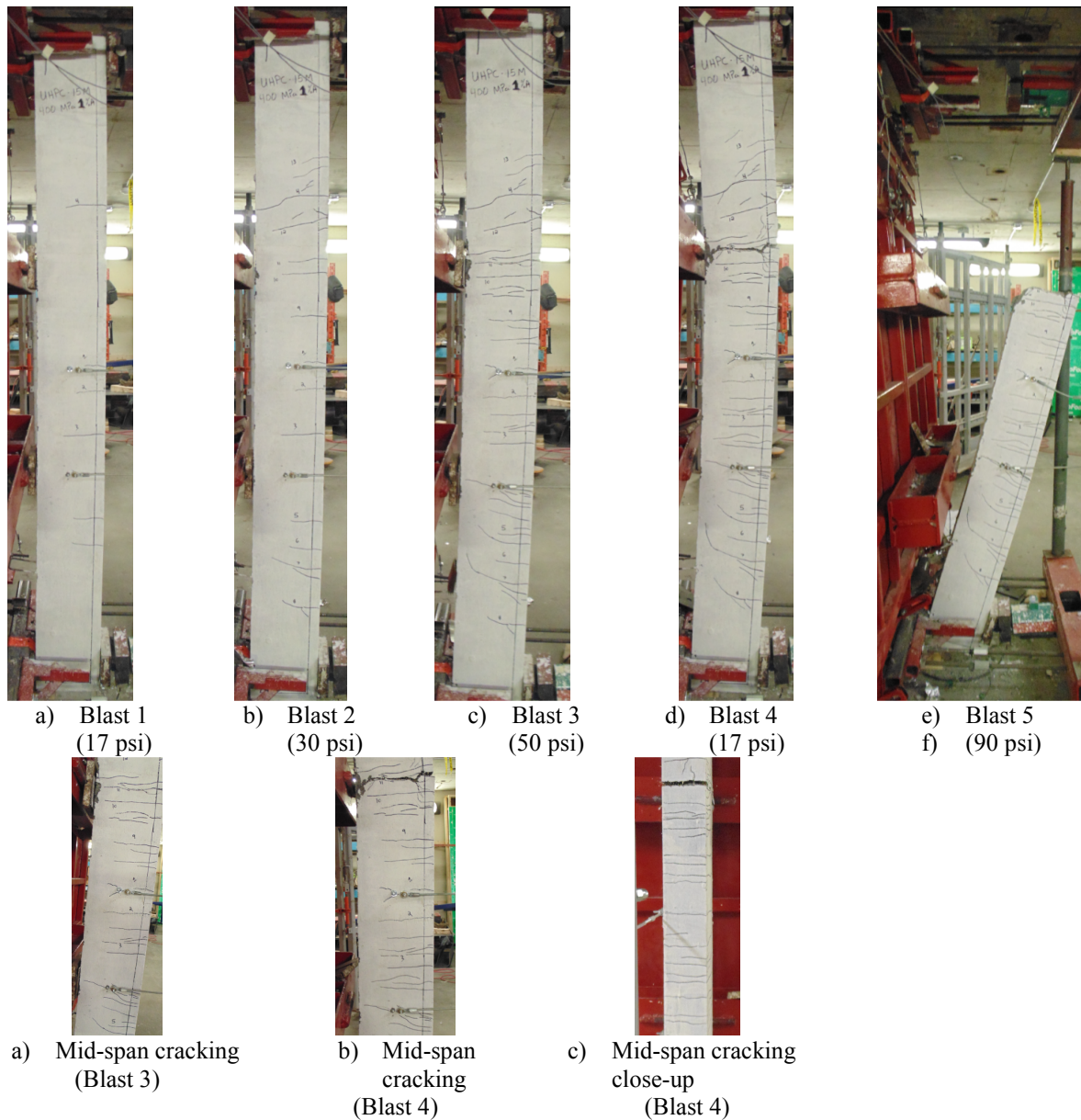


Figure 6-18: CRC-1%B-15M-0, Photographs Showing Results of Blast 1-5

6.5.2. Beam with UHPC and 2% Fiber B (CRC-2%B-15M-0)

Beam CRC-2%B-15M-0, has a dual purpose of examining the effect of fiber content in addition to the effect of fiber type (B vs. A). It is part of the UHPC & NSR series and is detailed with 15M normal strength bars without any shear reinforcement. The CRC mix has a fiber content of 2% of fiber type B. The imposed shockwave properties and resulting displacements can be found in Table 6-1. A complete description of each pressure-impulse and displacement time histories is presented in Figure 6-19, complimented by sequential blast photographs in Figure 6-20 CRC-2%B-15M-0, photographs showing results of blast 1-5. Data corresponding to the different blast tests along with descriptions can be found in Table 6-13.

As with the previous beam, Blast 1 (17 psi) kept the reinforcement within the elastic range, while Blast 2 (30 psi) pushed the strains just past yield levels. These shockwaves resulted in low support rotations of 0.43° and 0.95° , corresponding to superficial damage. Hairline cracks were developed during Blast 2, spreading half way into the shear spans. Blast 3 (50 psi) pushed the tension steel well into plastic response. Consequently, the support rotation approaches the moderate damage status with a value of 1.52° .

Blast 4 (70 psi) was sufficient to initiate fiber pullout on the tension face with a maximum crack width of 26 mm. Given the maximum recorded support rotation of 3.18° the specimen can be classified as having moderate-heavy damage. Through visual inspection, longitudinal rebar appears to remain in tact. Minimal secondary fragments were generated as the fibers held the concrete together. As depicted in Figure 6-20 CRC-2%B-15M-0 photographs showing results of blast 1-5, the beam experienced some pronounced curvature in the flexural region with cracks concentrating near mid-span on the tension side.

Blast 5 (90 psi) proved sufficient to completely sever the beam specimen in two separate pieces. The rebar rupture occurred at the location of fiber pull out; Figure 6-20 CRC-2%B-15M-0, photographs showing results of blast 1-5 depicts the failure. Although all instrumentation was detached from specimen prior to test, the failure mode is classified as a complete blow out with a support rotation exceeding 10° .

Table 6-13 CRC-2%B-15M-0 Acquired Blast Data

Blast #	δ_{max} (mm)	Support Rotation ($^\circ$)	δ_{res} (mm)	Peak Strain (mm/mm)	Residual Strain (mm/mm)	Comments
1	8.94	0.46	2.15	1428	760	- 5 hairline cracks develop in constant moment region - Specimen in elastic range
2	18.52	0.95	1.42	2940	1190	- Longitudinal reinforcement yields - Total of 12 cracks are visible
3	29.54	1.52	14.48	3616	1042	- Bars well into the inelastic response - Mean crack width is of 0.55 mm
4	62.04	3.18	38.3	3420 [#]	1042 [#]	- Crack concentrations at mid-span with the largest crack reaching 26 mm - Mean crack width is of 2.2 mm
5	N/A*	N/A*	N/A*	N/A*	N/A*	- Rebar rupture and specimen completely breaks in half

[#] Strain Gauge May Be Damaged Beyond Point of Valid Data ; * Data Unavailable, Complete Specimen Blow Out

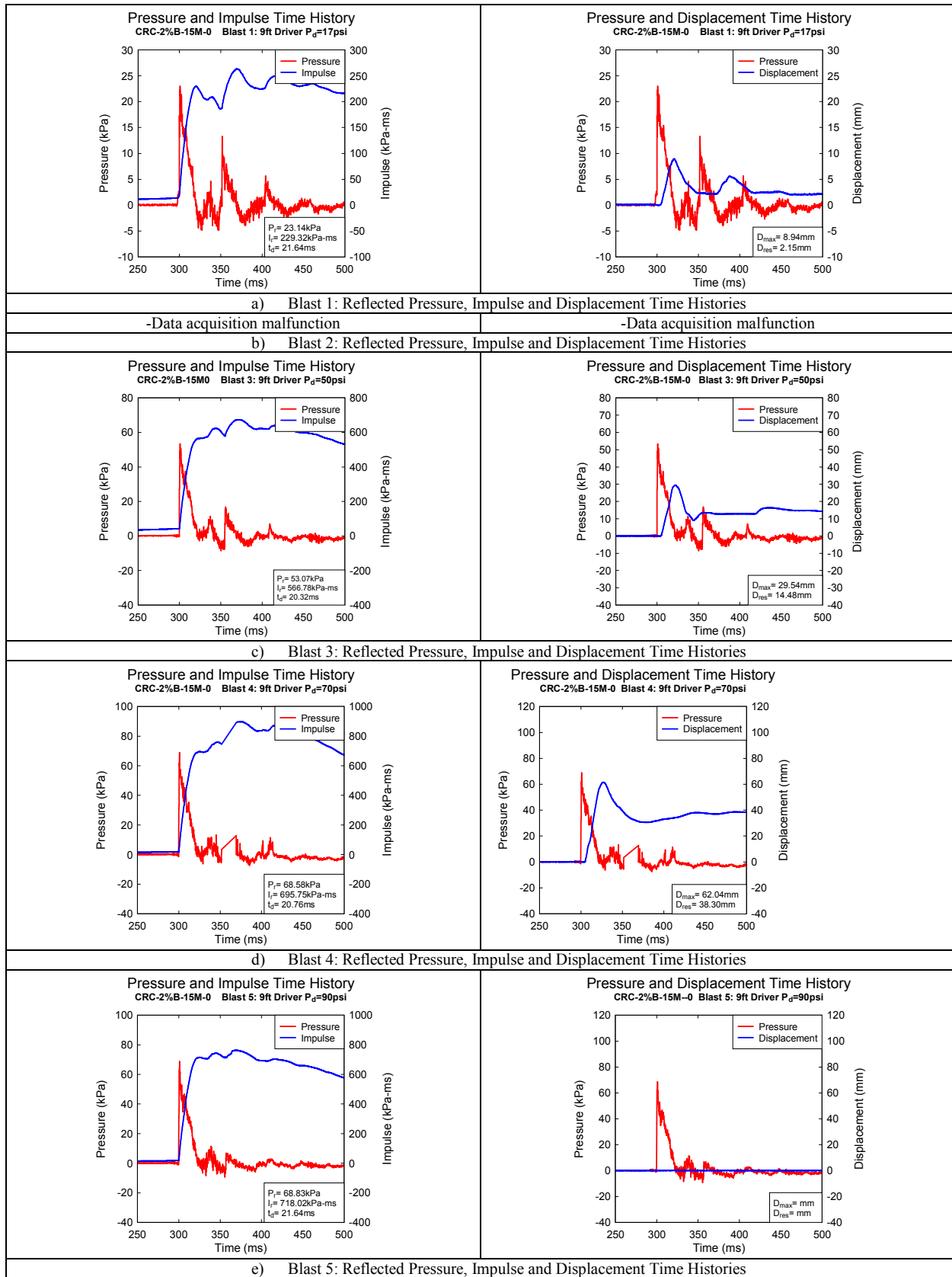


Figure 6-19 CRC-2%B-15M-0, Recorded Reflected Pressure, Impulse and Displacement for Blast 1

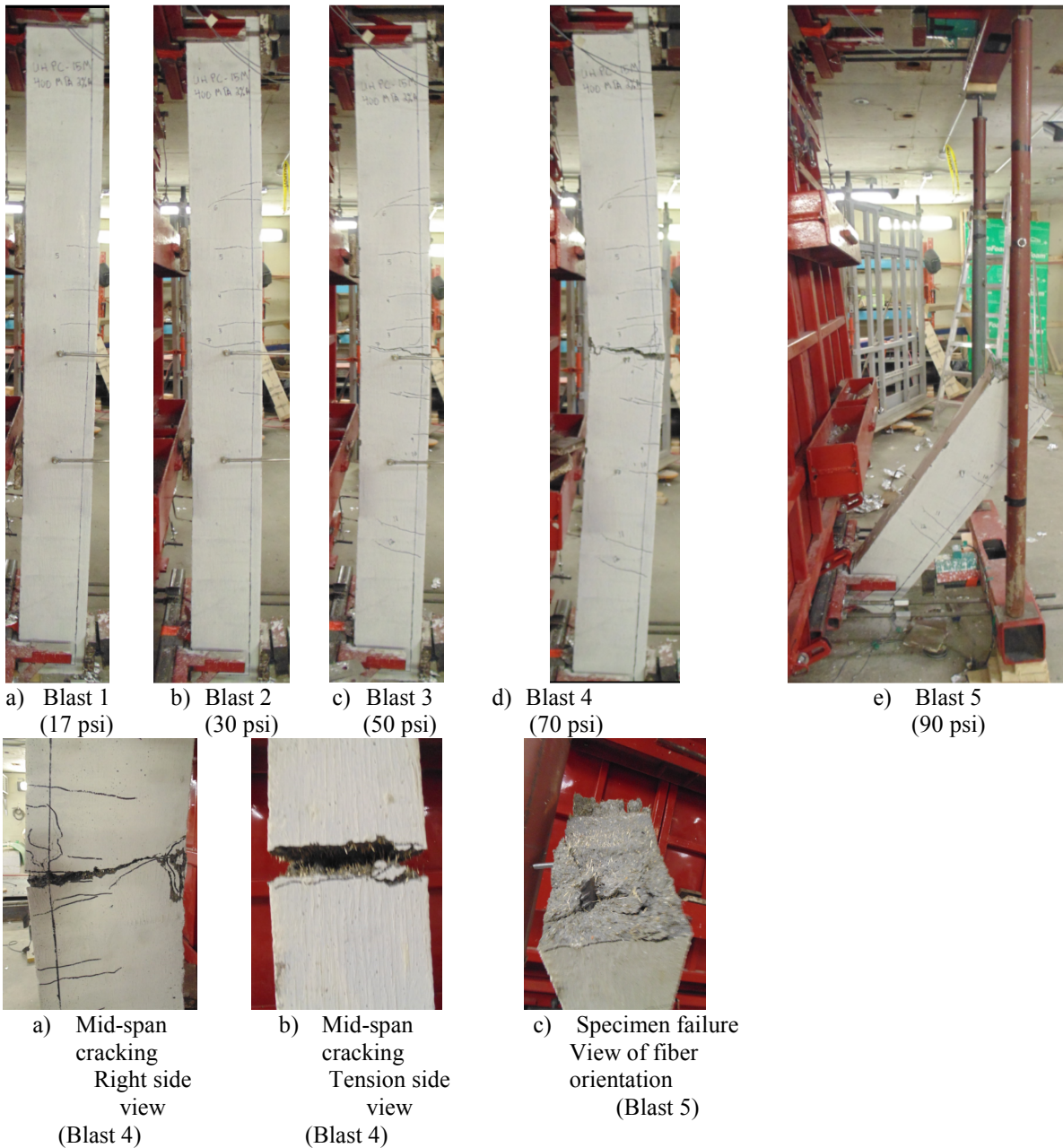


Figure 6-20 CRC-2%B-15M-0, Photographs Showing Results of Blast 1-5

6.5.3. Beam with UHPC and 3% Fiber B (CRC-3%B-15M-0)

The last specimen in the UHPC & NSR series is referred to as CRC-3%B-15M-0. It is detailed with 15M normal strength bars without any shear reinforcement. The CRC in the specimen has the highest fiber content of 3% of fiber type B. The shockwave properties and resulting displacements can be found in Table 6-1. Pressure-impulse and displacement time histories is presented in Figure 6-21, complimented by sequential blast photographs in Figure 6-22. Data and comments corresponding to the blast tests can be found in Table 6-14.

Similar to the previous 2 beams, Blast 1 (17 psi) tested the elastic response of this specimen, while Blast 2 (30 psi) strained the longitudinal reinforcement up to yield. The resulting maximum support rotation after Blast 2 was of 0.68° , which indicates only superficial damage.

Blast 3 (50 psi) caused a total of 6 cracks to emerge. The given blast pushed the reinforcement into its inelastic range. Damage is still quantified as superficial with a support rotation of 1.03° .

Blast 4 (70 psi) imposed adequate force as to cause specimen failure through means of fiber pullout in one principal location slightly above mid-span. Limited visible debris was generated as the fibers held all concrete together. The concrete on the compression face did not show any signs of concrete crushing whatsoever. Considering the maximum support rotation of 2.34° , the specimen is described as having damage quantified between moderate and heavy. Owing to the severe failures experienced by the previous beams no further blasts were applied to this specimen.

Table 6-14 CRC-3%B-15M-0 Acquired Blast Data

Blast #	δ_{max} (mm)	Support Rotation ($^\circ$)	δ_{res} (mm)	Peak micro Strain (mm/mm)	Residual micro Strain (mm/mm)	Comments
1	6.25	0.32	1.3	595	181	- Only one hairline crack is visible to the naked eye
2	13.26	0.68	1.39	2545	738	- Longitudinal steel reinforcement reached its yield strain - 6 hairline cracks appear
3	20.01	1.03	4.94	3393	946	- Specimen is within inelastic range - Mean crack width is 0.1 mm
4	45.65	2.34	23.88	N/A*	N/A*	- Fiber pull out occurs in localized crack reaching a maximum width of 21.2 mm

* Strain Gauge Rupture, No Data Recorded

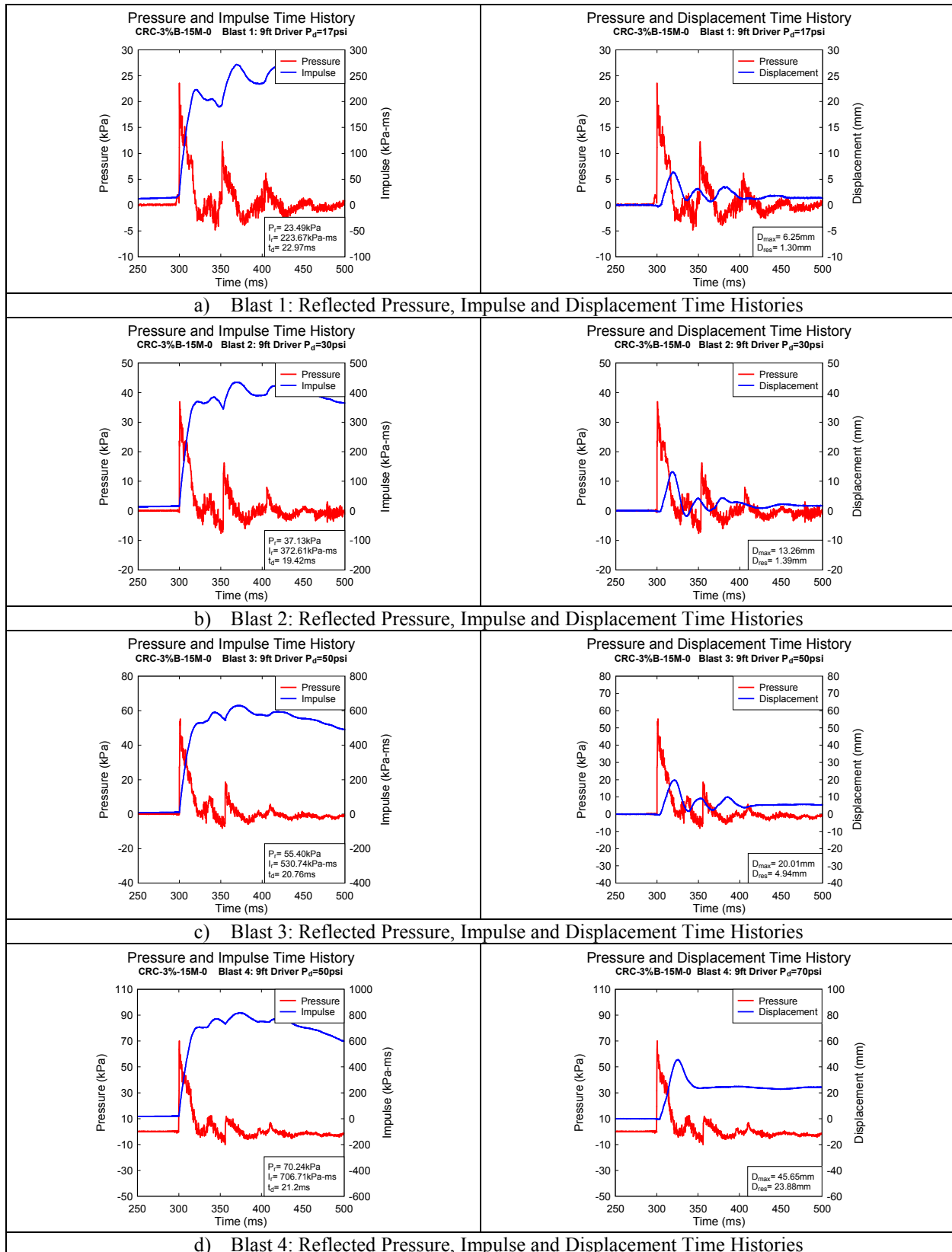
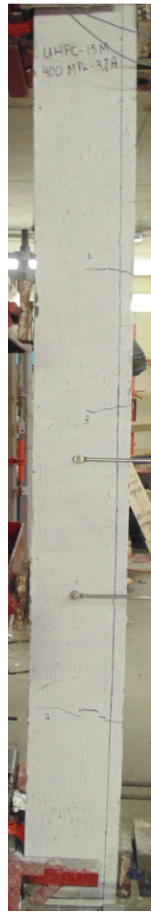


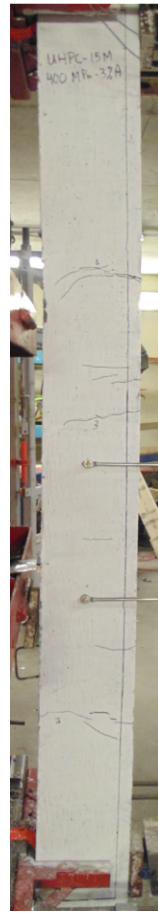
Figure 6-21 CRC-3%B-15M-0, Recorded Reflected Pressure, Impulse and Displacement for Blast 1 to 4



a) Blast 1
(17 psi)



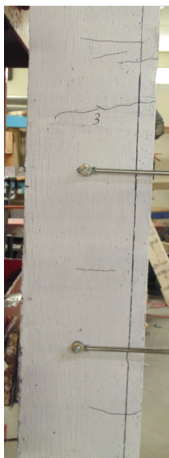
b) Blast 2
(30 psi)



c) Blast 3
(50 psi)



d) Blast 4
(70 psi)



e) Mid-span cracking
(Blast 3)



f) Mid-span cracking
right side
(Blast 4)



g) Mid-span cracking close-up on
tension side
(Blast 4)

Figure 6-22 CRC-2%B-15M-0, Photographs Showing Results of Blast 1-4

6.6. UHPC & HSR Series

6.6.1. Beam with UHPC and #4 HSR (CRC-2%A-#4-HSR-S)

Specimen CRC-2%A-#4-HSR-S is the first specimen tested in the UHPC & HSR series. It is built with UHPC (CRC with 2% Fiber A) and detailed with #4 high strength longitudinal reinforcement. As with all beams in this series it is equipped with transverse reinforcement in the shear spans. The imposed shockwave properties and resulting displacements can be found in Table 6-1. A complete description of each pressure-impulse and displacement time histories is presented in Figure 6-23, complimented by sequential blast photographs in Figure 6-24. Data and comments for each blast test can be found in Table 6-15.

Blast 1 (17 psi) kept the specimen within elastic response. The shockwave induced a support rotation of 0.26° and a total of 2 hairline cracks. Blast 2 (30 psi) strained the longitudinal reinforcement below the yielding strain of the MMFX bars. The maximum support rotation was 0.72° . The number of hairline cracks grew to six.

Blast 3 (50 psi) was also in the elastic range of the high-strength bars ($\epsilon_{\max}=0.0045$ compared to $\epsilon_y=0.0056$ from coupons). Support rotation is slightly above 1° . A total of 11 cracks have developed throughout the constant moment region and shear span, however no diagonal crack formation was observed near the top and bottom supports as depicted in Figure 6-24.

Blast 4 (70 psi) caused complete blowout failure of the specimen; initiated by major crack widening at the upper point load location, leading to rupture of the #4 high-strength bars. Through slow motion video analysis, the rupture can be considered as a hazardous failure as the specimen was nearly completely severed only being held together by approximately 40 mm of fiber reinforced concrete. The mean crack width of 0.08 mm remained identical to that of blast 3 to the exception of the major failure plane at the upper point load location. Debris and projectiles were limited to minimal dust generation upon member failure.

Table 6-15 CRC-2%A-#4-HSR-S Acquired Blast Data

Blast #	δ_{max} (mm)	Support Rotation ($^\circ$)	δ_{res} (mm)	Peak Strain (mm/mm)	Residual Strain (mm/mm)	Comments
1	5.05	0.26	0.95	696	169	- Two hairline cracks appear in the constant moment region
2	14.01	0.72	2.13	2206	297	- Longitudinal reinforcement is elastic - 6 cracks develop along beam span
3	21.9	1.12	0.49	4502	956	- Reinforcement slightly below yield - Several additional cracks appear for a total of 11 cracks having an average width of 0.08 mm - No diagonal cracks appear
4				5486	1543	- Rebar rupture occurs at location of upper point load application - Crack width at rupture point is 54.7 mm

* LVDT instrumentation was removed to protect equipment from excessive damage at specimen failure

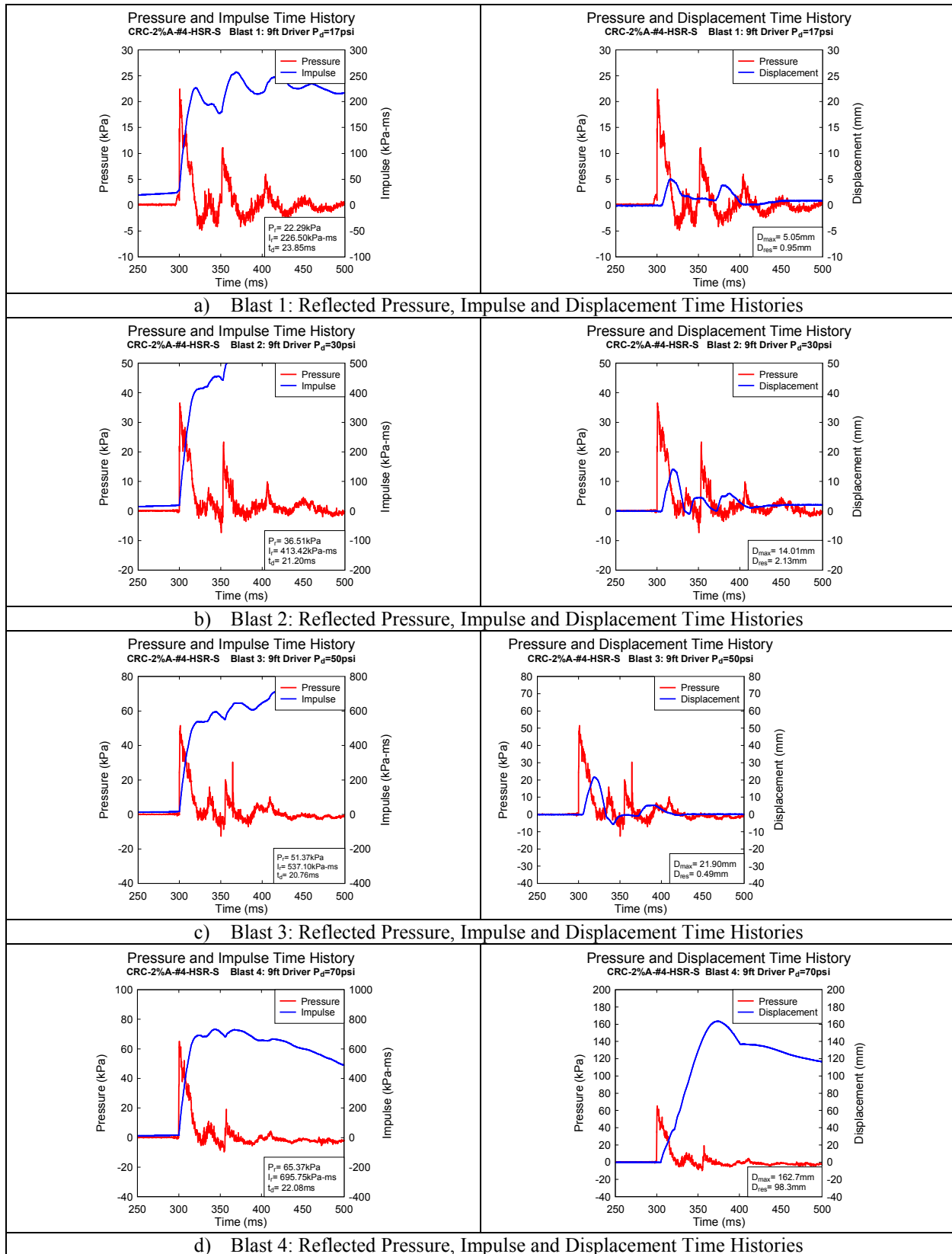
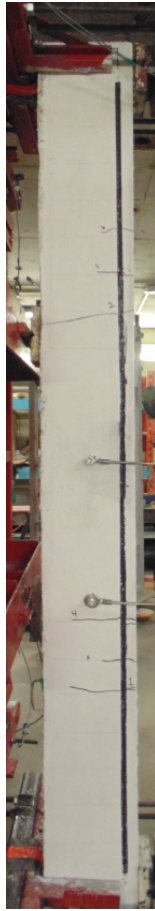


Figure 6-23 CRC-2%A-#4-HSR-S, Recorded Reflected Pressure, Impulse and Displacement for Blast 1 to 4



a) Blast 1
(17 psi)



b) Blast 2
(30 psi)



c) Blast 3
(50 psi)



d) Blast 4
(17 psi)



e) Mid-span cracking
left side
(Blast 4)



f) Mid-span cracking
(Blast 4)



g) Mid-span cracking
right side
(Blast 4)

Figure 6-24 CRC-2%B-#4-HSR-0, Photographs Showing Results of Blast 1-4

6.6.2. Beam with UHPC and #5 HSR (CRC-2%A-#5-HSR-S)

Beam CRC-2%A-#5-HSR-S is the second specimen of the UHPC & HSR series. It is identical to the previous beam but is detailed with #5 high strength bars. The shockwave properties and resulting displacements can be found in Table 6-1. Pressure-impulse and displacement time histories are presented in Figure 6-25, complimented by sequential blast photographs in Figure 6-26. Data and comments for each blast test can be found in Table 6-16.

Blast 1 (17 psi) and Blast 2 (30 psi) kept the high-strength reinforcement within the elastic range. The shockwave induced minimal superficial damage with corresponding support rotations of 0.34° and 0.72° , with a total of 7 hairline cracks after Blast 2, but crack widths remained hairline.

Blast 3 (50 psi) increased the strains, but the data indicates reinforcement was still below yielding ($\epsilon_{\max}=0.004159$ vs. $\epsilon_y=0.0061$ from coupons). Cracking developed further, however no diagonal crack formation as depicted in Figure 6-26.

Blast 4 (70 psi) imposed substantial mid-span displacement and a resulting support rotation of 1.44° which approaches the description for moderate damage as per current standards. The strains in the reinforcement indicate they are now in the inelastic range. The beam begins to experience some pronounced curvature in the flexural region with cracks concentrating near the load points.

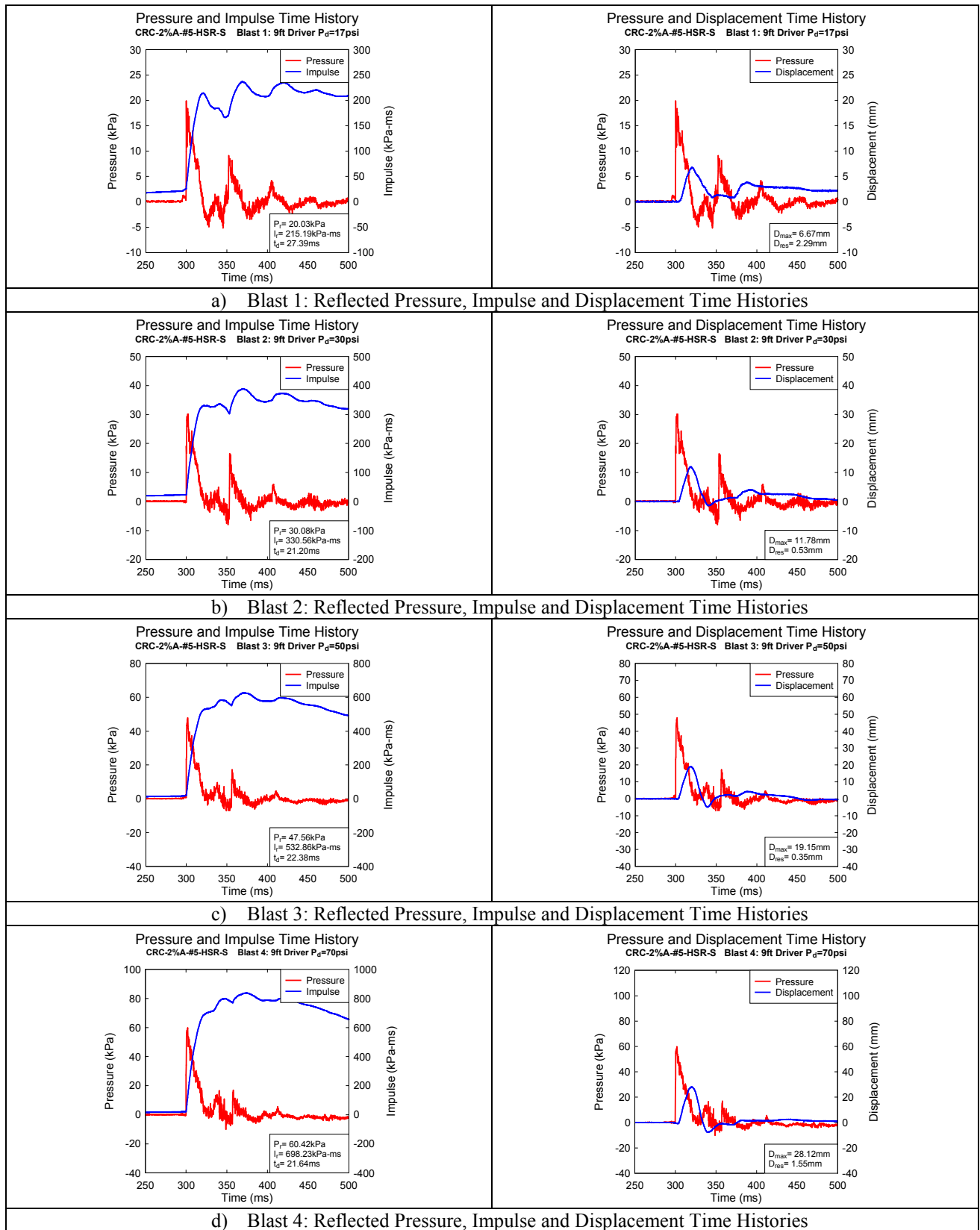
Blast 5 (90 psi) caused the reinforcement to go well into inelastic response, with the specimen enduring moderate damage with a corresponding support rotation of 2.05° . Minor flexural shear cracks begin to develop in upper and lower shear spans with an average crack width of 0.25 mm.

Blast 6 (110 psi) caused the specimen to fail through means of longitudinal reinforcement rupture. Damage was considered hazardous as the specimen was completely severed after experiencing rebar rupture. No signs of concrete crushing were visible despite the severe failure. Additionally, apart from the specimen breaking in half, little to not debris was generated.

Table 6-16 CRC-2%A-#5-HSR-S Acquired Blast Data

Blast #	$\delta_{\max}$ (mm)	Support Rotation ($^\circ$)	δ_{res} (mm)	Peak Strain (mm/mm)	Residual Strain (mm/mm)	Comments
1	6.67	0.34	2.29	936	370	- Total of 3 hairline cracks
2	11.78	0.60	0.53	2204	735	- Specimen remains in elastic range - 4 new hairline cracks develop
3	19.15	0.98	0.35	4159	1056	- Reinforcement still elastic - A total of 8 hairline cracks are recorded
4	28.12	1.44	1.55	6309	2242	- Reinforcement at yield levels - Crack concentrations near upper and lower point load locations
5	40.01	2.05	6.36	12306	6613	- Upwards of 24 cracks develop with a mean width of 0.25 mm
6	N/A*	N/A*	N/A*	N/A*	N/A*	- Complete specimen failure - Longitudinal reinforcement rupture

* Strain Gauge failure, LVDT Removed to Prevent Equipment Damage at Member Failure



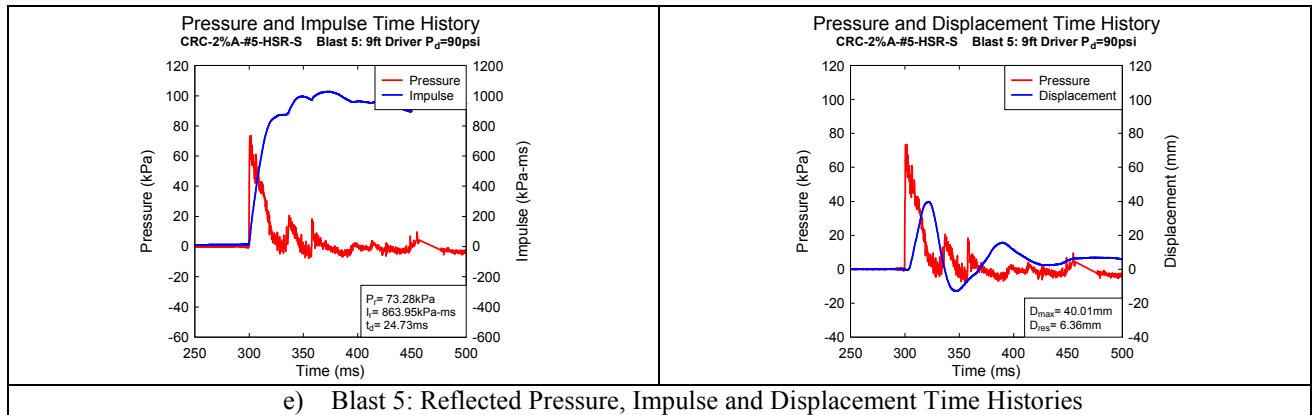


Figure 6-25 CRC-2%A-#5-HSR-S, Recorded Reflected Pressure, Impulse and Displacement for Blast 1

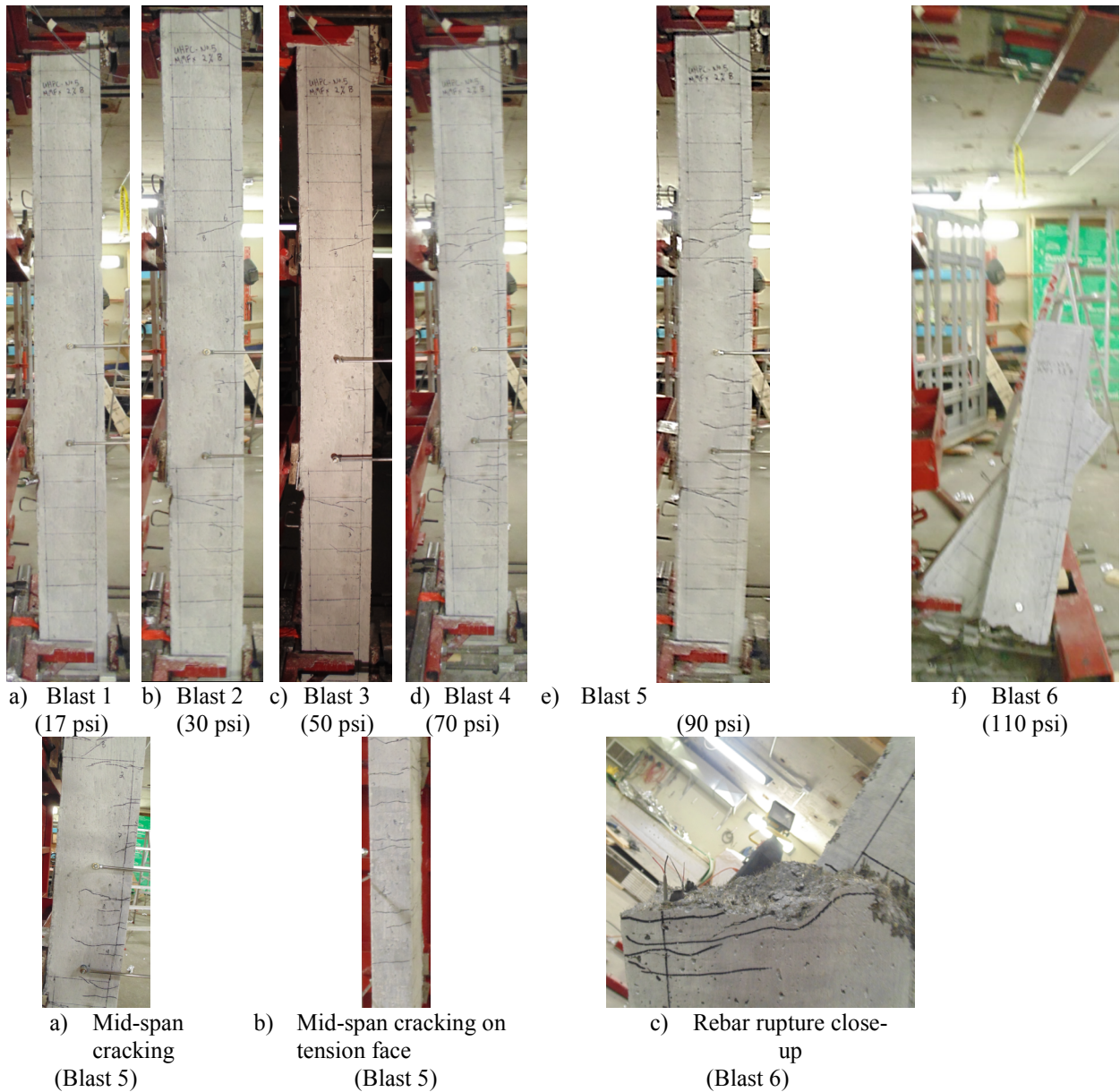


Figure 6-26 CRC-2%B-#5-HSR-S, Photographs Showing Results of Blast 1-6

6.6.3. Beam with UHPC and #6 HSR (CRC-2%A-#6-HSR-S)

The final specimen of the experimental program is constructed using UHPC and #6 high-strength reinforcing bars, it is also the final specimen of the UHPC & HSR series. The imposed shockwave properties and resulting displacements can be found in Table 6-1. Pressure-impulse and displacement time histories are presented in Figure 6-27, complimented by sequential blast photographs in Figure 6-28. Data and comments for each blast test are found in Table 6-17.

Blast 1 (17 psi) and Blast 2 (30 psi) resulted in limited strains deformations and elastic strains in the high-strength tension steel bars. Minimal superficial damage with corresponding support rotations of 0.31° and 0.57° were recorded for these blasts. All cracking remained hairline.

Blast 3 (50 psi) data was lost because of a malfunction of the data acquisition system, as such no data pertaining to strain, displacement and pressure-impulse was recorded. The maximum and residual displacement were approximated through slow motion video analysis. After this blast a total of 12 cracks developed throughout the constant moment region and shear span, however no diagonal crack formation was observed as depicted in Figure 6-28.

Blast 4 (70 psi) imposed considerable mid-span displacement and a resulting support rotation of 1.28° which approaches moderate damage as per current standards. As depicted in Figure 6-28, no spalling occurs as the fibers bridge all visible crack openings.

Blast 5 (90 psi) caused the reinforcement to go into inelastic response and endure near moderate damage with a corresponding support rotation of 1.73° . Minor flexural shear cracks begin to develop in upper and lower shear spans with an average crack width of 1.1 mm.

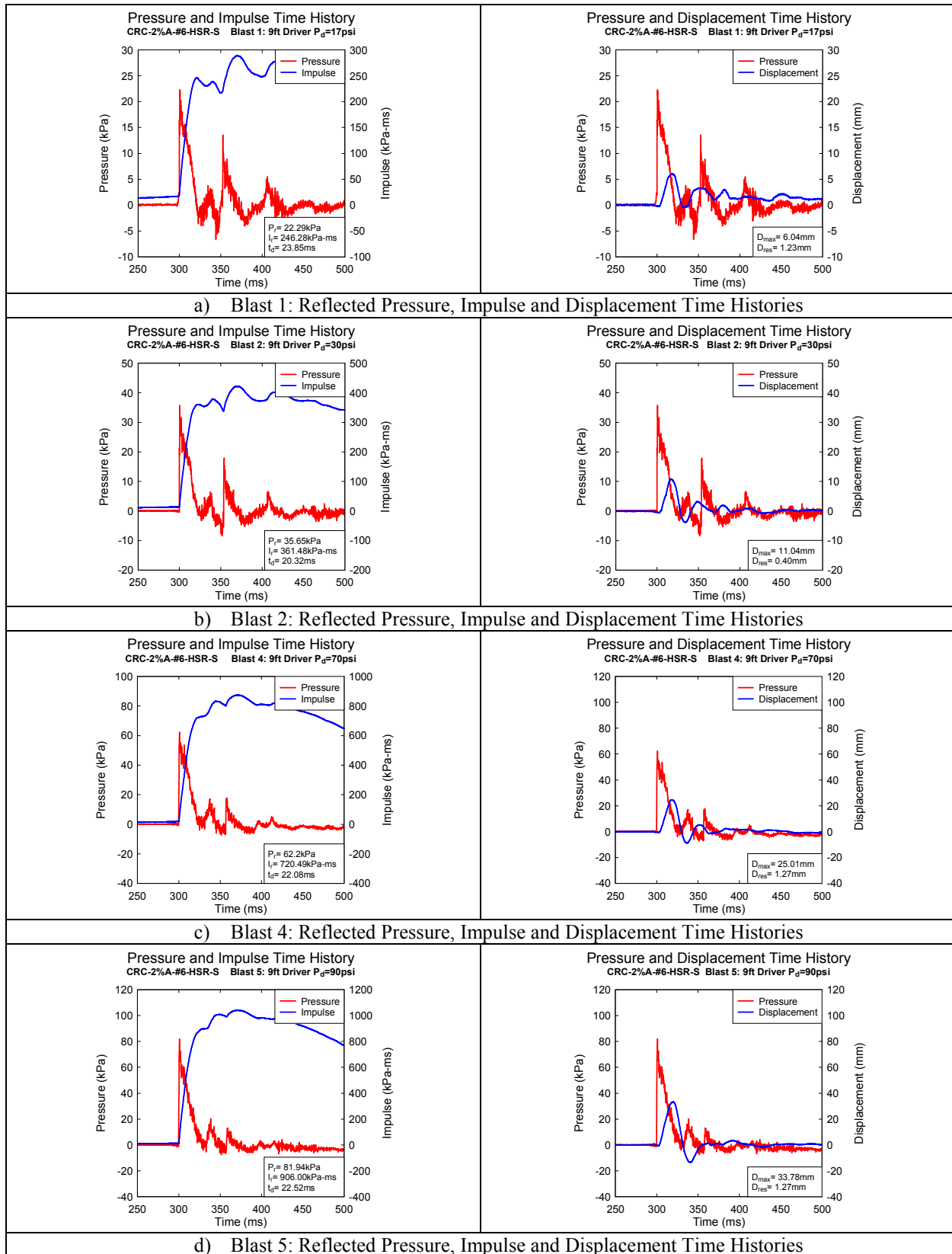
Blast 6 (110 psi) caused the high-strength bars to go into inelastic response and the specimen to endure moderate-heavy damage with a corresponding support rotation of 2.56° . Flexural shear cracks developed in the upper and lower shear spans with an average crack width of 2.0 mm. The beam begins to experienced some pronounced curvature in the flexural region with cracks concentrating near both point load locations and at mid-span.

Blast 7 (120 psi) ultimately caused the specimen to fail in a unique fashion when compared to other specimens. Concrete crushing occurred on the compression face, while a large crack developed through the entire depth of the beam. Damage was classified as heavy-hazardous with a support rotation of 5.04° . Despite the damage endured by the specimen little debris and no projectiles were generated upon examination of the high-speed video.

Table 6-17 CRC-2%A-#6-HSR-S Acquired Blast Data

Blast #	δ_{max} (mm)	Support Rotation (°)	δ_{res} (mm)	Peak Strain ($\mu\text{mm/mm}$)	Residual Strain ($\mu\text{mm/mm}$)	Comments
1	6.04	0.31	1.23	1167	448	- Specimen in elastic range - Total of 4 visible hairline cracks
2	11.04	0.57	0.4	2498	736	- Specimen remains elastic - 8 visible hairline cracks
3	17.10	0.87	0.58	N/A*	N/A*	- No data recorded - 12 cracks having a mean width of 0.04 mm in the constant moment and shear span - No diagonal cracks so far
4	25.01	1.28	1.27	7450	2924	- Steel reinforcement reaches yield - Diagonal cracks begin to form in shear spans - Mean crack width is 0.08 mm
5	33.78	1.73	1.27	10028	4660	- Upwards of 22 cracks along the beam span have an average width of 1.1 mm - Clear formation of plastic hinge near mid-span with large strains in reinforcement
6	49.82	2.56	16.64	N/A*	N/A*	- Crack at upper point load location has a maximum width of 6 mm
7	98.42	5.04	67.29	N/A*	N/A*	- Failure, with concrete crushing and splitting at mid-span and major longitudinal steel reinforcement elongation - Beam is Heavily damaged

*Data not available



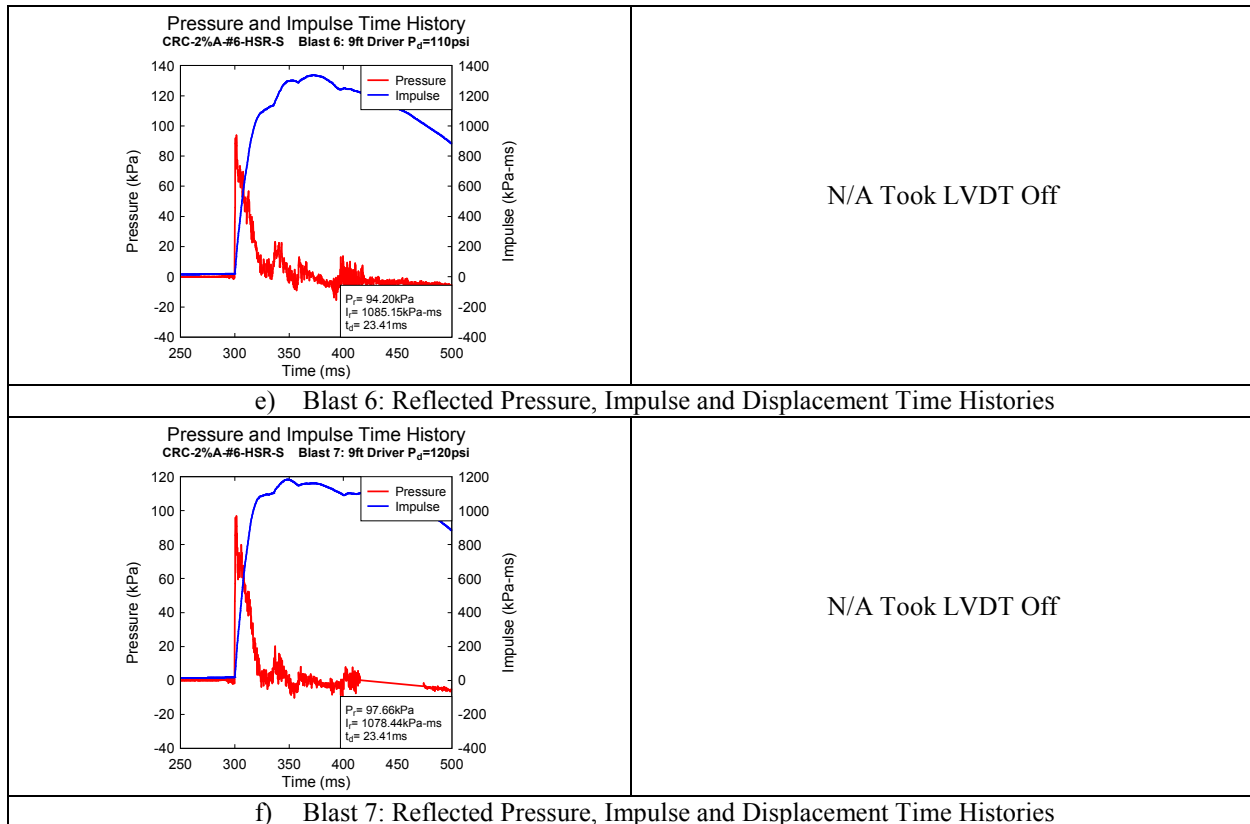


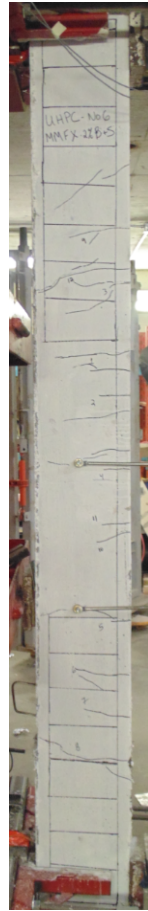
Figure 6-27 CRC-2%A-#6-HSR-S, Recorded Reflected Pressure, Impulse and Displacement for Blast 1 to 7



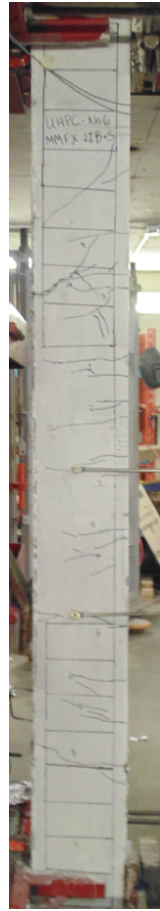
a) Blast 2
(30 psi)



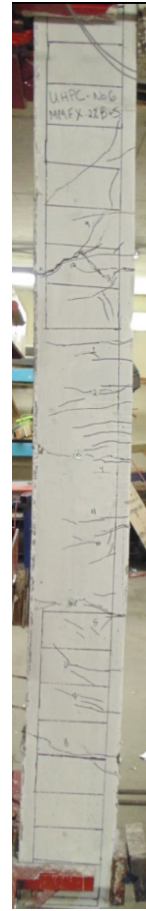
b) Blast 3
(50 psi)



c) Blast 4
(70 psi)



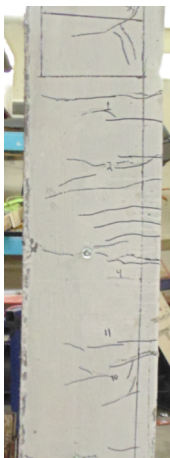
d) Blast 5
(90 psi)



e) Blast 6
(110 psi)



f) Blast 7
(120 psi)



g) Mid-span
cracking
(Blast 6)



h) Mid-span concrete
crushing/concrete splitting
(Blast 7)



i) Mid-span concrete
crushing/concrete splitting
Tension face (Blast 7)

Figure 6-28 CRC-2%B-#6-S, Photographs Showing Results of Blast 1-7

Chapter 7. Discussion of Dynamic Experimental Results

7.1. Chapter Overview

This chapter provides a detailed comparison of the experimental results of the 14 specimens tested under dynamic blast loading. The effects of specific design parameters are examined by comparing maximum and residual mid-span displacements and corresponding support rotations, debris and fragment generation, crack widths, blast load resistance and failure mechanisms. The parameters investigated are presented in the following sequence:

1) **Effect of Reinforcement Ratio**

Control Series

- o NSC-0%-#4-0 vs. NSC-0%-15M-S vs. NSC-0%-20M-S

UHPC + NSR Series

- o CRC-2%A-#4-0 vs. CRC-2%A-15M-S vs. CRC-2%A-20M-S

UHPC + HSR Series

- o CRC-2%A-#4-HSR-S vs. CRC-2%A-#5-HSR-S vs. CRC-2%A-#6-HSR-S

2) **Effect of Concrete Type**

- o NSC-0%-#4-0 vs. CRC-2%A-#4-0
- o NSC-0%-15M-S vs. CRC-2%A-15M-S
- o NSC-0%-20M-S vs. CRC-2%A-20M-S

3) **Effect of Steel Reinforcement Type**

- o CRC-2%A-15M-S vs. CRC-2%A-#5-HSR-S
- o CRC-2%A-20M-S vs. CRC-2%A-#6-HSR-S

4) **Effect of Fiber Reinforcement Type**

- o CRC-2%A-15M-S vs. CRC-2%B-15M-0

5) **Effect of Fiber Reinforcement Content**

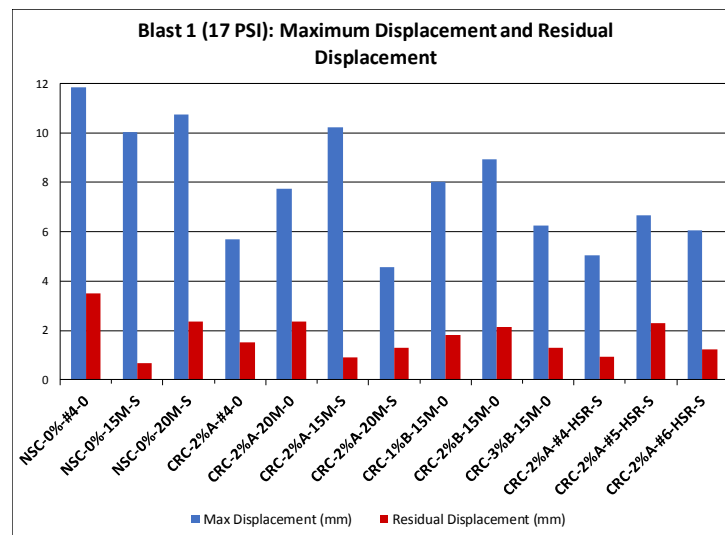
- o CRC-1%B-15M-0 vs. CRC-2%B-15M-0 vs. CRC-3%B-15M-0

6) **Effect of Transverse Reinforcement**

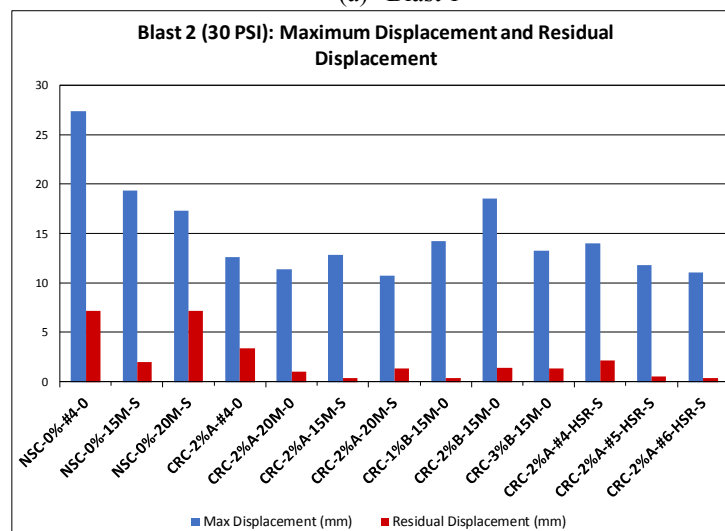
- o NSC-0%-15M-0 vs. NSC-0%-15M-S
- o CRC-2%A-20M-0 vs. CRC-2%A-20M-S

7.2. General Observations

The following section presents a summary and brief overview of the experimental results obtained for blasts 1 through blast 7; summarized in terms of maximum and residual mid-span displacements. Details on the type of failure mechanism experienced by each beam specimen is also provided within a legend contained within each of the bar charts. The bar charts demonstrate the important effect of steel reinforcement ratio in all the series, the advantages of UHPC vs. normal strength concrete, the enhanced blast performance of beams built with high strength reinforcement, the effect of fiber type and content as well as the impact of transverse reinforcement. More detailed discussions are presented in subsequent sections.

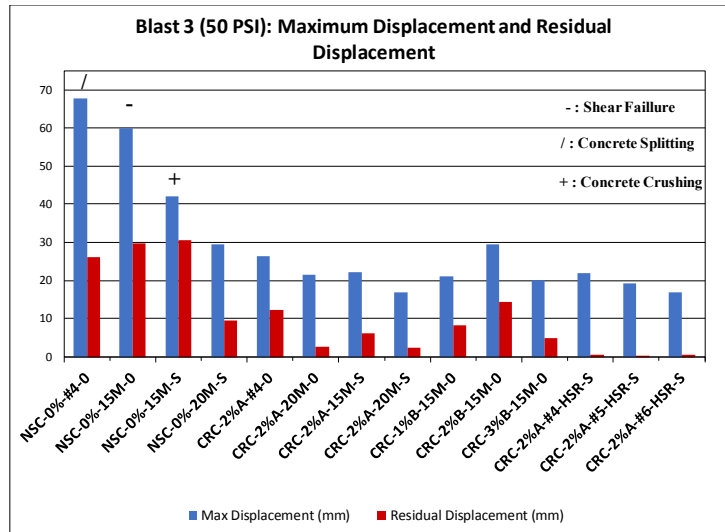


(a) Blast 1

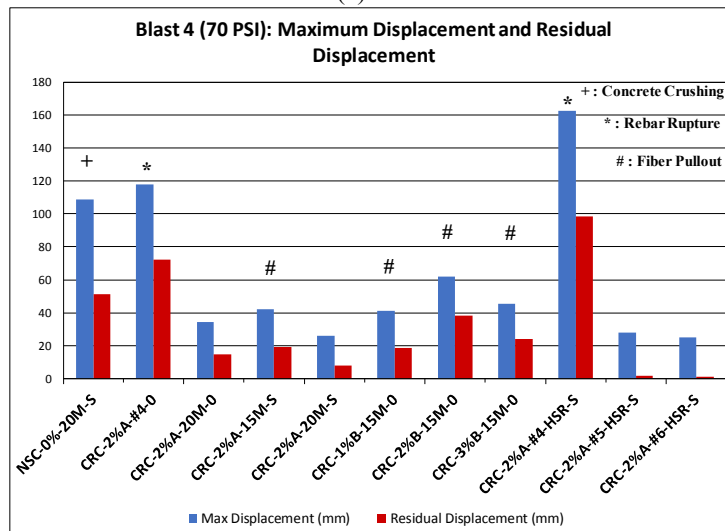


(b) Blast 2

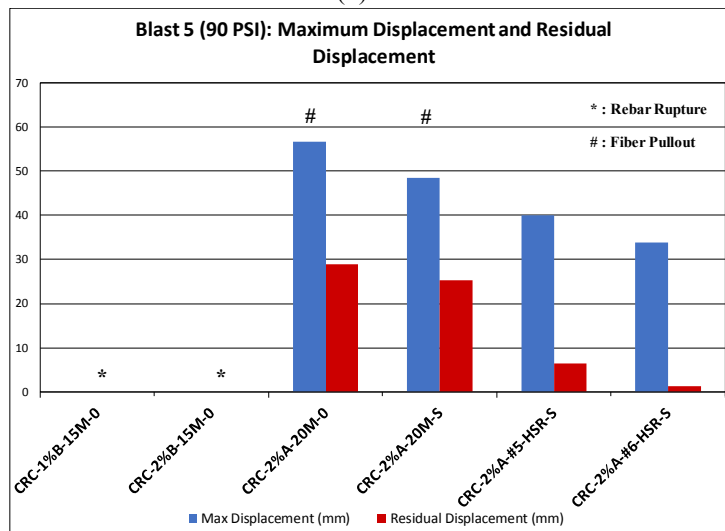
Figure 7-1 Maximum and Residual Displacements (Blast 1 and 2)



(a) Blast 3

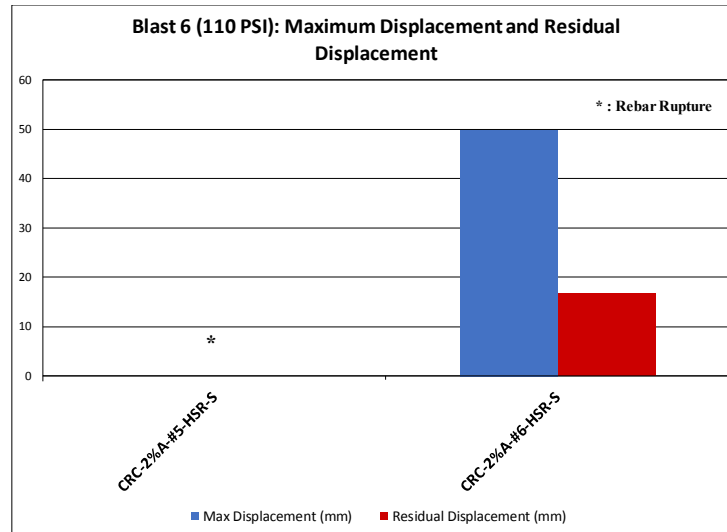


(b) Blast 4

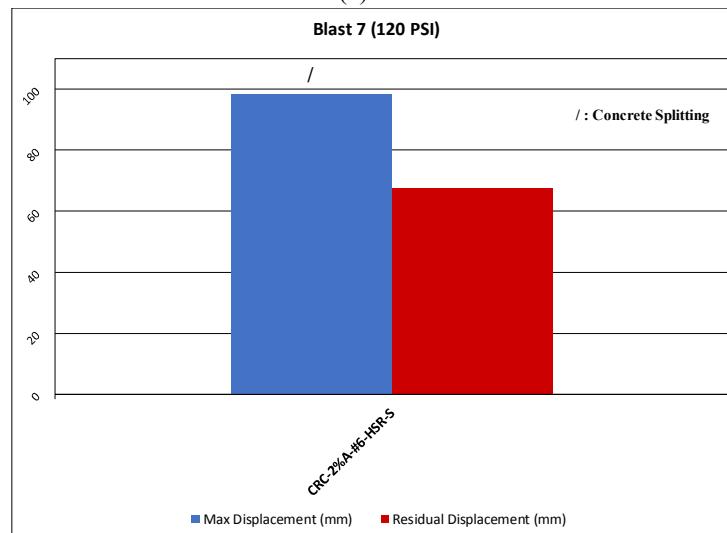


(c) Blast 5

Figure 7-2 Maximum and Residual Displacements (Blast 3, 4 and 5)



(a) Blast 6



(a) Blast 7

Figure 7-3 Maximum and Residual Displacements (Blast 6 and 7)

7.3. Effect of Steel Reinforcement Ratio

The dynamic test results illustrate the important impact of longitudinal steel reinforcement ratio on the blast performance of beams in terms of blast capacity, displacement response and failure mode. The effects of longitudinal steel reinforcement ratio are best assessed by examining the response of the following specimens:

- Control Specimens
 - o NSC-0%-#4-0 vs. NSC-0%-15M-S vs. NSC-0%-20M-S
- UHPC & NSC Specimens
 - o CRC-2%A-#4-0 vs. CRC-2%A-15M-S vs. CRC-2%A-20M-S
- UHPC & HSR Specimens
 - o CRC-2%A-#4-HSR-S vs. CRC-2%A-#5-HSR-S vs. CRC-2%A-#6-HSR-S

7.3.1. Effect of Steel Reinforcement Ratio: Control Series

Three specimens from the control series are used to study the effect of longitudinal steel reinforcement ratio on the blast performance; NSC-0%-#4-0 ($\rho=1.0\%$), NSC-0%-15M-S ($\rho=1.5\%$) and NSC-0%-20M-S ($\rho=2.4\%$). All specimens are designed using normal strength concrete and normal strength reinforcement. Specimen NSC-0%-#4-0 was constructed without transverse reinforcement, whereas NSC-0%-15M-S and NSC-0%-20M-S have stirrups spaced at 100mm o/c throughout the shear span. Photographs illustrating the failure modes experienced by each specimen are found in Figure 7-4, followed by bar chart comparisons of maximum and residual displacements, along with displacement time histories at selected blasts in Figure 7-5.

The results obtained in blast 1 tested the specimens within the elastic range and therefore the effect of reinforcement ratio does not show a clear trend at this load intensity. Results for maximum and residual displacement at blast 2 (30 psi) (Figure 7-5a) present a nice trend with maximum displacement reducing almost linearly with reinforcement ratio. The maximum displacement experienced by NSC-0%-20M-S ($\delta_{\max}=17.34$ mm) is decreased by 37% and 11% from NSC-0%-#4-0 ($\delta_{\max}=27.39$ mm) and NSC-0%-15M-S ($\delta_{\max}=19.32$ mm), respectively. Blast 3 (50 psi) imposed sufficient force as to cause failure in two of the three specimens. NSC-0%-#4-0 experienced mid-span concrete crushing, resulting in a support rotation of $\theta=3.47^\circ$ ($\delta_{\max}=67.7$ mm), which corresponds to “moderate – heavily damage” based on current standards (United States of America Department of Defense, 2008). Specimen, NSC-0%-15M-S, experienced a concrete crushing failure at mid-span, with a large concrete segment separating and falling from the beam compression zone. Support rotation at failure was recorded at $\theta=3.07^\circ$ ($\delta_{\max}=59.8$ mm), representing a 13% reduction when compared to the specimen detailed with #4 reinforcement. Specimen NSC-0%-20M-S successfully resisted blast 3, while experiencing only minor support rotation of $\theta=1.52^\circ$ ($\delta_{\max}=51.3$ mm), representing reductions of 56% and 50% when compared to specimens detailed with #4 and 15M rebar, respectively (Figure 7-5b). Specimen NSC-0%-20M-S experienced a severe concrete crushing failure at blast 4 (70 psi) with a recorded support rotation of $\theta=5.56^\circ$, which is considered “heavily damaged” based on current standards.

The effect of higher reinforcement ratio proved to result in smaller residual displacements, with NSC-0%-20M-S having a smaller residual displacement than NSC-0%-15M-S at blast 2, and Blast 3.

Examining the failure modes in Figure 7-4, it can be observed that increasing the reinforcement ratio from $\rho=1.0\%$ (#4 bars) to $\rho=1.5\%$ (15M bars), the failure remains primarily due to concrete crushing. The beams with 15M and 20M bars show similar failure mechanisms, with severe concrete crushing at failure, however failure is delayed from blast 3 to blast 4 as ρ increases from 1.5% to 2.4%.

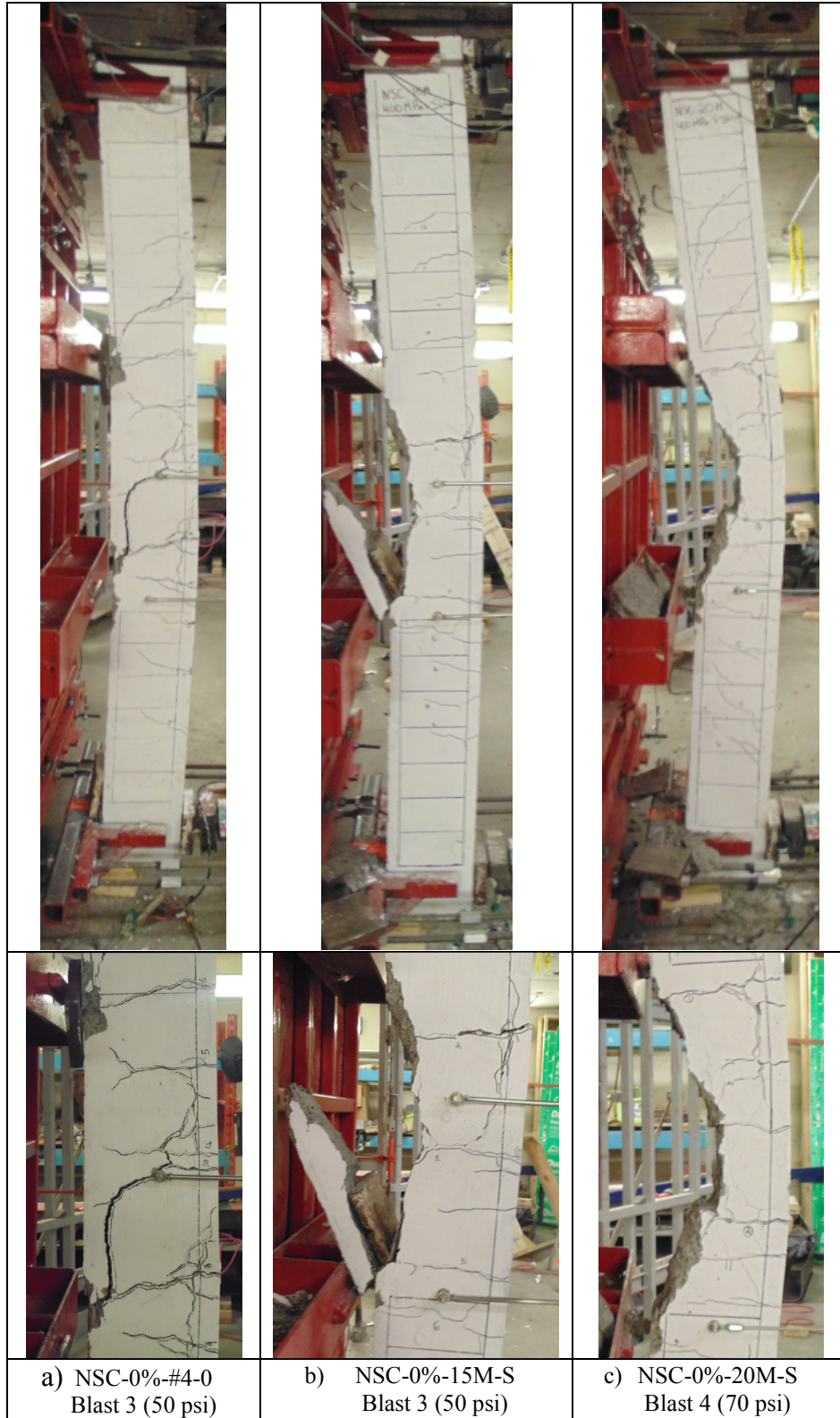


Figure 7-4 Photographs: Effect of Reinforcement Ratio in Control Series

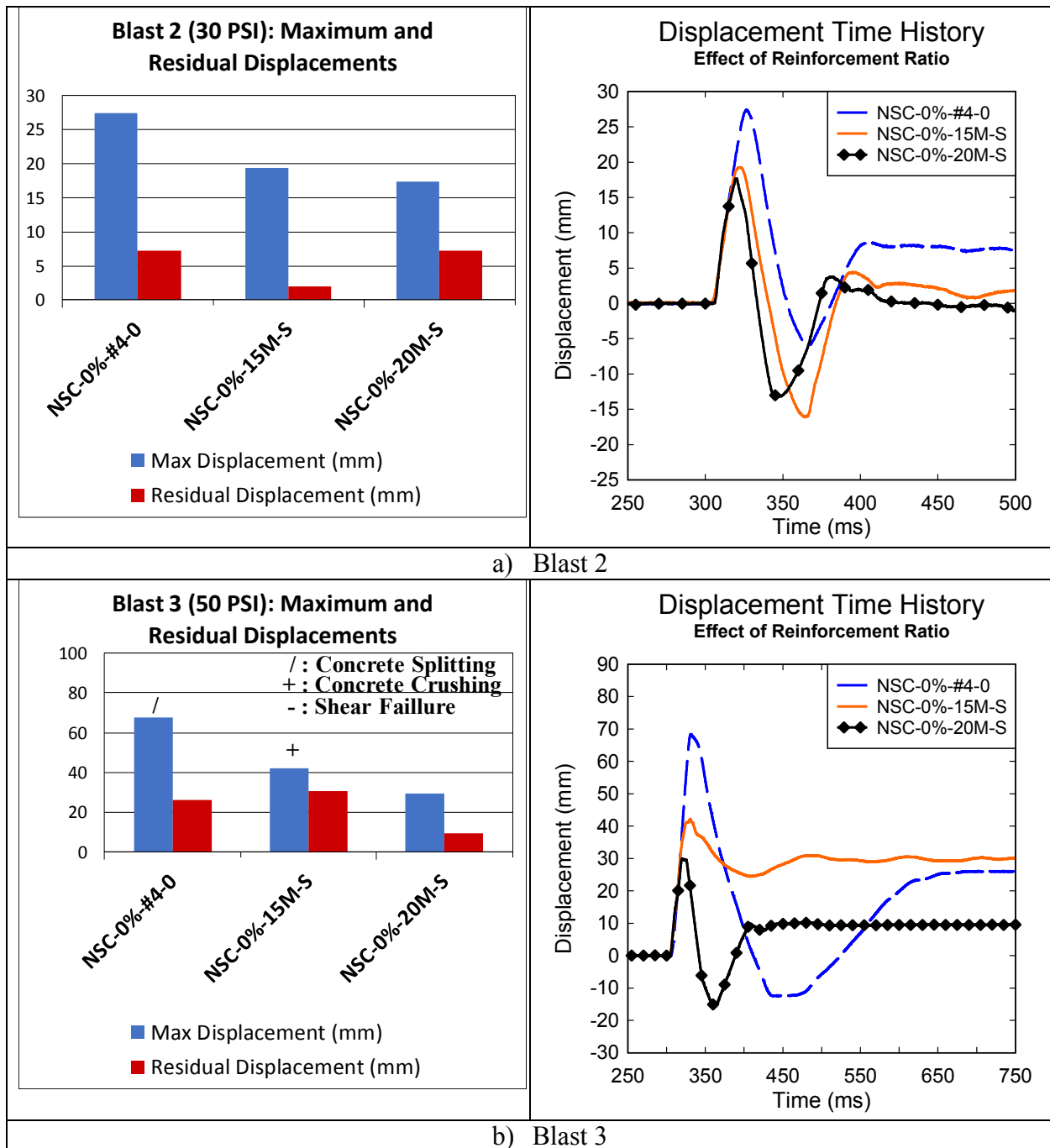


Figure 7-5 Displacement comparison; Effect of Reinforcement Ratio in Control Series

7.3.2. Effect of Steel Reinforcement Ratio: UHPC & NSR Series

Three specimens were selected from the UHPC & NSR series to illustrate the effect of steel reinforcement ratio; CRC-2%A-#4-0 ($\rho=1.0\%$), CRC-2%A-15M-S ($\rho=1.5\%$) and CRC-2%A-20M-S ($\rho=2.4\%$). All three specimens are designed using ultra-high performance concrete (CRC with 2% of type A fibers) and normal strength steel reinforcement. Specimen CRC-2%A-#4-0 was constructed without any transverse reinforcement, whereas CRC-2%A-15M-S and CRC-2%A-20M-S have stirrups spaced at 100mm o/c in the shear spans. Photographs illustrating failure modes for each specimen are found in Figure 7-6, followed by bar chart comparisons of maximum and residual displacements and displacement time-histories at selected blasts in Figure 7-7.

Specimen response to blast 1 (17 psi) remained in the elastic region, hence the effect of reinforcement ratio was not apparent. Blast 2 (30 psi) resulted in the yielding of the #4 and 15M reinforced specimens, however the mid-span and residual displacements still do not present any trends. Results for blasts 3 through 5 present a clearer trend with respect to the effect of reinforcement ratio on displacement control.

Blast 3 (50 psi) imposes such force as to cause a CRC-2%A-#4-0 to experience a maximum displacement of 26.23 mm, while CRC-2%A-15M-S and CRC-2%A-20M-S experienced reductions of 16% and 36%, respectively (Figure 7-7a). Similarly, residual displacements were better controlled, with the 20M specimen showing reductions of 61 % and 80.1% when compared to the 15M and #4 companions.

Blast 4 (70 psi) caused specimen CRC-2%A-#4-0 to experience a rebar rupture. Specimen CRC-2%A-15M-S also failed, however the increase in reinforcement ratio altered the failure mechanism from rebar rupture to fiber pullout with a reduction in mid-span displacement of 64% when compared to the #4 specimen. Specimen CRC-2%A-20M-S survived blast 4 with further reduced mid span displacement by factors of 78% and 32% when compared to #4 and 15M specimens (Figure 7-7b). Residual displacements are also affected, with the 20M specimen showing reductions of 59 % and 89% when compared to the companions with 15M and #4 bars.

Blast 5 (90 psi) resulted in the fiber pullout failure of specimen CRC-2%A-20M-S with a maximum mid-span displacement of 48.48 mm and corresponding support rotation of $\theta=2.49^\circ$ corresponding to “moderate-heavy damage” as per current standards. Slow motion camera analysis reveals that no major fragmentation occurred and only minor dust was generated at failure.

In summary increasing the amount of normal strength reinforcement in the UHPC beams resulted in better control of maximum and residual displacements at equivalent blasts. Increasing the ratio from 1% (#4 bars) to 1.5% (15M bars) was sufficient to prevent rebar rupture at Blast 4, while a further increase to 2.4% (20M bars) delayed failure until Blast 5, resulting in an increase in blast capacity.

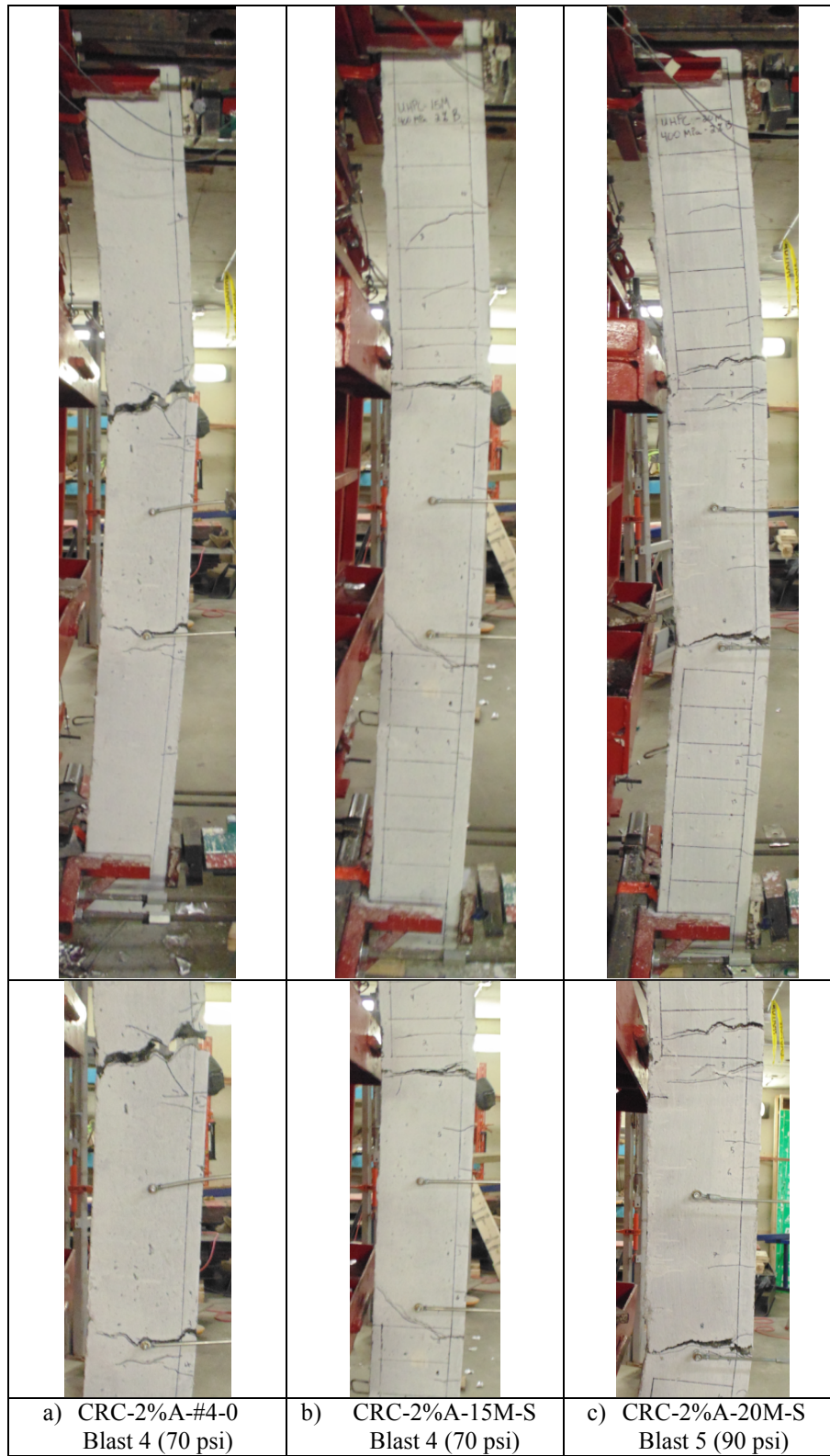


Figure 7-6 Photographs; Effect of Reinforcement Ratio in UHPC & NSR Series

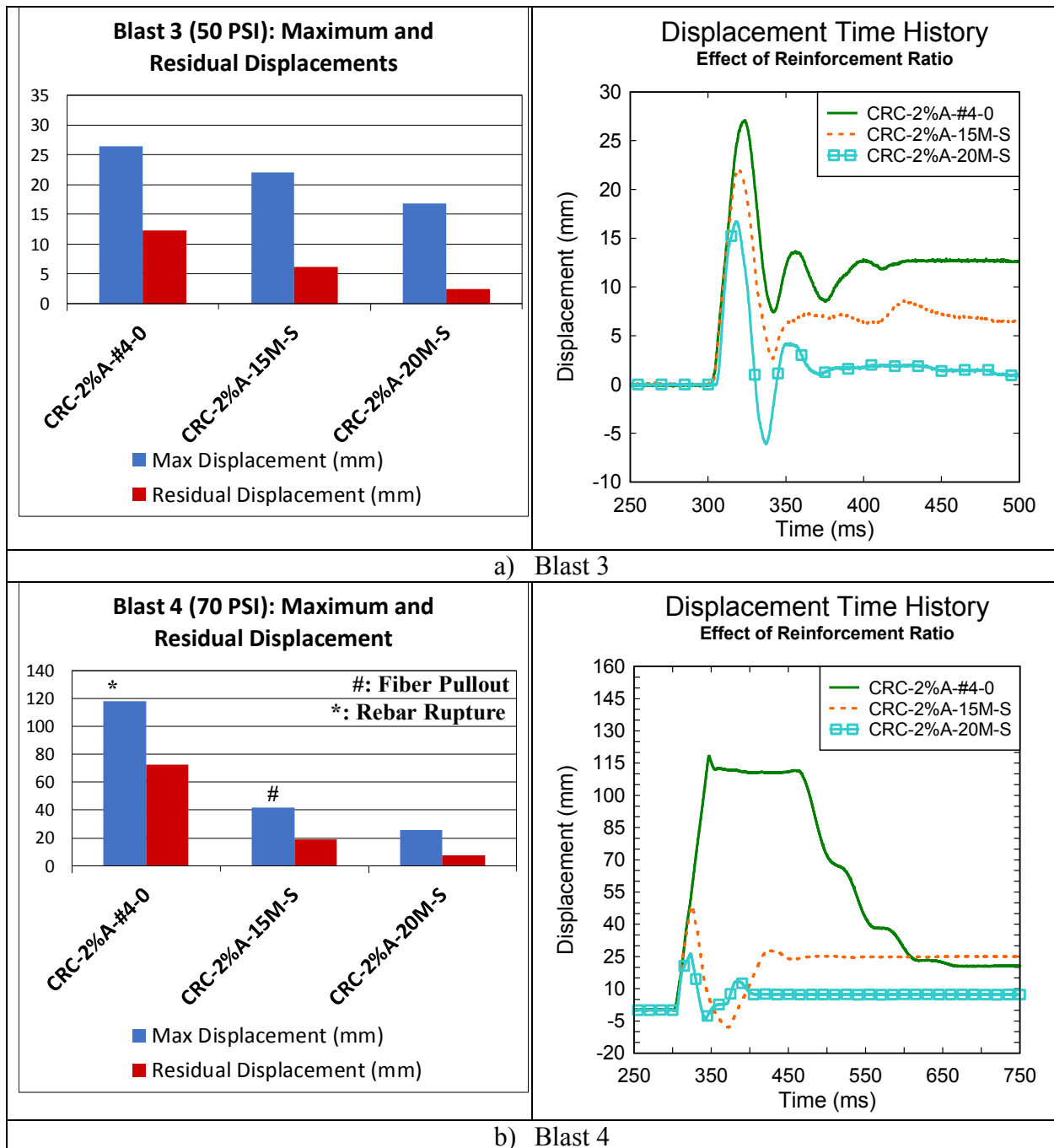


Figure 7-7 Displacement comparison; Effect of Reinforcement Ratio in UHPC & NSR Series

7.3.3. Effect of Steel Reinforcement Ratio: UHPC & HSR Series

Three specimens from the UHPC & HSR series can be used to underline the effect of steel reinforcement ratio; CRC-2%A-#4-HSR-S ($\rho=1.0\%$), CRC-2%A-#5-HSR-S ($\rho=1.5\%$), CRC-2%A-#6-HSR-S ($\rho=2.3\%$). These specimens are designed using ultra-high performance concrete (CRC with 2%A fibers) and high strength reinforcement. All three specimens are detailed with transverse reinforcement spaced at 100mm o/c in the shear spans. Photographs illustrating failure modes experienced by each specimen are found in Figure 7-8, followed by bar charts comparisons of maximum and residual displacements and displacement time-histories at selected blasts in Figure 7-9.

The combination of the two high performance materials, UHPC and high strength steel provided optimal results, and this behavior improved as the reinforcement ratio increased. As in the previous sets Blast 1 (17 psi) does not show a direct with regards to reinforcement ratio as all specimens were still well within their elastic range.

Blast 2 (30 psi) and 3 (50 psi) were still within the elastic response of the specimens with high-strength reinforcement. Nonetheless some trends are observed when comparing maximum displacement results in Figure 7-9 a & b. Blast 2 imposed a maximum displacement of 11 mm on specimen CRC-2%A-#6-HSR-S, representing a 6.3% and 21% decrease when compared to CRC-2%A-#4-HSR-S and CRC-2%A-#5-HSR-S. A similar trend continued through Blast 3 as the #6 specimen ($\rho=2.3\%$) experienced a mid span displacement of 17mm with a corresponding support rotation of ($\theta=0.87^\circ$), whereas the #5 ($\rho=1.5\%$) and #4 ($\rho=1.0\%$) specimens incurred maximum mid-span displacement of 19.15 mm ($\theta=0.98^\circ$) and 21.9 mm ($\theta=1.12^\circ$), respectively.

Blast 4 (70 psi) resulted in the flexural failure of CRC-2%A-#4-HSR-S by means of rebar rupture, whilst specimens CRC-2%A-#5-HSR-S and CRC-2%A-#6-HSR-S survived this blast and sustained maximum mid-span displacements of 28.12 mm ($\theta=1.44^\circ$) and 25.01 ($\theta=1.28^\circ$). Blast 5 (90 psi) imposed a maximum mid-span displacement of 40 mm on specimen CRC-2%A-#5-HSR-S while specimen CRC-2%A-#6-HSR-S reduced maximum displacement by 16% (Figure 7-9d).

Blast 6 (110 psi) proved to be detrimental to specimen CRC-2%A-#5-HSR-S, as it resulted in rupture of the #5 longitudinal reinforcement. Further examination of the high-speed video shows the beam completely breaking in half at mid-span, resulting in a hazardous failure mode. Conversely, specimen CRC-2%A-#6-HSR-S survived blast 6, with a maximum mid-span displacement of 49.82 mm ($\theta=2.56^\circ$) characterized as “moderately damaged”. Failure of this beam would occur at blast 7 (120 psi), with the specimen experiencing a concrete splitting failure as some concrete crushing occurs amidst major steel reinforcement elongation and very pronounced cracking throughout the flexural region and shear span. The maximum support rotation experienced by the specimen was of $\theta=5.04^\circ$, considered “heavily damaged”.

In summary, the increase in reinforcement ratio proved to have considerable impact on the response of the specimens, as the data depicts significant reduction in maximum mid-span displacements (hence support rotations) as the amount of high-strength reinforcement in the UHPC

beams is increased. Moreover, the high-strength reinforcement ratio had an important effect on blast capacity and failure mode. Increasing the ratio from $\rho = 1.0\%$ (#4 bars) to $\rho = 1.5\%$ (#5 bars) delayed failure from blast 4 to blast 6, although the reduced strain capacity of the high-strength bars results in rebar rupture. Increasing the ratio further to $\rho = 2.3\%$ in the #6 specimen not only delayed the failure to blast 7, but also altered the failure and prevented rebar rupture.

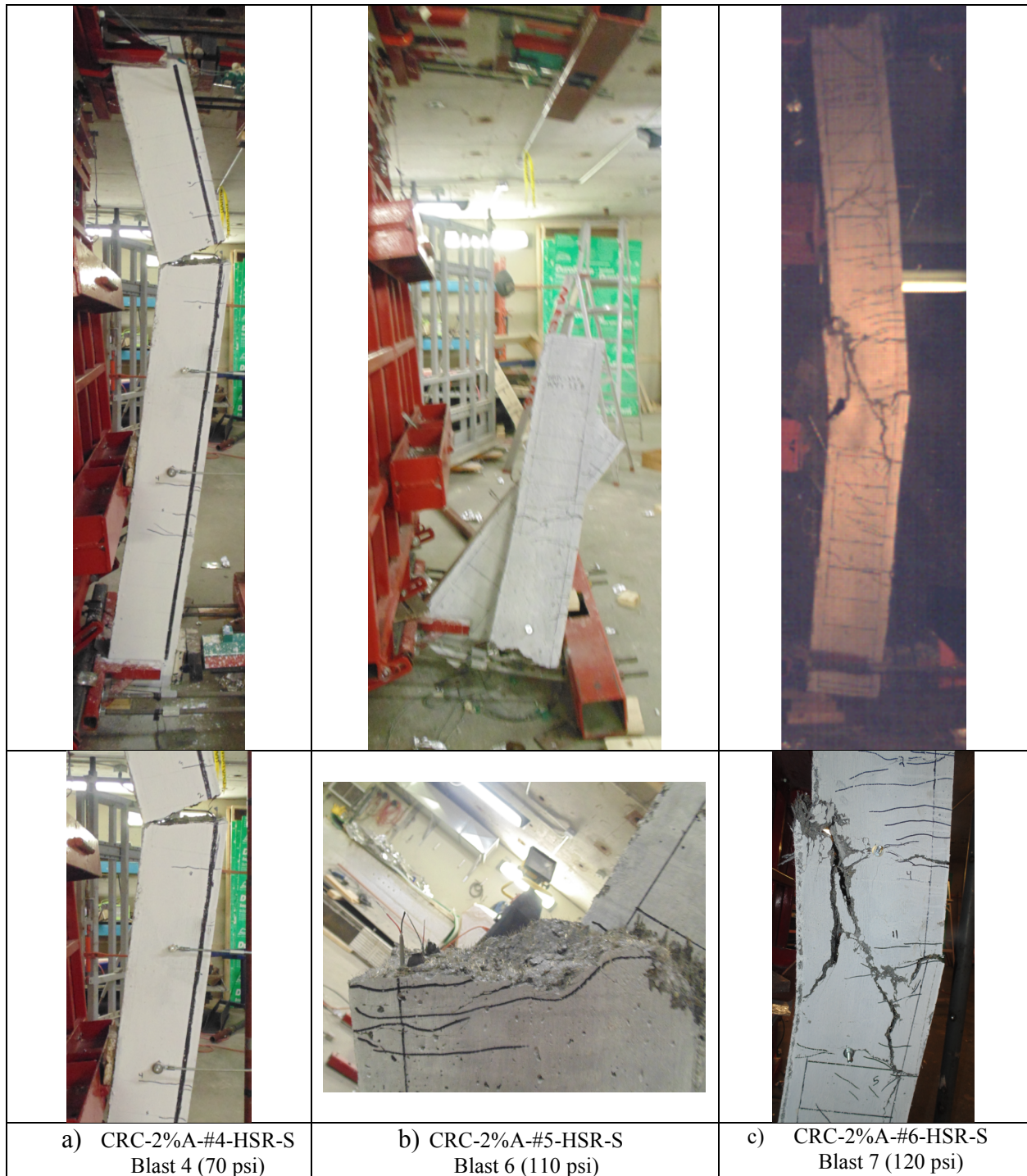
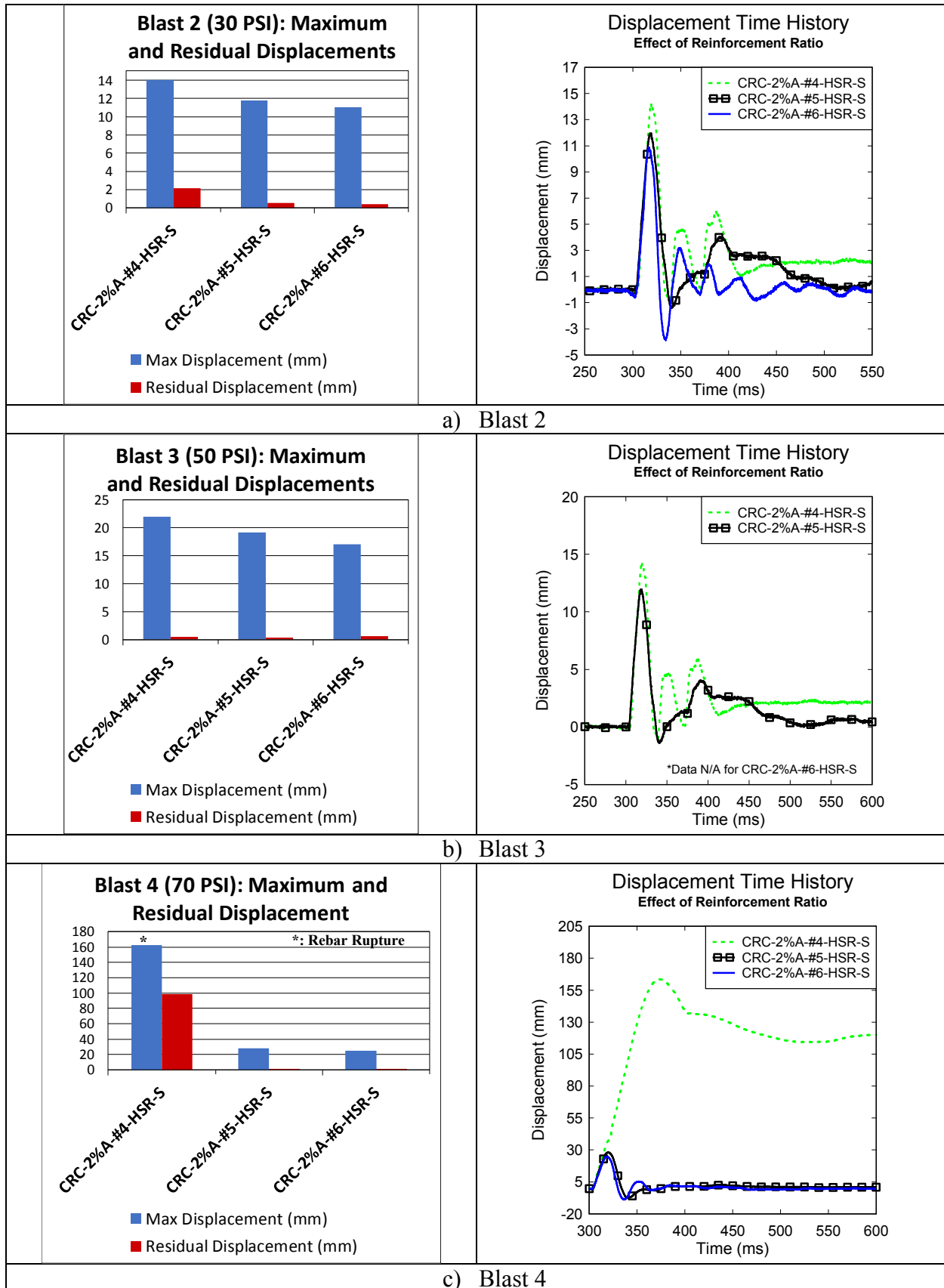


Figure 7-8 Photographs; Effect of Reinforcement Ratio in UHPC & HSR Series



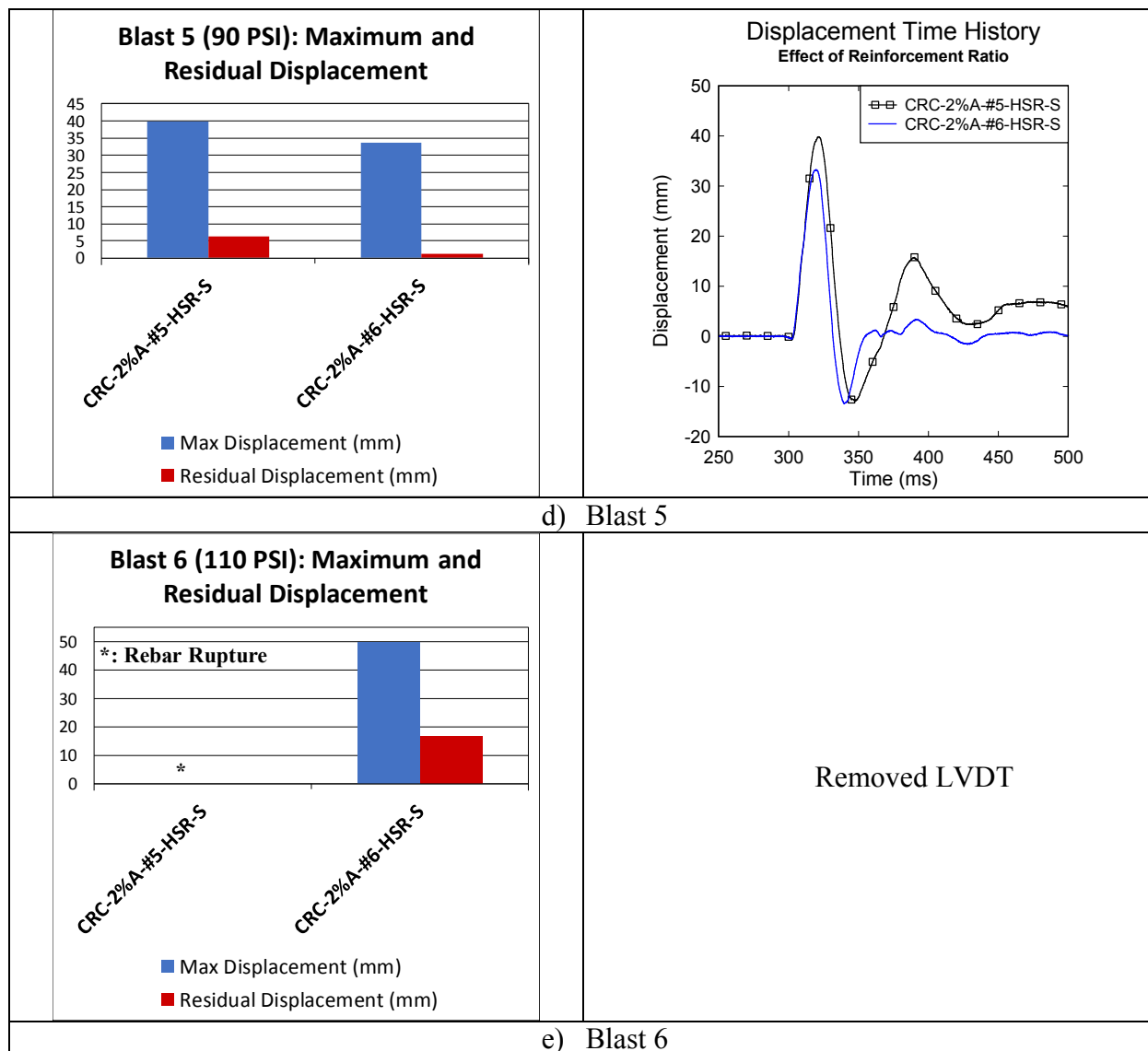


Figure 7-9 Displacement Comparisons; Effect of Reinforcement Ratio in UHPC & HSR Series

7.4. Effect of Concrete Type

Ultra-high performance concrete has enhanced compressive and tensile resistance when compared to conventional normal strength concrete. The provision of fibers also increases toughness and ductility of the material. The effect of UHPC on blast response is assessed in the current study by examining the response of the following companions sets with #4, 15M and 20M bars:

- Control vs. UHPC beams with #4 bars: NSC-0%-#4-0 vs. CRC-2%A-#4-0
- Control vs. UHPC beams with 15M bars: NSC-0%-15M-S vs. CRC-2%A-15M-S
- Control vs. UHPC beams with 20M bars: NSC-0%-20M-S vs. CRC-2%A-20M-S

The beams in each set had identical reinforcement properties, with exception of the concrete type (normal strength concrete vs. CRC with 2%A fibers). Photographs illustrating the failure modes of each specimen are found in Figure 7-10, followed by bar chart comparisons of maximum and residual displacements and displacement-histories in Figure 7-11 through Figure 7-12. Lastly, Figure 7-13, presents a comparison of the resulting generated debris and fragmentation of specimens upon their respective failure blast.

While Blast 1 (17 psi) was meant to investigate the elastic response of the specimens, the #4 and 20M reinforced specimens designed with UHPC present reductions in maximum mid-span displacement of 51.8% and 57.8%, when compared to the companions with conventional concrete (Figure 7-11a). The 15M companions present almost identical displacements at this blast.

Blast 2 (30 psi) results shows reduced displacements for the UHPC beams in all three companion sets. Reductions in maximum mid-span displacements are in the order of 54%, 33% and 38% for the #4, 15M and 20M UHPC companions as displayed in Figure 7-11b. Similarly, the UHPC beams show reductions in residual displacements.

Blast 3 (50 psi) imposed adequate force as to specimen NSC-0%-#4-0 to suffer a concrete splitting failure and specimen NSC-0%-15M-S to fail due to concrete crushing as depicted in Figure 7-10. The companion beams designed with UHPC survived blast 3, with reduced maximum mid-span and residual displacements. Comparing the 20M set, conventional concrete specimen NSC-0%-20M-S experienced a maximum displacement 176% greater than CRC-2%A-20M-S designed with UHPC. The use of UHPC also effectively eliminated residual deformations (Figure 7-12a).

Blast 4 (70 psi) resulted in the flexural failures of CRC-2%A-#4-S and CRC-2%A-15M-S by means of rebar rupture and fiber pullout, respectively, representing an increase in blast capacity when compared to their companions with normal strength concrete. Specimen NSC-0%-20M-S also failed with the mode of failure associated with concrete crushing. Its UHPC companion, CRC-2%A-20M-S, survives blast 4 with maximum and residual mid-span displacements of 25.86mm and 7.77mm, with corresponding maximum support rotation of $\theta=1.33^\circ$. The deformation presents a reduction of 76.3% when compared to the NSC companion (Figure 7-12b).

Blast 5 (110 psi) resulted in failure of specimen CRC-2%A-20M-S due to fiber pullout as depicted in Figure 7-10. High-speed video shows debris and fragmentation was limited to dust as the fibers held the concrete together, even at failure. Maximum and residual displacements were recorded to be 48.48mm and 25.3mm with a corresponding support rotation of ($\theta=2.49^\circ$), which can be described as “moderate damage” as per current standards.

In summary, the results demonstrate clear benefits associated with UHPC in beams tested under blast loads. First displacements at equivalent blasts are better controlled when comparing the companion beams built with UHPC and conventional concrete. Moreover, the use of UHPC results in considerable increase in blast capacity in all three sets. Failure mode is also altered due to the

increased toughness of UHPC, with crushing and spalling eliminated. Nonetheless for beams with low amounts of reinforcement failure can occur due to rebar rupture, as observed in the UHPC beams with #4 bars (indeed, the use of larger reinforcement ratio is better suited to match the high compressive strength of UHPC). An additional advantage witnessed in the UHPC specimens is the considerable reduction in debris and fragmentation generation as the fibers hold the concrete together even at failure in contrast to the normal strength concrete specimens (NSC-0%-15M-S & NSC-0%-20M-S) which produced significant fragmentation due to concrete crushing at failure.

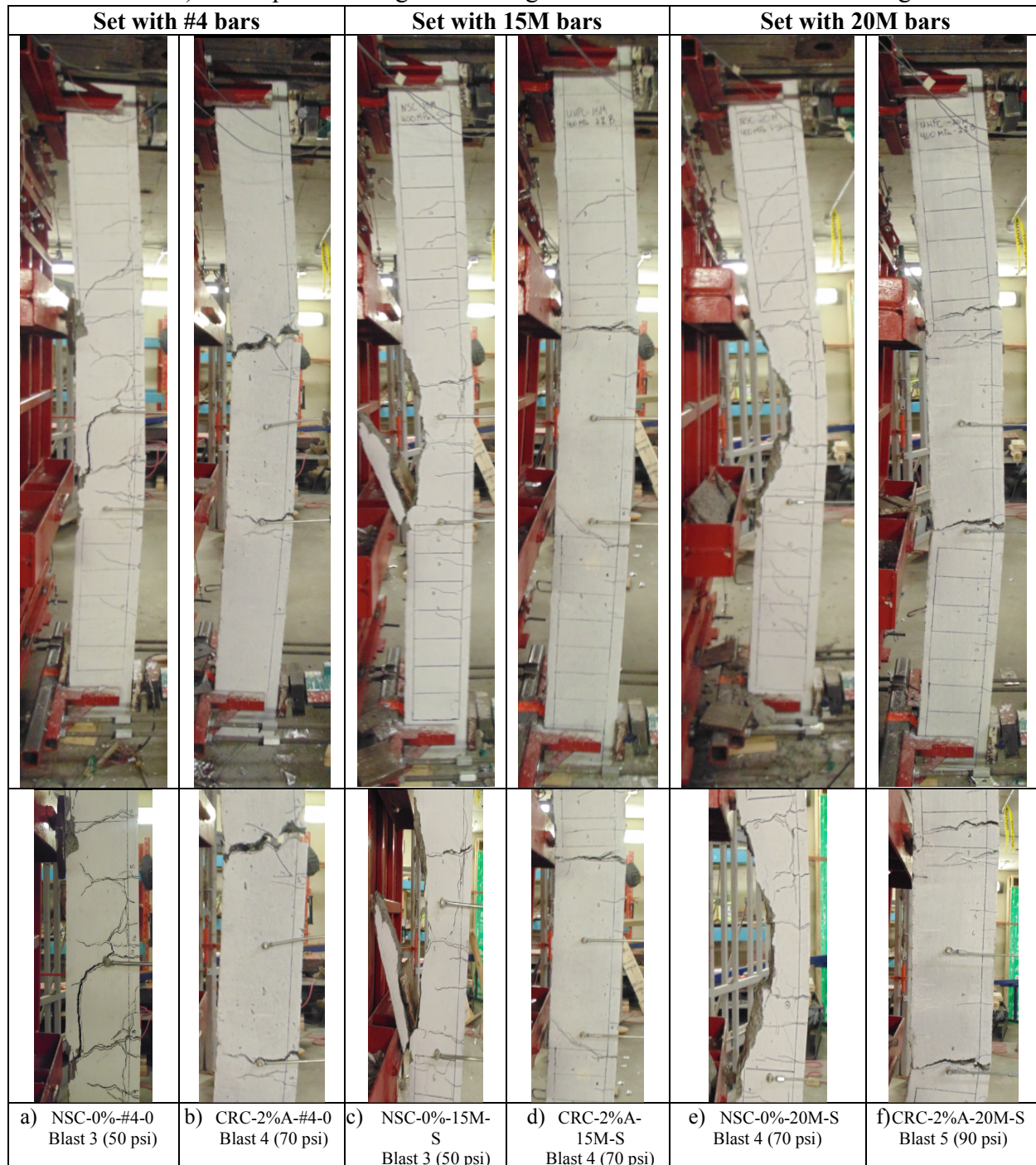


Figure 7-10 Photographs; Effect of Concrete Type

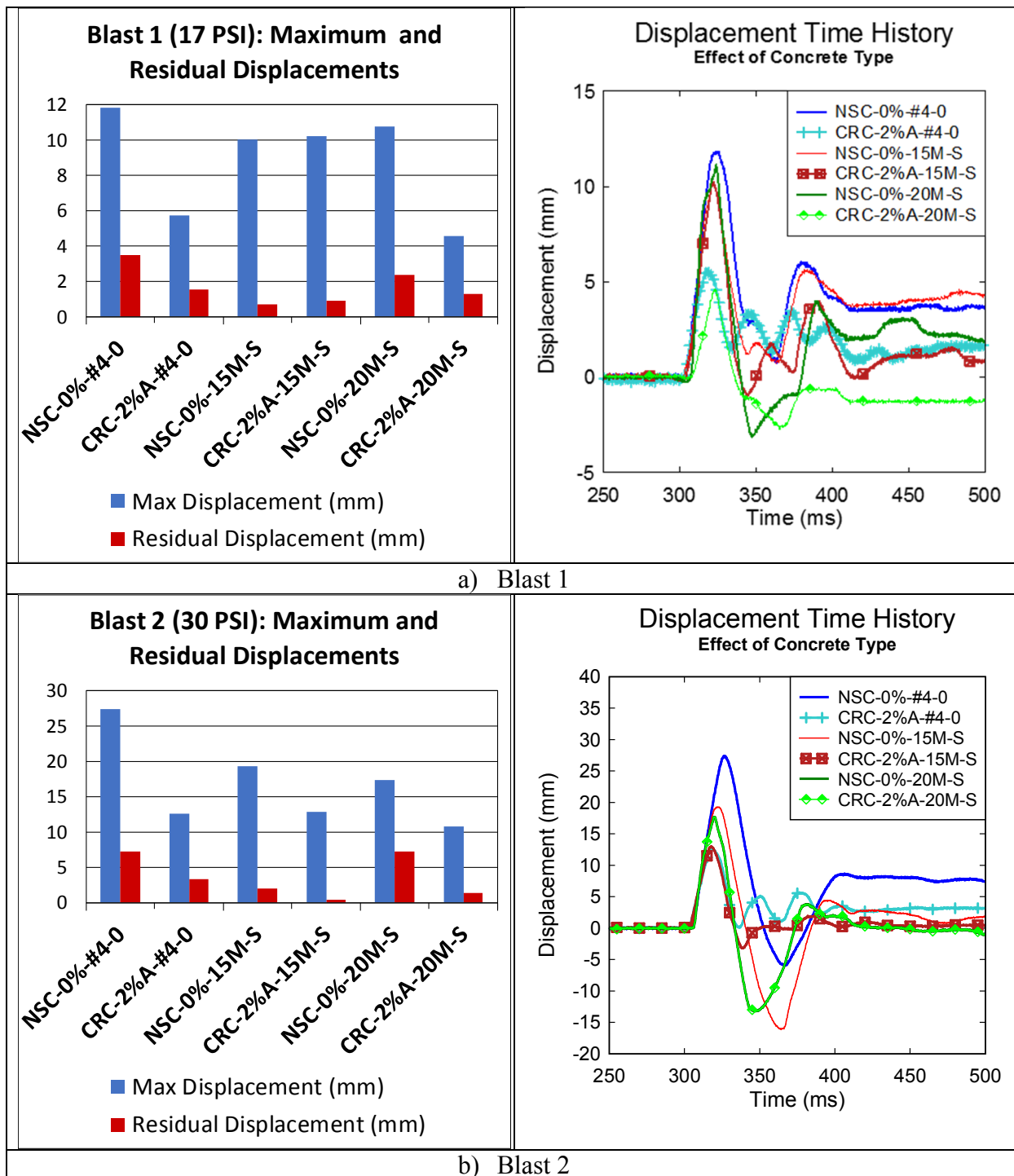


Figure 7-11 Displacements Comparisons; Effect of Concrete Type (1/2)

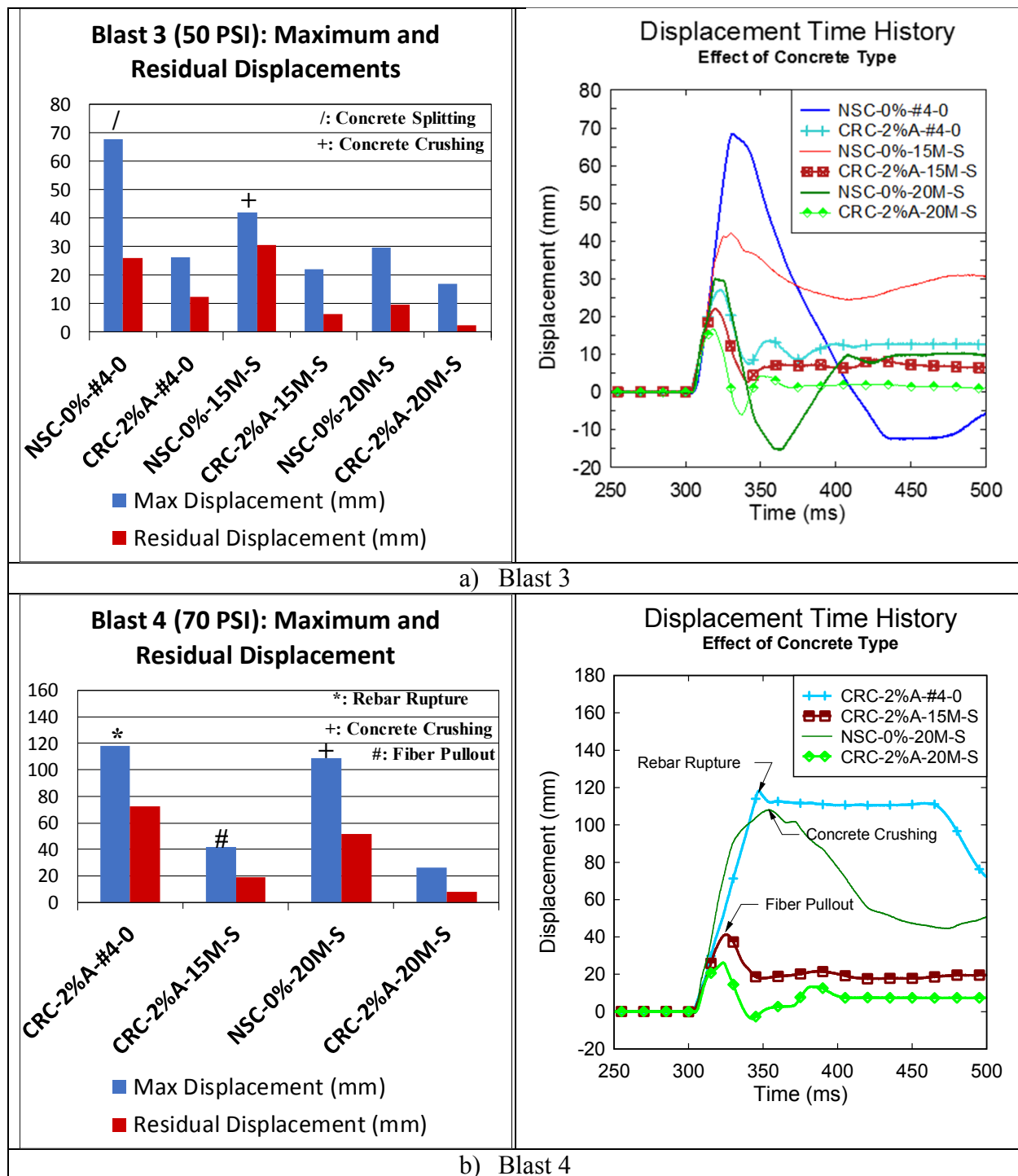


Figure 7-12 Displacements Comparisons; Effect of Concrete Type (2/2)



Figure 7-13 Debris and Fragmentation Control: Effect of Concrete Type

7.5. Effect of Longitudinal Steel Reinforcement Type

The effects steel reinforcement type is assessed by examining the response of the following companion specimens built with UHPC and normal-strength vs. high-strength steel:

- UHPC beams with #4 NSR vs. #4 HSR: CRC-2%A-#4-0 vs. CRC-2%A-#4-HSR-S
- UHPC beams with 15M NSR vs. #5 HSR: CRC-2%A-15M-S vs. CRC-2%A-#5-HSR-S
- UHPC beams with 20M NSR vs. #6 HSR: CRC-2%A-20M-S vs. CRC-2%A-#6-HSR-S

The beams in each companion set are nominally identical, except for the reinforcement type utilized. The first and second set of beams are designed with #4 ($\rho=1.0\%$) and 15M/#5 ($\rho=1.5\%$) normal strength vs. high strength bars. The third set of beams are designed with 20M normal strength and #6 high strength bars, with a slightly lower reinforcement ratio for the beam with #6 bars ($\rho=2.3\%$ vs. 2.4%). All beams are built with CRC having 2% of fiber A, while transverse reinforcement is provided in all beams except CRC-2%A-#4-0. Photographs illustrating failure modes experienced by each specimen are found in Figure 7-14, followed by bar chart and time-history comparisons of displacements (Figure 7-15 & Figure 7-16).

Early loading at Blast 1 (17 psi) does not show any clear trends. Maximum displacements at Blast 2 (30 psi) display a narrow range for maximum mid-span displacements in all specimens (between 10mm and 14mm). However, since response remains elastic, the residual displacements for #4 and #6 specimens with high strength reinforcement are considerably lower than those of the companions with #4 and 20M normal strength reinforcement (with reductions of 58% and 248%, as shown in Figure 7-15a).

As depicted in Figure 7-15b, the maximum mid-span displacements at Blast 3 (50 psi) are reduced by factors of 20% and 15% for the specimens with high-strength bars in the #4 and #5 companion sets, while the 20M normal strength specimen and #6 high strength specimen present almost identical maximum mid-span displacements. As displayed in Figure 7-15, the residual displacements for the beams with high strength MMFX bars are considerably less in all three sets, with reductions of 88.6%, 95.1% and 0.8% for the #4, #5 and #6 beams, respectively.

Blast 4 (70 psi) resulted in the flexural failure of both specimens with #4 bars (CRC-2%A-#4-S and CRC-2%A-#4-HSR-S). The UHPC specimen with 15M normal strength bars (CRC-2%A-15M-S) also failed at this blast due to fiber pullout, while the companion beam with high-strength bars survives this blast intensity. Surviving specimens: CRC-2%A-#5-HSR-S, CRC-2%A-20M-S and CRC-2%A-#6-HSR-S experience maximum mid-span displacements of 28.12mm ($\theta=1.44^\circ$), 25.86mm ($\theta=1.33^\circ$) and 25.01mm ($\theta=1.28^\circ$), respectively (Figure 7-16a). It is noted that the #5 high-strength specimen shows reduction in maximum displacement when compared to the beam with 20M normal-strength bars, despite the lower reinforcement ratio ($\rho=1.5\%$ vs. 2.4%). Residual displacements for the specimens with high-strength bars are also considerably better controlled when compared to the normal strength steel specimen, with reduction of 92.1% and 84.4% for the #5 and #6 beams.

Blast 5 (90 psi) exceeded specimen CRC-2%A-20M-S's capacity, resulting in a fiber pullout at two major cracks corresponding to the point load application points, with maximum and residual mid-span displacements of 48.5mm and 25.3mm. The two surviving specimens CRC-2%A-#5-HSR-S and CRC-2%A-#6-HSR-S, experienced respective maximum and residual mid-span displacements of ($\delta_{max}=40.0\text{mm}$ & $\delta_{res}=6.3\text{mm}$) and ($\delta_{max}=33.7\text{mm}$ & $\delta_{res}=1.2\text{mm}$).

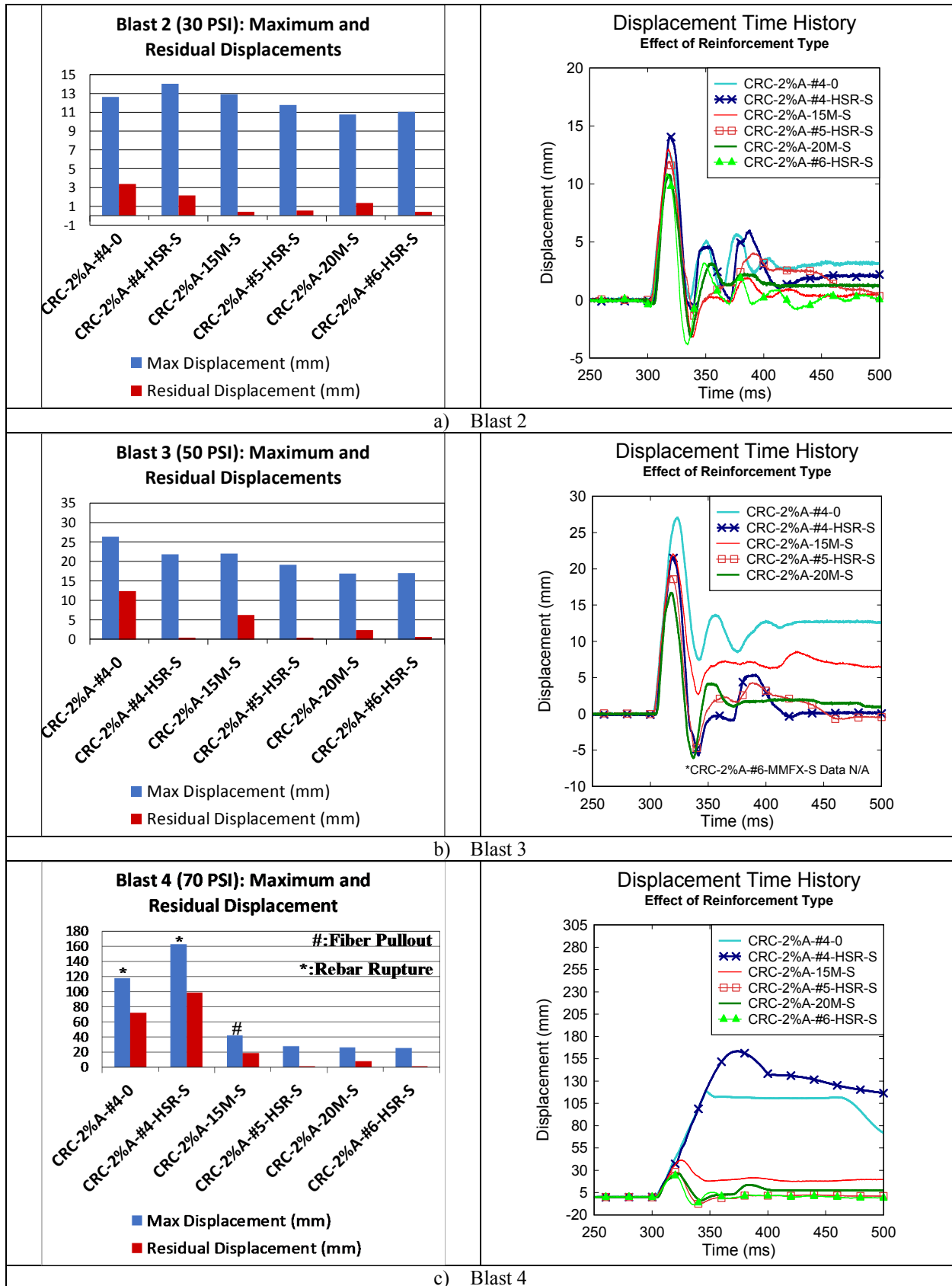
Failure of specimen CRC-2%A-#5-HSR-S would be delayed to Blast 6 (110 psi), with the beam experience a rebar rupture failure mechanism, completely severing in half as depicted in Figure 7-14. Despite this, the blast capacity of the beam is increased when compared to its 15M companion with normal-strength reinforcement.

Use of high-strength bars also increases the blast capacity of beam CRC-2%A-#6-HSR-S, which survived blast 6. Failure of this beam occurs at Blast 7 (120 psi) by means of concrete splitting. As depicted in Figure 7-14, the #6 high strength reinforced specimen endured this intense blast with damage and cracking in the constant moment region and shear spans, but without rebar rupture, with a recorded maximum mid-span displacement of 98.42mm.

In summary, the results demonstrate important benefits associated with the combined use of UHPC and high-strength MMFX reinforcement. First, the use of high-strength steel improved control of maximum displacements and support rotations at equivalent blasts. Implementing high strength reinforcement also presents great advantages in terms of minimizing the residual displacements following a blast, as in blasts 1-7. As seen in #5 and #6 companion sets, the high strength reinforcement also delayed failure to more intense blast shockwaves. Using high strength reinforcement allows for the full capacity of the UHPC concrete to be utilized as depicted in the case of blast 7 of specimen CRC-2%A-#6-HSR-S where both the concrete and steel reinforcement were exerted to their limits at failure.

#4 NSR vs. #4 HSR		15M NSR vs. #5 HSR		20M NSR vs. #6 HSR	
a) CRC-2%A-#4-S Blast 4 (70 psi)	b) CRC-2%A-#4-HSR-S Blast 4 (70 psi)	c) CRC-2%A-15M-S Blast 4 (70 psi)	d) CRC-2%A-20M-S Blast 5 (90 psi)	e) CRC-2%A-#5-HSR-S Blast 6 (110 psi)	f) CRC-2%A-#6-HSR-S Blast 7 (120 psi)

Figure 7-14 Photographs; Effect of Reinforcement Type



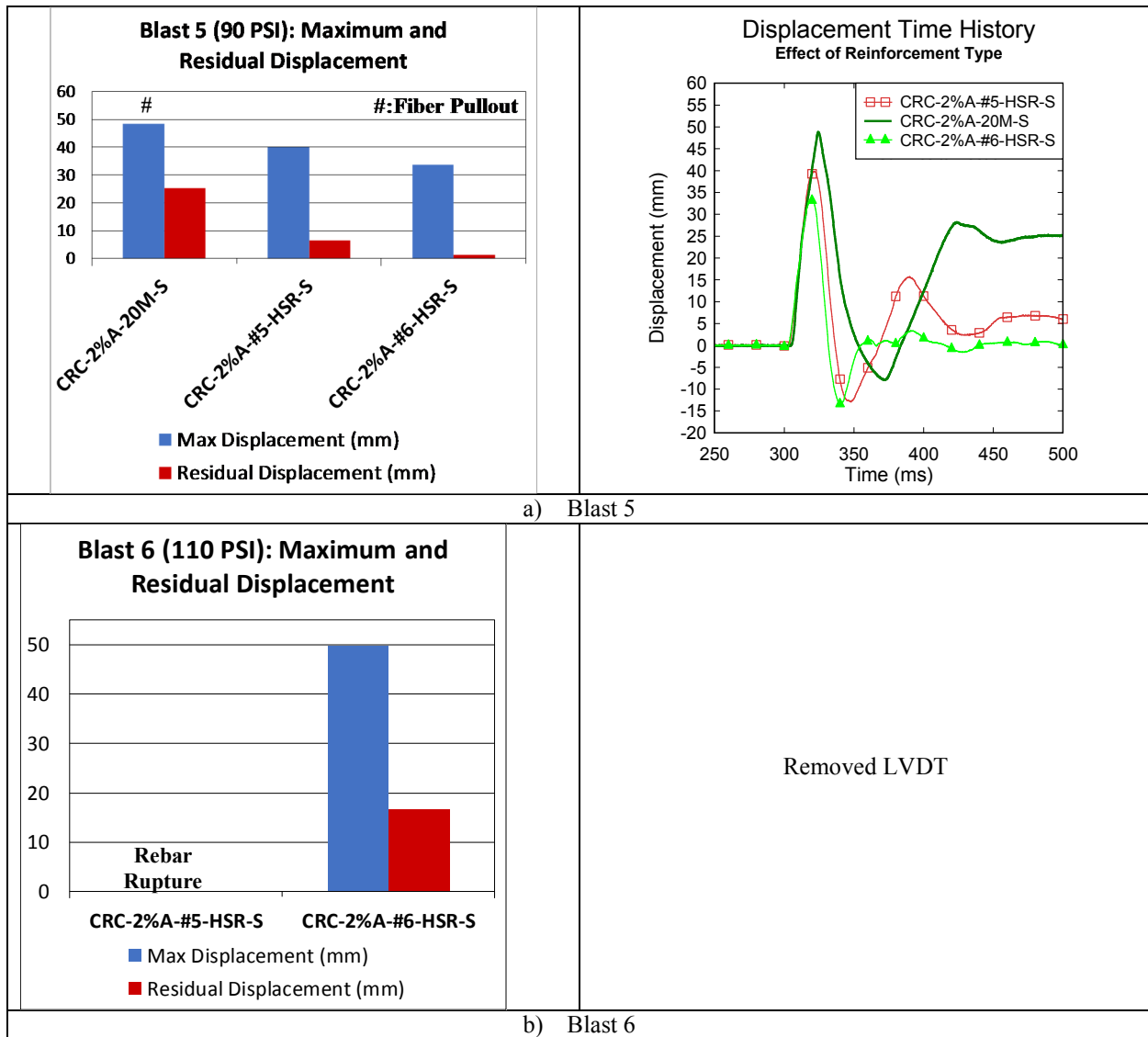


Figure 7-16 Displacement Comparisons; Effect of Steel Reinforcement Type (2/2)

7.6. Effect of Fiber Properties

Two types of steel fibers were selected to investigate the effect of fiber type on blast performance of UHPC beams. Both fibers are straight (smooth) and have the same length of 13 mm, however fiber type A has an aspect ratio of 62 and a tensile strength of 2750 MPa, whereas fiber type B has an aspect ratio of 43 and a tensile strength of 3150 MPa (fiber properties can be found in Table 3-5). The effect of fiber type is best assessed by examining the response of the following specimens:

- o CRC-2%A-15M-S vs. CRC-2%B-15M-0

The beams have similar properties in that they are built with 15M longitudinal bars and UHPC, but the CRC mixtures in the beams contain 2% of type A and type B fibers, respectively. Specimen CRC-2%A-15M-S is also detailed with transverse reinforcement spaced at 100mm in the shear span, whereas specimen CRC-2%B-15M-0 does not have any transverse reinforcement. Photographs illustrating failure modes for the two specimen are found in Figure 7-17, followed by bar chart and time-history comparisons of displacements in Figure 7-18.

Blast 1 (17 psi), was meant to investigate the elastic response of the specimens, hence the maximum mid-span displacements experienced by both specimens are similar. Blast 2 (30 psi) imposed sufficient force as to cause yielding of the longitudinal reinforcement in both specimens. The maximum mid-span deflections for specimens CRC-2%A-15M-S and CRC-2%B-15M-0 are $\delta_{max}=12.89\text{mm}$ & $\delta_{max}=18.52\text{mm}$, representing a 30.1% reduction in terms of maximum deflection for the beam with type A fibers.

Blast 3 (50 psi) caused specimen CRC-2%A-15M-S to experience a maximum and residual mid-span displacement of 22.07mm and 6.21mm, which are inferior to those experienced by CRC-2%B-15M-0 by 34% and 133.2%, respectively (Figure 7-18b).

Blast 4 (70 psi) resulted in the flexural failure of both specimens by means of fiber pullout as displayed in Figure 7-17. The specimen designed with fiber type A experienced reduced maximum and residual displacements by 48% and 101% when compared to the specimen designed with fiber type B (Figure 7-18c). Both specimens experienced displacement and crack opening concentrations in the constant moment region, while all other cracks remained hairline.

In summary, the limited tests conducted in this study indicate that fiber type A, which had a larger aspect ratio of 62 (but reduced tensile strength) was more effective at reducing maximum and residual mid-span displacements in the test beams when compared to fiber type B. Hence, based on these limited tests, it can be concluded that the more critical fiber parameter affecting blast performance is the aspect ratio, whereas the increase in the tensile strength of the fiber itself is not as critical. This can partly be explained by the fact that the weak link in the fiber failure mechanism is the bond between the concrete and fiber, which results in fiber pullout rather than fiber fracture. Further research is recommended.

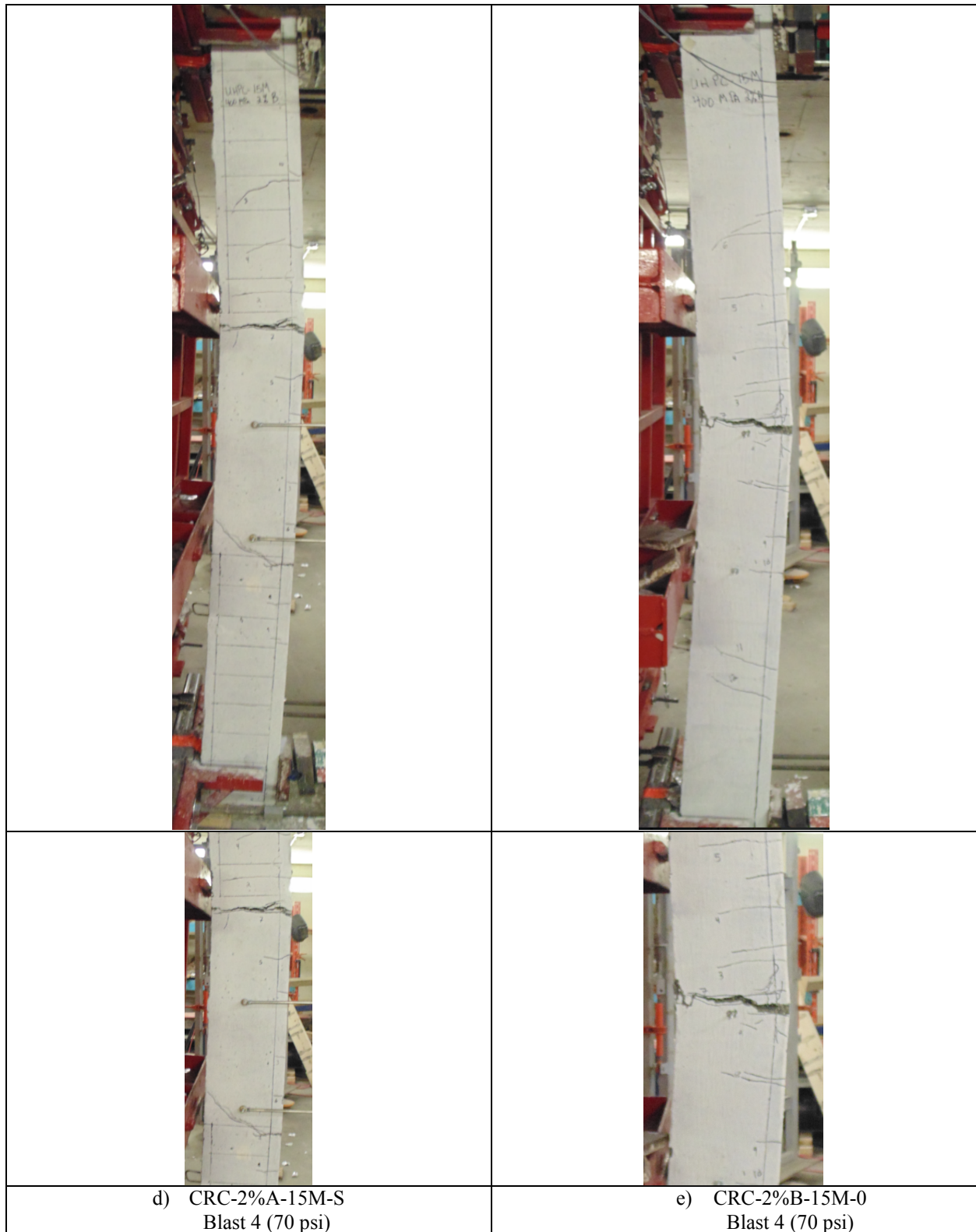


Figure 7-17 Photographs; Effect of Fiber Reinforcement Type

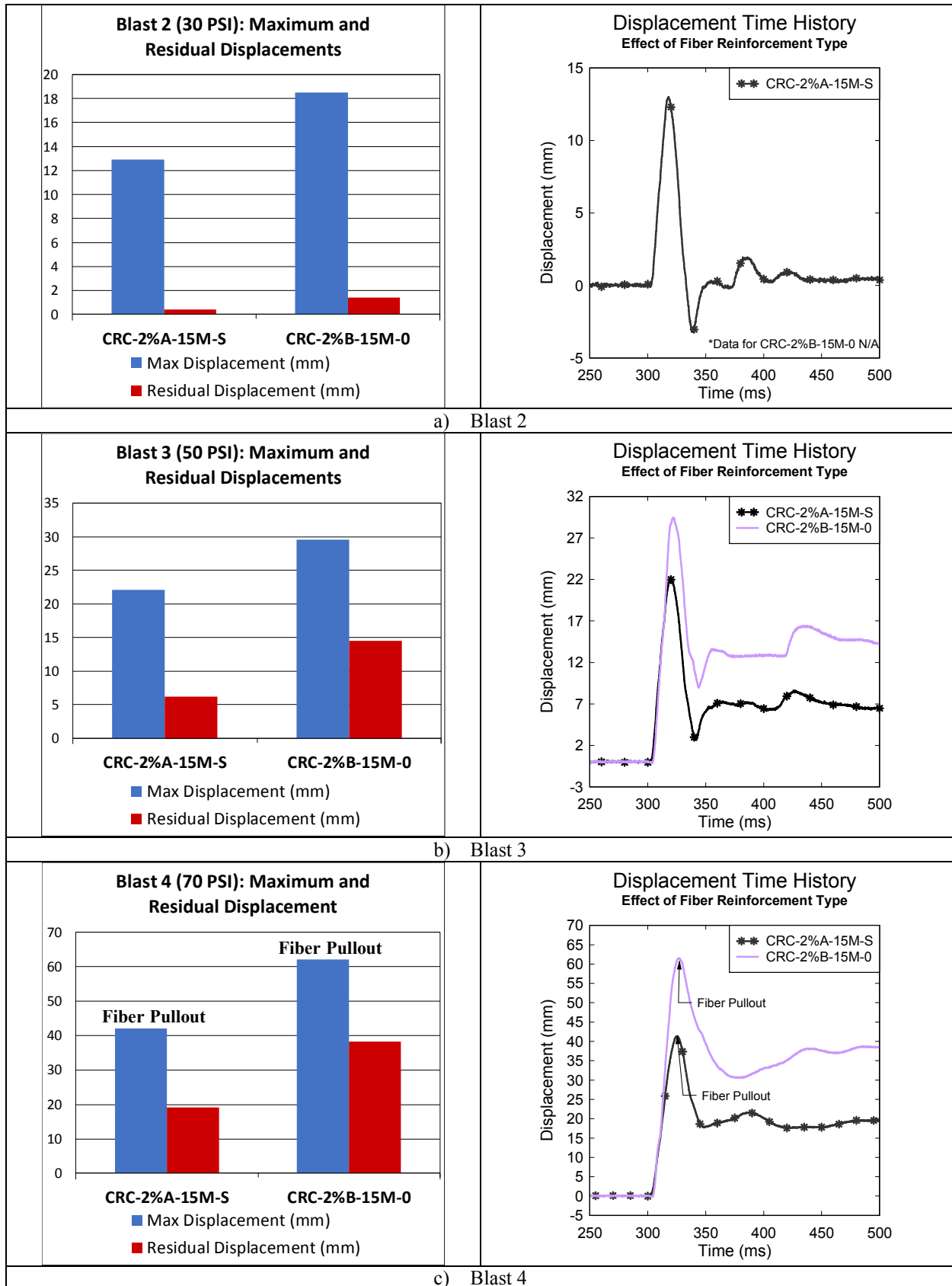


Figure 7-18 Displacement Comparisons; Effect of Fiber Type

7.7. Effect of Fiber Content

Three beam specimens with varying fiber content ranging from 1%-3% were selected to examine the effect of volumetric fiber content on the blast performance of the UHPC beams:

- o CRC-1%B-15M-0 vs. CRC-2%B-15M-0 vs. CRC-3%B-15M-0

All three beams were designed with 15M normal strength longitudinal reinforcement and did not have transverse reinforcement. Specimens CRC-1%B-15M-0 vs. CRC-2%B-15M-0 vs. CRC-3%B-15M-0 were constructed with CRC and B type fibers at volumetric fiber contents of 1% , 2% and to 3%, respectively. Photographs illustrating the failure modes experienced by each specimen are found in Figure 7-19, followed by bar charts and time history comparisons of displacements in Figure 7-21.

Blast 1 (17 psi) does not show any trends with respect to the effect of fiber content.

Blast 2 (30 psi) imposed sufficient force as to cause yielding of the longitudinal reinforcement in all three specimens. However, no clear trend can be established for maximum and residual mid-span displacements as the specimen with 1%, 2% and 3% experienced: ($\delta_{max}=14.25\text{mm}$ & $\delta_{res}=0.4\text{mm}$), ($\delta_{max}=18.52\text{mm}$ & $\delta_{res}=1.42\text{mm}$) and ($\delta_{max}=13.26\text{mm}$ & $\delta_{res}=1.39\text{mm}$), respectively.

Blast 3 (50 psi) caused displacement ranging from 20mm to 30mm, with the volumetric fiber content of 3% experiencing the least maximum and residual mid-span displacements ($\delta_{max}=20.01\text{mm}$ & $\delta_{res}=4.94\text{mm}$), whereas specimen CRC-2%B-15M-0 experienced the greatest displacements recorded at ($\delta_{max}=29.54\text{mm}$ & $\delta_{res}=14.48\text{mm}$). Average crack widths are measured at 0.18mm, 0.53mm and 0.1 mm for corresponding fiber contents of 1%, 2% and 3%.

Blast 4 (70 psi) resulted in the flexural failure of all three specimens by means of fiber pullout as displayed in Figure 7-19. The specimen designed with 1% volumetric fiber content experienced the least maximum and residual displacements of 40.8mm and 18.5mm, while the specimen designed with 2% volumetric fiber content recorded the greatest displacements ($\delta_{max}=62.04\text{mm}$ & $\delta_{res}=38.3\text{mm}$) and lastly the specimen designed with 3% fiber content experienced displacements of ($\delta_{max}=45.65\text{mm}$ & $\delta_{res}=23.88\text{mm}$). All three specimens experienced similar damage with major cracking and fiber pullout concentrated in one localized region within the constant moment region as depicted in Figure 7-19.

The results obtained do not show a clear trend with regards to the effect of fiber content between 1% and 3% fiber ratio. The results obtained are quite similar with all three specimens failing at blast 4 by means of fiber pullout. More research should be conducted to draw more definitive conclusions.

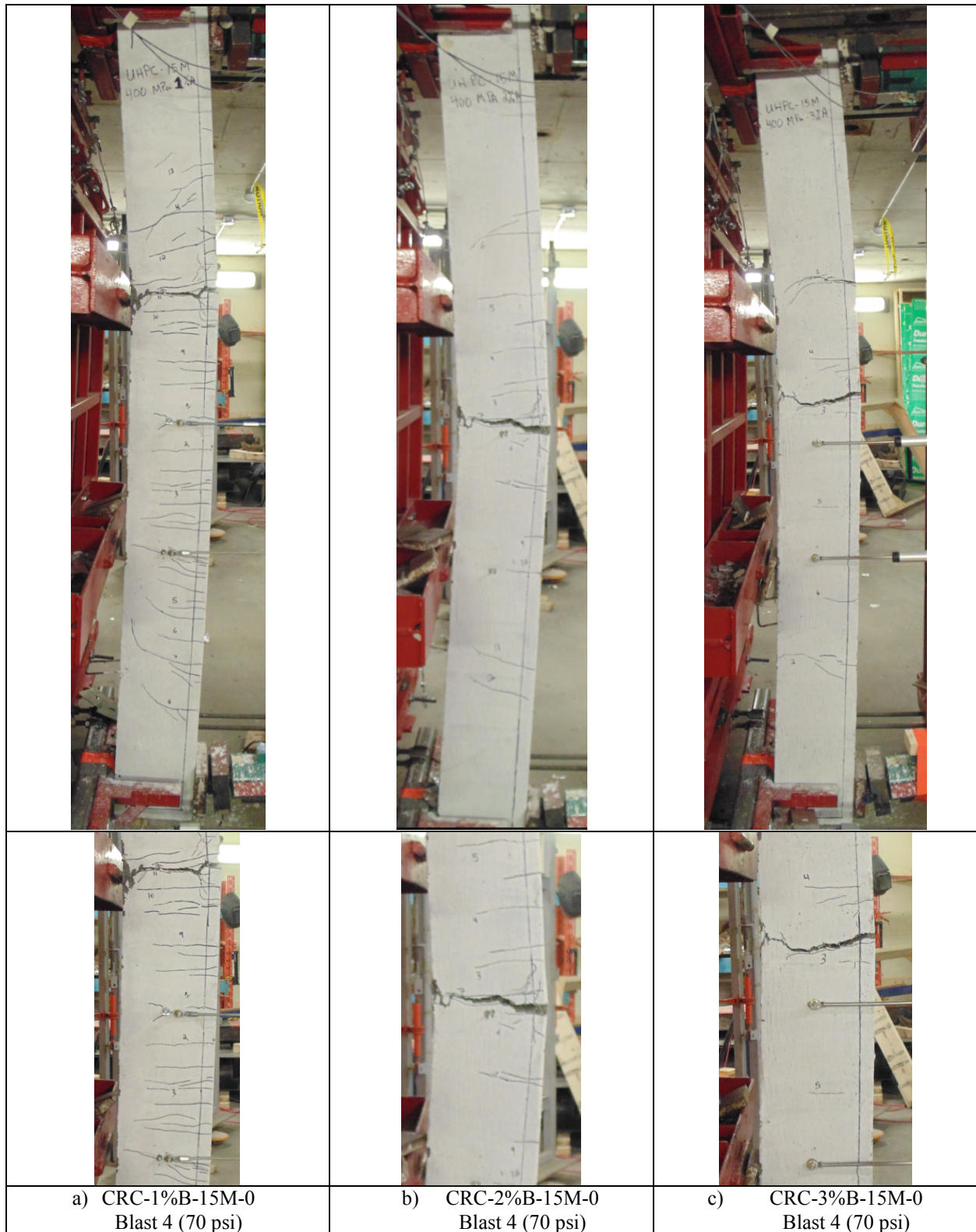


Figure 7-19 Photographs; Effect of Volumetric Fiber Content

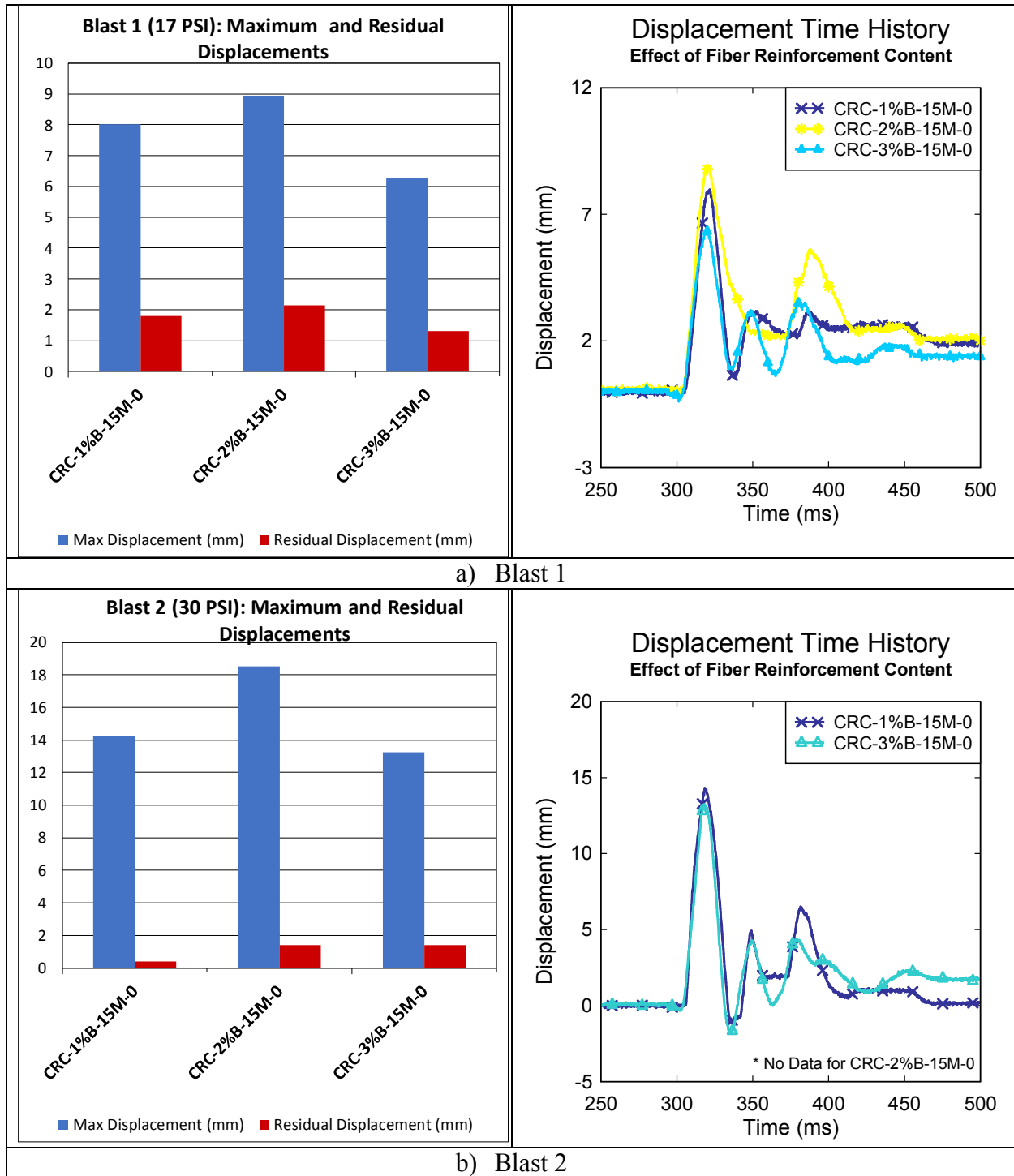


Figure 7-20 Displacement Comparisons; Effect of Fiber Content (1/2)

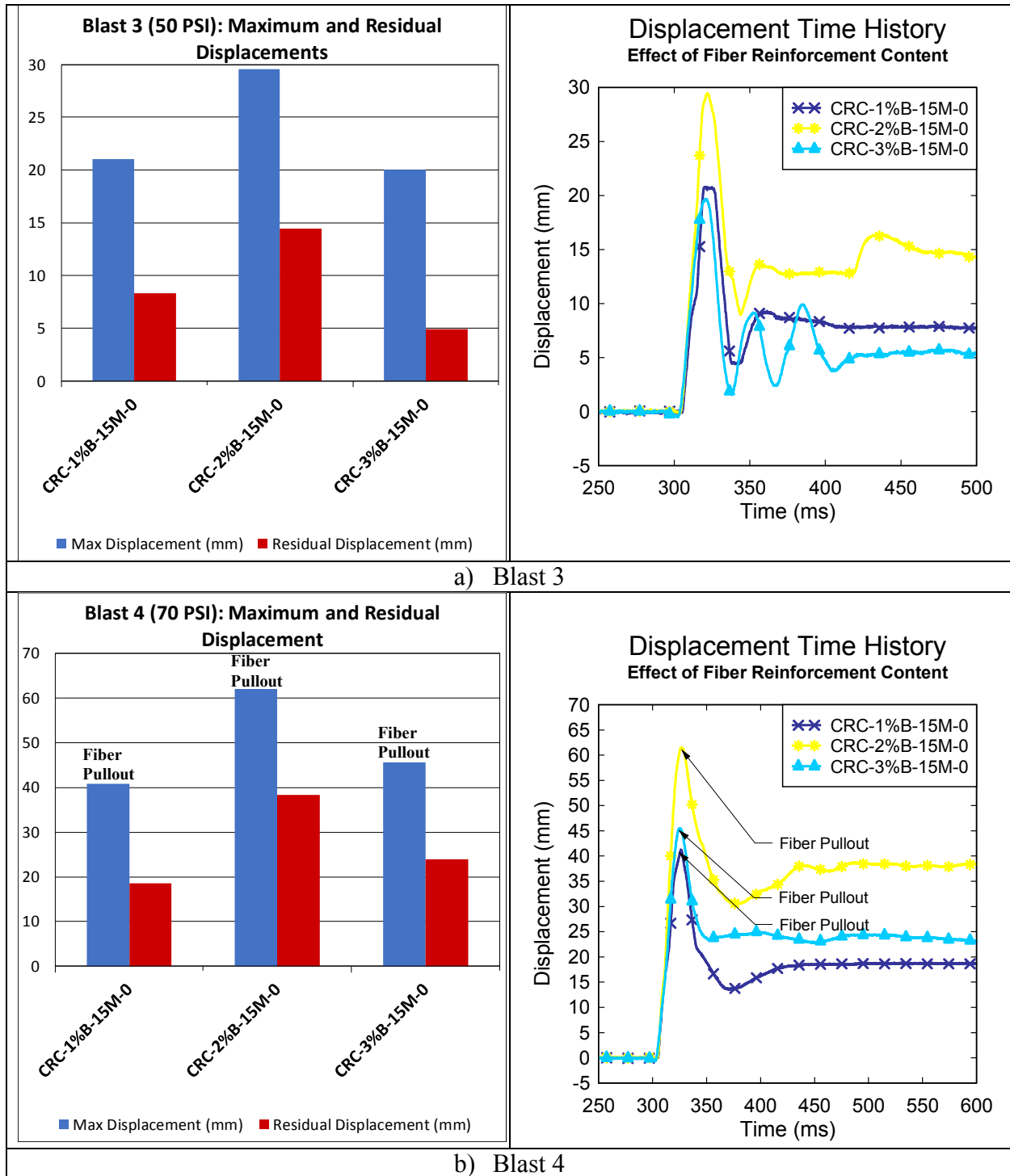


Figure 7-21 Displacement Comparisons; Effect of Fiber Content (2/2)

7.8. Effect of Transverse Reinforcement

Two companion sets comprised of two specimens from the control series and two specimens from the UHPC & NSR series are used to investigate the effect of transverse reinforcement on the blast performance of the beams.

- Control Series with 15M bars: NSC-0%-15M-0 vs. NSC-0%-15M-S
- UHPC and NSR Series with 20M bars: CRC-2%A-20M-0 vs. CRC-2%A-20M-S

The control series set consists of two normal strength concrete beams, having 15M longitudinal reinforcement, where one specimen has no transverse reinforcement (NSC-0%-15M-0), whereas the second has stirrups spaced at 100mm in the shear spans. Similarly, the UHPC and NSR specimens are designed with CRC having 2%A fibers and normal strength 20M longitudinal reinforcement, with the first specimen (CRC-2%A-20M-0) built without stirrups and the second having stirrups spaced at 100mm in the shear spans. Photographs illustrating failure modes experienced by each specimen is found in Figure 7-22, followed by bar chart comparisons of displacements in Figure 7-23 and Figure 7-24.

Due to technical problems, no data was collected for specimen NSC-0%-15M-0 for blasts 1-2. Blast 3 (50 psi) resulted in the brittle shear failure of specimen NSC-0%-15M-0. As illustrated in Figure 7-22, a major diagonal shear crack formed in the top shear span near the support. The maximum and residual displacements were 67.72mm and 26.22mm. In contrast, specimen NSC-0%-15M-S, which was detailed stirrups, prevented shear failure, with failure caused by means of concrete crushing in the constant moment region, with lower maximum and residual mid-span displacements of 42.05mm and 30.6mm.

While both UHPC specimens failed in flexure due to the provision of fibers, better displacement control was observed at the various loading stages for the specimen with stirrups. As early as Blast 1 (17 psi), specimen CRC-2%A-20M-0 experienced displacements of $\delta_{max}=7.73\text{mm}$ & $\delta_{res}=2.36\text{mm}$, which are greater by factors of 141% and 144% when compared to those of CRC-2%A-20M-S ($\delta_{max}=4.5\text{mm}$ & $\delta_{res}=1.3\text{mm}$). Blast 2 (30 psi) imposed sufficient force as to cause yielding of the longitudinal reinforcement in all specimens. Specimen CRC-2%A-20M-S presented a reduction of 5.7% in maximum mid-span displacement when compared to CRC-2%A-20M-0. Comparing the UHPC specimens at blast 3 (50 psi), it is found that the specimen detailed with transverse reinforcement reduced the maximum mid-span displacement by 31%.

Blast 4 (70 psi) imposed sufficient force as to cause specimen CRC-2%A-20M-0 to experience maximum and residual displacement of 34.06mm and 14.55mm with a corresponding support rotation of $\theta=1.75^\circ$. Specimen CRC-2%A-20M-S records $\delta_{max}=25.86\text{mm}$ & $\delta_{res}=7.77\text{mm}$, with a corresponding support rotation of $\theta=1.33^\circ$, representing better deformation control for the specimen with stirrups. Blast 5 (90 psi) ultimately caused the flexural failure of both UHPC specimens as a result of fiber pullout in the constant moment region (Figure 7-24). The two specimens experienced a major crack opening at the point load application points. Although the addition of transverse reinforcement in specimen CRC-2%A-20M-S did not delay the failure to a

greater blast shockwave, nor did it alter the failure mechanism, the addition of stirrups did reduce the maximum and residual mid-span displacements by 17% and 15%, respectively.

In summary, the results confirm the importance of providing shear reinforcement in conventional reinforced concrete beams to prevent brittle shear failure. The addition of transverse reinforcement demonstrated the ability to alter the failure mechanism in the NSC beams with 15M bars from brittle shear to a more ductile flexural failure mode. The provision of fibers in UHPC increases shear capacity and can substitute for transverse reinforcement, however the addition of stirrups improves performance in terms of controlling maximum and residual displacements.

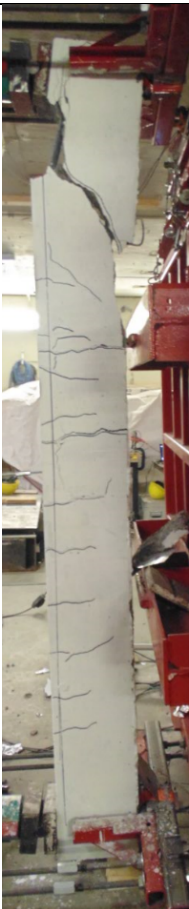
Control NSC set with 15M bars		UHPC set with 20M bars	
			
a) NSC-0%-15M-0 Blast 3 (50 psi)	b) NSC-0%-15M-S Blast 3 (50 psi)	c) CRC-2%A-20M-S Blast 5 (90 psi)	d) CRC-2%A-20M-S Blast 5 (90 psi)

Figure 7-22 Photographs; Effect of Transverse Reinforcement

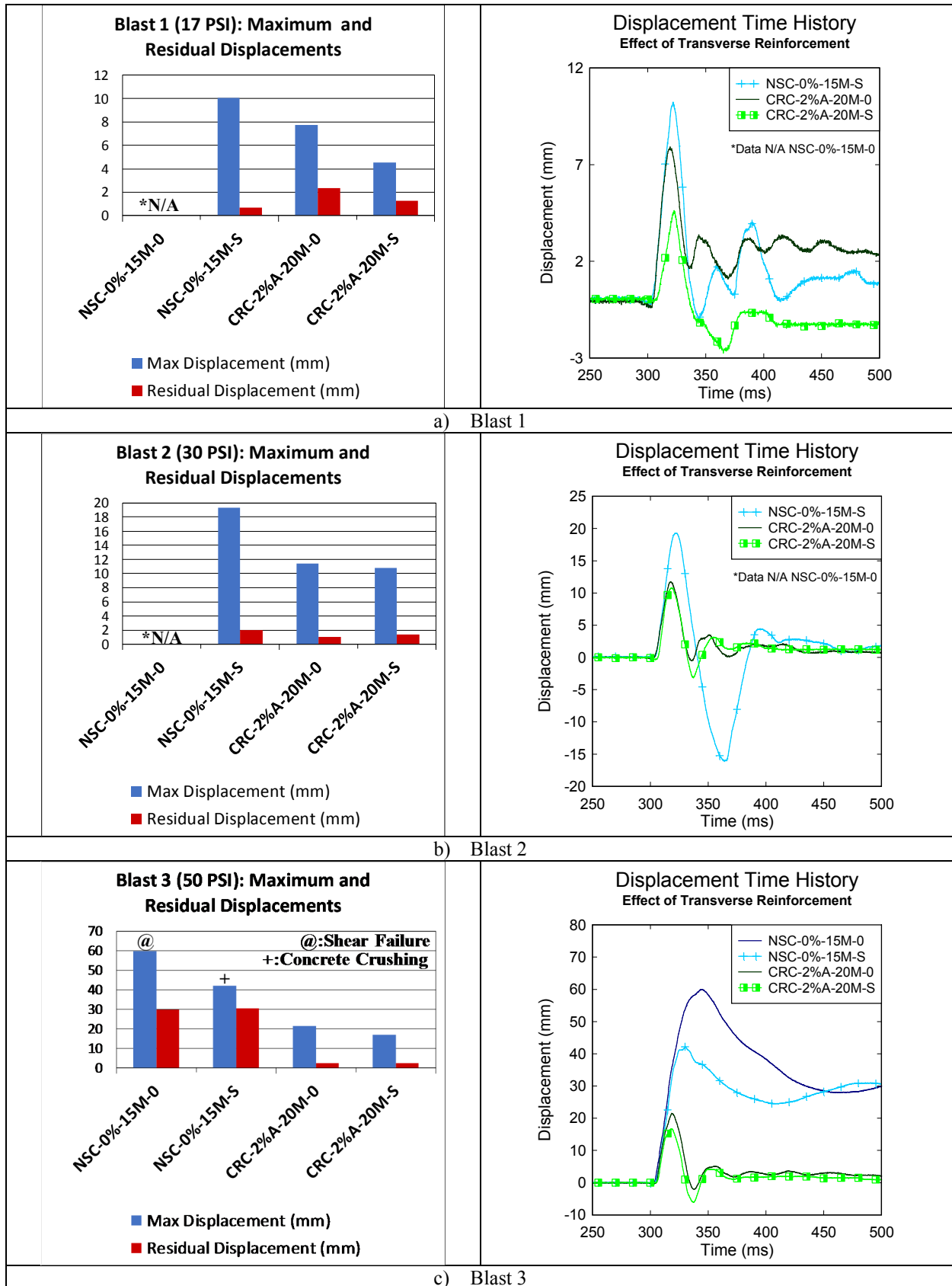


Figure 7-23 Displacement Comparisons; Effect of Transverse Reinforcement (1/2)

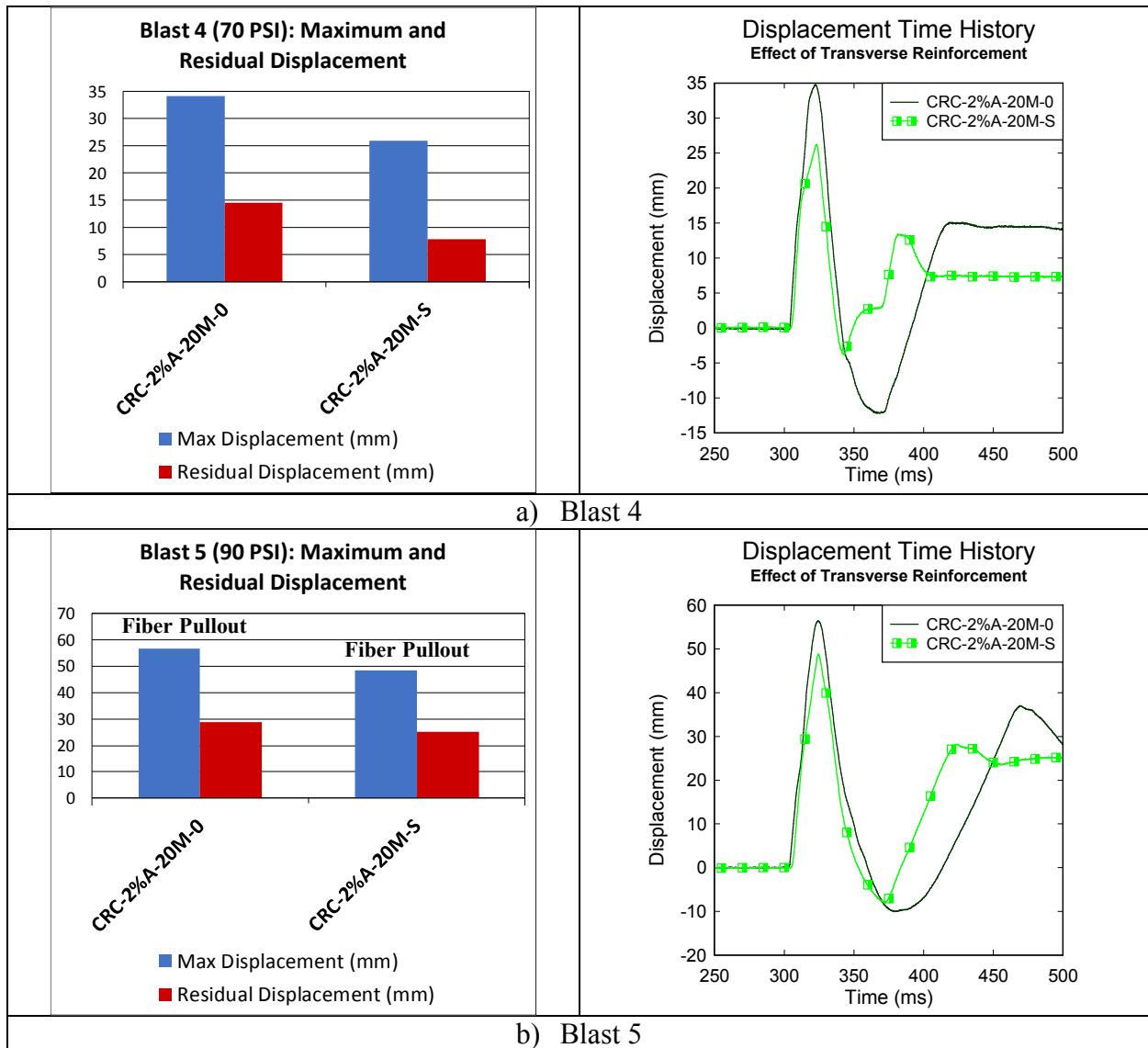
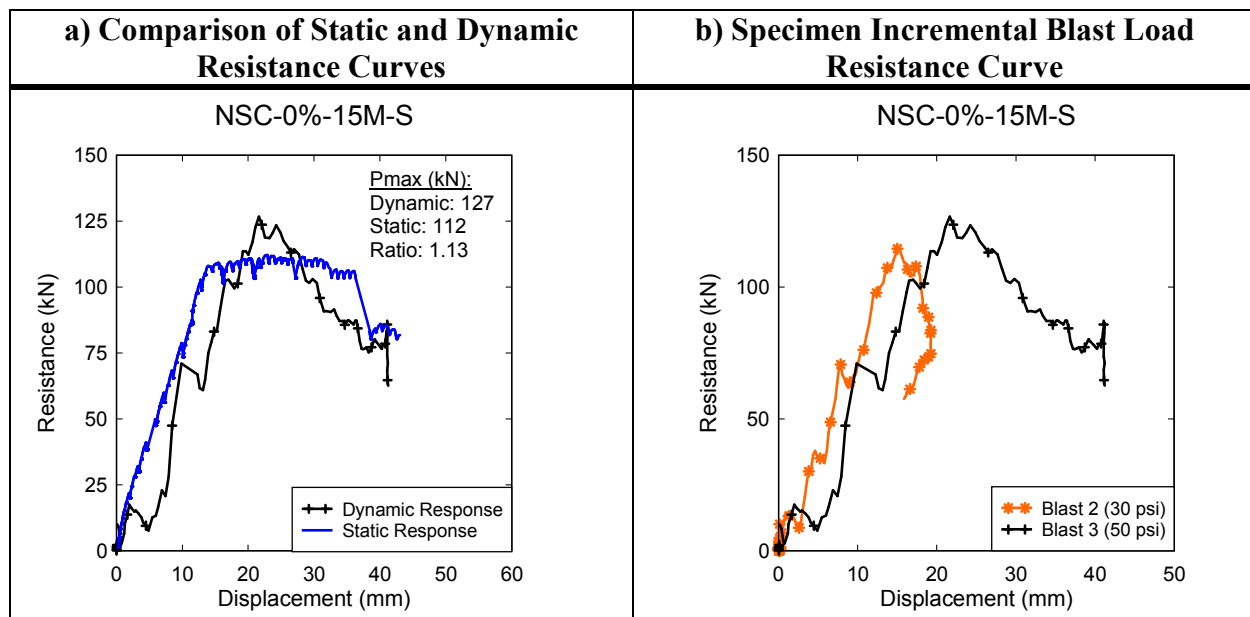


Figure 7-24 Displacement Comparisons; Effect of Transverse Reinforcement (2/2)

7.9. Comparison of Dynamic Reactions and Static Resistance

In this section the resistance of companion beams tested under static and dynamic loading are compared. For the dynamic tests, the resistance is obtained by summing the dynamic reactions recorded by the two load-cells in the shock-tube setup. The results are presented in Figure 7-25 in terms of maximum dynamic load and maximum static load as well as their ratio ($\frac{p_{max}^d}{p_{max}^s}$). Notably, due to the dynamic effects on material properties caused by the high strain rate effects, the specimens subjected to dynamic blast loading show greater ultimate loads than their quasi-statically loaded counterparts. The average dynamic increase ratio observed for the sample specimens in the three series presented in Figure 7-25 is 1.20. Furthermore, examining the incremental blast loads presented in column b) of Figure 7-25, it becomes apparent that the dynamic to static maximum load ratio increases with increase in blast energy with the largest dynamic resistance curve observed as the beams approach their maximum blast capacity. An interesting observation in all series is the degradation of the stiffness at later blasts. The specimens in this study were subjected to multiple blasts, and the accumulated damage from earlier blasts may have caused this degradation in stiffness. The result indicates the need for further research on specimen tested under single versus repeated blast loading.



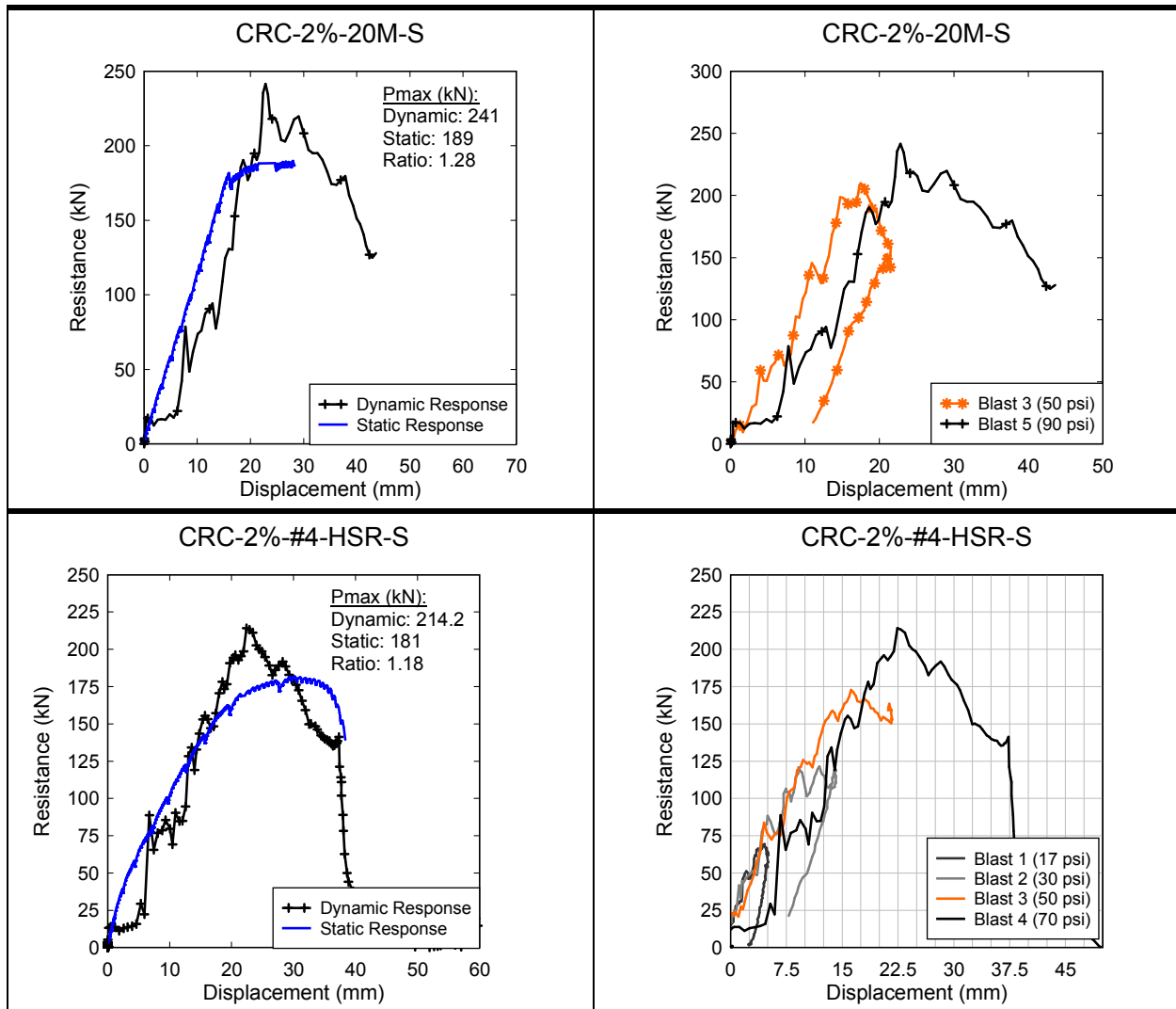


Figure 7-25 a) Comparison of Resistance Curves Under Static and Dynamic Loading and b) Incremental Blast Load Resistance Curves

Chapter 8. Static and Dynamic Analytical Predictions

8.1. Chapter Overview

This chapter presents a detailed overview of the static and dynamic analysis, with a comparison of the analytically computed and experimental beam specimen responses. Sectional and member analyses are conducted by defining constitutive material models for concrete and steel reinforcement, incorporating dynamic effects. The analytical procedure was automated in a VBA computer program developed as part of this master's program called *UO Resistance*. Section 8.2 defines the various material stress-strain models utilized in the analysis. Section 8.3 introduces the dynamic increase factor models for concrete and steel reinforcement which were implemented in the dynamic analysis of specimens tested under blast loading. The procedure employed to obtain the inelastic moment-curvature and load-displacement relationships is described in Section 8.4. Details on the Single Degree of Freedom (SDOF) solution are provided in section 8.5. Sections 8.6 and 8.7 present a comparison of the analytical and experimental results for the beams tested under static and blast loading, respectively. To examine the effect of various modelling parameters on the static and dynamic predictions, sensitivity analyses are presented in Sections 8.6 and 8.8.

8.2. Constitutive Material Models

This section presents the constitutive stress-strain models for plain concrete, ultra-high performance concrete (UHPC), normal strength steel and high strength steel.

8.2.1. Plain Concrete Model

Hognestad (1951) and Linear Tension Model

The unconfined concrete stress-strain relationship in the conventional concrete beams is modeled using the Hognestad (1951) model, which is defined using an ascending second degree parabola as shown in equation (8-1). This relationship reaches it peak of f'_c at a peak strain of $\epsilon_0 = 0.002$.

$$f_c = f'_c \left[\frac{2\epsilon_c}{\epsilon_0} - \left(\frac{\epsilon_c}{\epsilon_0} \right)^2 \right] \quad (8-1)$$

The post-peak response is defined using a linearly descending branch from peak compressive stress to $0.85f'_c$ at a corresponding strain of 0.0038. The descending branch is extended to an ultimate strain (ϵ_{cu}), which is taken as 0.005. The tensile contribution of concrete was assumed to be linear from 0 stress up to a stress of $0.33\sqrt{f'_c}$ having a prescribed tensile modulus of elasticity of 30 GPa. A sample stress-strain curve for plain concrete is shown in Figure 8-1.

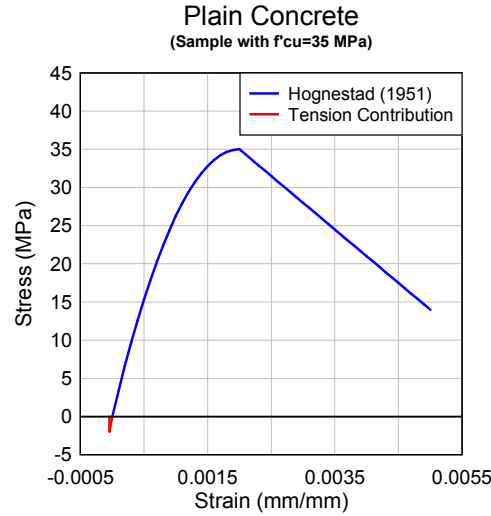


Figure 8-1 Unconfined plain concrete stress-strain model sample.

8.2.2. Ultra-High Performance Concrete Models

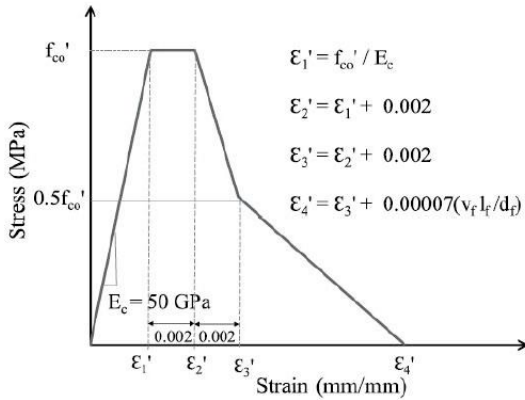
Two models were selected to represent the behaviour of Ultra-High Performance Concrete (UHPC) based on research conducted by Melançon (2015). The compression stress-strain relationship implemented was that proposed by Hosinieh et al. (2015) and the tensile contribution was modeled using the relationships proposed by Lok & Pei (1998).

Hosinieh et al. (2015)

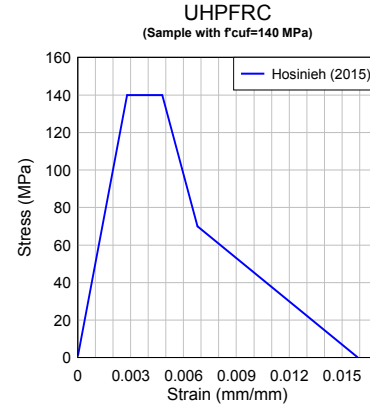
The simplified stress-strain relationship proposed by Hosinieh et al. (2015) is a quadrilinear model governed by four principle stress and strain values presented in Table 8-1. The model prescribes a modulus of elasticity $E_c = 50$ GPa, up until its first strain point ϵ'_1 of 0.002 mm/mm and a peak stress value of f'_{cuf} . The stress remains constant from ϵ'_1 to the second strain point ϵ'_2 , then follows a linearly descending branch to the third strain point ϵ'_3 corresponding to a stress equivalent to 50% of f'_{cuf} . The final descending branch incorporates steel fiber contribution in terms of volumetric fiber quantity and fiber aspect ratio presented in the ϵ'_4 equation, with a corresponding stress of 0MPa. A sample stress-strain curve is shown in Figure 8-2.

Table 8-1 Hosinieh Principle Stress-Strain Values (Hosinieh, Aoude, Cook, & Mitchell, 2015)

<u>Strain</u>	<u>Stress</u>
$\epsilon'_1 = \frac{f'_{cuf}}{E_c}$ (8-2)	$f_{cuf1} = f'_{cuf}$ (8-3)
$\epsilon'_2 = \epsilon'_1 + 0.002$ (8-4)	$f_{cuf2} = f'_{cuf}$ (8-5)
$\epsilon'_3 = \epsilon'_2 + 0.002$ (8-6)	$f_{cuf3} = 0.5(f'_{cuf})$ (8-7)
$\epsilon'_4 = \epsilon'_3 + 0.00007 \left(\frac{v_f l_f}{d_f} \right)$ (8-8)	$f_{cuf4} = 0$ (8-9)



a) UHPC unconfined compression model



b) Sample UHPC stress-strain curve

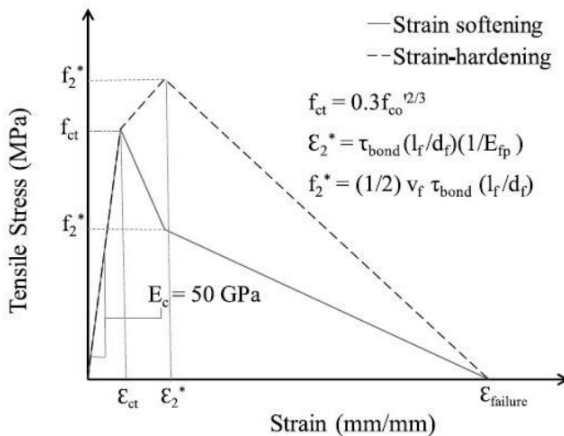
Figure 8-2 Unconfined UHPC stress-strain models (Hosinieh, Aoude, Cook, & Mitchell, 2015).

Lok & Pei (1998)

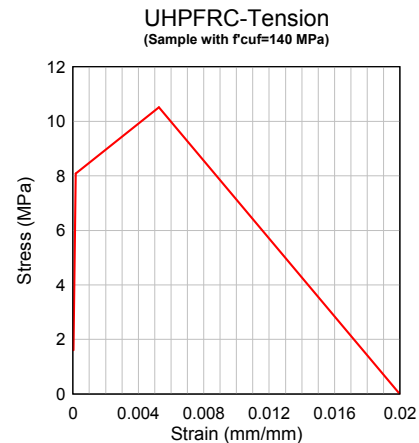
The generic fiber-reinforced concrete tension stress-strain model proposed by Lok & Pei (1998) begins by following the modulus of elasticity of the concrete up to the cracking stress of concrete, f_{ctf1} as defined in Table 8-1 and Figure 8-3. Beyond the cracking strain denoted (ϵ'_1), the model is governed by a linear segment connecting the cracking stress to f_{ctf2} , which is a function of the volumetric fiber content and fiber aspect ratio, where either strain-softening or hardening is possible based on the fiber parameters. The final descending segment neutralizes the concrete tensile contribution at a final strain of 0.02 mm/mm as suggested by Lok & Pei (1998).

Table 8-2 Principle Stress-Strain Values (Lok & Pei, 1998)

Strain	Stress
$\epsilon'_1 = \frac{f'_{ctf}}{E_c}$ (8-10)	$f_{ctf1} = 0.3(f'_{cu})^{2/3}$ (8-11)
$\epsilon'_2 = \tau_{bond} \frac{l_f}{d_f} \frac{1}{E_{fp}}$ (8-12)	$f_{ctf2} = 0.5v_f \tau_{bond} \frac{l_f}{d_f}$ (8-13)
$\epsilon'_3 = 0.02$ (8-14)	0 (8-15)
$\tau_{bond} = 0.6(f'_{cu})^{2/3}$ (8-16)	



c) UHPC tension model



d) Sample UHPC tension stress-strain curve

Figure 8-3 Tension fiber-reinforced concrete model (Lok & Pei, 1998)

8.2.3. Steel Reinforcement Models

Normal-strength Steel Reinforcement

The stress-strain relationship proposed by Park & Paulay (1975) is utilized to model the behaviour of the normal strength (400MPa) longitudinal reinforcement bars, as summarized in equations (8-17) through (8-19); where ε_{sf} is the steel strain, f_s is the steel stress, ε_y and f_y correspond to the yield strain and stress, ε_{sh} refers to strain hardening, ε_{su} and f_{su} are the ultimate strain and stress, respectively. Lastly, variables r and m are calculated as shown in equations (8-20) and (8-21). A sample stress-strain curve is presented in Figure 8-4.

$$f_s = \begin{cases} E_s \varepsilon_{sf} & \text{for } 0 \leq \varepsilon_{sf} \leq \varepsilon_y & (8-17) \\ f_y & \text{for } \varepsilon_y \leq \varepsilon_{sf} \leq \varepsilon_{sh} & (8-18) \\ f_y \left[\frac{m(\varepsilon_{sf} - \varepsilon_{sh}) + 2}{60(\varepsilon_{sf} - \varepsilon_{sh}) + 2} + \frac{(\varepsilon_{sf} - \varepsilon_{sh})(60 - m)}{2(30r + 1)^2} \right] & \text{for } \varepsilon_{sh} \leq \varepsilon_{sf} \leq \varepsilon_{su} & (8-19) \end{cases}$$

$$r = (\varepsilon_{su} - \varepsilon_{sh}) \quad (8-20)$$

$$m = \frac{\left(\frac{f_{su}}{f_y}\right) (30r + 1)^2 - 60r - 1}{15r^2} \quad (8-21)$$

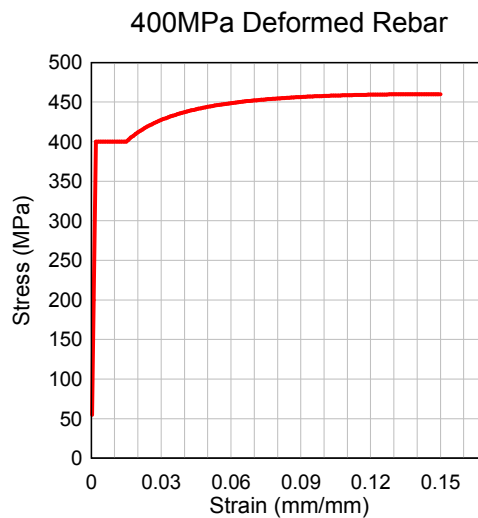


Figure 8-4 Regular Grade Deformed Steel Reinforcement Stress-Strain Model

High Strength Steel (MMFX) Model

The behaviour of the high strength ASTM A1035 (MMFX) longitudinal steel reinforcement was modeled, using two approaches. The first stress-strain relationship implemented in the analysis was taken directly from the coupon test data obtained through material testing. The second model consist of a set of equations (8-22 through 8-24) prescribed by the ACI Task Group ITG-6 (2010) for ASTM A1035 steel. Figure 8-5 presents sample experimental and ITG-6 stress-strain curves.

$$f_s = \begin{cases} 200,000 * \varepsilon_{sf} \text{ (MPa)} & \text{for } \varepsilon_{sf} \leq 0.0024 & (8-22) \\ 1170 - \frac{2.96}{\varepsilon_{sf} + 0.0019} \text{ (MPa)} & \text{for } 0.0024 \leq \varepsilon_{sf} \leq 0.02 & (8-23) \\ 1040 \text{ (MPa)} & \text{for } \varepsilon_{sf} \leq 0.06 & (8-24) \end{cases}$$

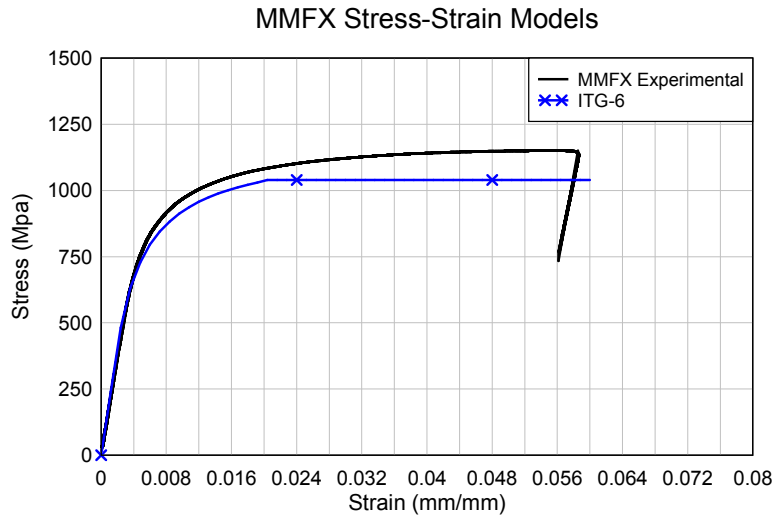


Figure 8-5 MMFX₂ Stress-Strain Models

8.3. Dynamic Increase Factors

Structural elements subjected to high strain rate loading are subject to enhanced capacity; this augmentation in capacity is considered by multiplying the constitutive material stress-strain models by specific dynamic increase factors (DIF) proposed by researchers and blast design guidelines. This sub-section presents the dynamic increase factors for the various materials utilized in the research program. For the strain-rate dependant models, the dynamic strain-rate imposed by the University of Ottawa's shock-tube is assumed to be $\dot{\varepsilon} = 1s^{-1}$ (Saatcioglu, Lloyd, Jaques, Braimah, & Doudak, 2011).

8.3.1. Dynamic Increase Factors for Concrete

CEB (1990)

The model proposed by the Euro-International Concrete Committee presented in equations (8-25) through (8-27) was the default model used to account for dynamic effects on the response of plain concrete in compression (DIF_c). When plotted on a log-scale the two-part model is bi-linear with a transition strain-rate of $30s^{-1}$. The equation set also takes in account the unconfined compressive strength of the concrete through parameter α , with the model predicting reduced strain-rate sensitivity as the concrete strength is increased. Using the assumed strain-rate of $1s^{-1}$ the DIF_c for the unconfined concrete in the plain concrete specimens is predicted to be 1.34.

$$DIF_c = \begin{cases} \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^{1.026\alpha} & \text{for } \dot{\epsilon} \leq 30s^{-1} \quad (8-25) \\ \gamma \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^{1.026\alpha} & \text{for } \dot{\epsilon} > 30s^{-1} \quad (8-26) \end{cases}$$

$$\log \gamma = 6.156\alpha - 2; \quad \alpha = \frac{1}{\left(5 + \frac{9f'_{cu}}{10}\right)} \quad ; \quad \dot{\epsilon}_s = 30 \cdot 10^{-6}s^{-1} \quad (8-27)$$

Zhang and Mindness (2011)

The Zhang and Mindness (2011) model was used to account for dynamic effects in UHPC in compression (DIF_c). The model was developed based on dynamic impact testing of high strength steel fiber reinforced concrete in compression. This bi-linear model considers the unconfined strength of fiber-reinforced concrete using parameter β . The transition point in the model is defined using strain-rate $\dot{\epsilon}_{BLT}$. For the assumed strain rate of $\dot{\epsilon} = 1s^{-1}$ and the typical compressive strength of UHPC implemented in the current study, this model yields a DIF_c value of 1.20.

$$DIF_c = \begin{cases} (\varphi \dot{\epsilon})^\alpha & \text{for } \dot{\epsilon} < \dot{\epsilon}_{BLT} \quad (8-28) \\ \beta (\dot{\epsilon})^{1/3} & \text{for } \dot{\epsilon} \geq \dot{\epsilon}_{BLT} \quad (8-29) \end{cases}$$

$$\alpha = \frac{\ln(\beta \cdot \dot{\epsilon}_{BLT}^{\frac{1}{3}})}{\ln(\varphi \cdot \dot{\epsilon}_{BLT})} \quad ; \quad \beta = \frac{5}{9} \cdot e^{\left(\frac{-f'_{cuf}}{230}\right)} \quad ; \quad \varphi = 1 \cdot 10^5 \quad ; \quad \dot{\epsilon}_{BLT} = 25 \cdot e^{\left(\frac{f'_{cuf}}{130}\right)} \quad (8-30)$$

Malvar & Ross (1998)

Dynamic effects in tension concrete were considered using the model proposed by Malvar and Ross (1998). This bi-linear model adapts the previously discussed CEB model, with differences in the transition strain-rate and factors used to obtain DIF_t . Due to limited research on dynamic increase factors of UHPC in tension (DIF_t), the Malvar and Ross (1998) model was also selected for the analysis of the ultra-high performance concrete beams. At $\dot{\epsilon} = 1s^{-1}$, the model yields a DIF_t values of 1.52 and 1.13 for the NSC and UHPC used in the current study.

$$DIF_c = \begin{cases} \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^{\delta} & \text{for } \dot{\epsilon} \leq 1s^{-1} \\ \beta^* \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^{1/3} & \text{for } \dot{\epsilon} > 1s^{-1} \end{cases} \quad (8-31)$$

$$\delta = \frac{1}{\left(5 + \frac{8f'_{cuf}}{f'_{cr}}\right)} ; \log \beta^* = 6\delta - 2 ; f'_{cr} = 10MPa ; \dot{\epsilon}_s = 30 \cdot 10^{-6}s^{-1} \quad (8-32)$$

8.3.2. Dynamic Increase Factors for Steel Reinforcement

Two types of steel reinforcement were included in the research program. The dynamic increase model utilized for the normal strength steel was that proposed by Saatcioglu et al. (2011).

$$\text{at yield: } DIF_y = 0.034 \ln(\dot{\epsilon}) + 1.30 \geq 1.0 \quad (8-34)$$

$$\text{at ultimate: } DIF_u = 0.0101 \ln(\dot{\epsilon}) + 1.10 \geq 1.0 \quad (8-35)$$

As defined in the equations above, the model defines two dynamic increase factor values; one at yield (DIF_y) and one at ultimate (DIF_u), based on research which indicates reduced strength increase for steel reinforcement at ultimate. At $\dot{\epsilon} = 1s^{-1}$ the model predicts factors of $DIF_y=1.3$ to $DIF_u=1.1$. In between yield and ultimate a linear transition was applied from $DIF_y=1.3$ to $DIF_u=1.1$. Moreover, the strain up to yield was also multiplied by the yield dynamic increase factor such that the modulus of elasticity remains constant (Hong & Kang, 2016). Due to limited research on the effects of high strain rates on high strength steel, a constant factor of 1.1, corresponding to the DIF at ultimate predicted by the Saatcioglu model, was selected for the MMFX bars.

8.3.3. Design Dynamic Increase factors (UFC-3-340-02, 2008)

The UFC-3-340-02 (2008) document produced by the Department of Defense of the United States of America, proposes dynamic increase factors for concrete and steel reinforcement which can be used in design. As presented in Figure 8-6 the factors are prescribed for far and close range blast

loads, and are function of the state of stress which these materials are subjected to; bending, diagonal tension, direct shear, bond and compression (DoD, 2008). Using the UFC criteria, the dynamic increase factors for concrete in compression and steel reinforcing bars at yield/ultimate were selected to be 1.19 and 1.17 /1.05, respectively, such as depicted in Figure 8-7.

Figure 8-6 Design Dynamic Increase Factors (UFC-3-340-02, 2008)

TYPE OF STRESS	FAR DESIGN RANGE			CLOSE-IN DESIGN RANGE		
	Reinforcing Bars		Concrete	Reinforcing Bars		Concrete
	f_{dy}/f_y	f_{du}/f_u	f'_{dc}/f'_c	f_{dy}/f_y	f_{du}/f_u	f'_{dc}/f'_c
Bending	1.17	1.05	1.19	1.23	1.05	1.25
Diagonal Tension	1.00	-----	1.00	1.10	1.00	1.00
Direct Shear	1.10	1.00	1.10	1.10	1.00	1.10
Bond	1.17	1.05	1.00	1.23	1.05	1.00
Compression	1.10	-----	1.12	1.13	-----	1.16

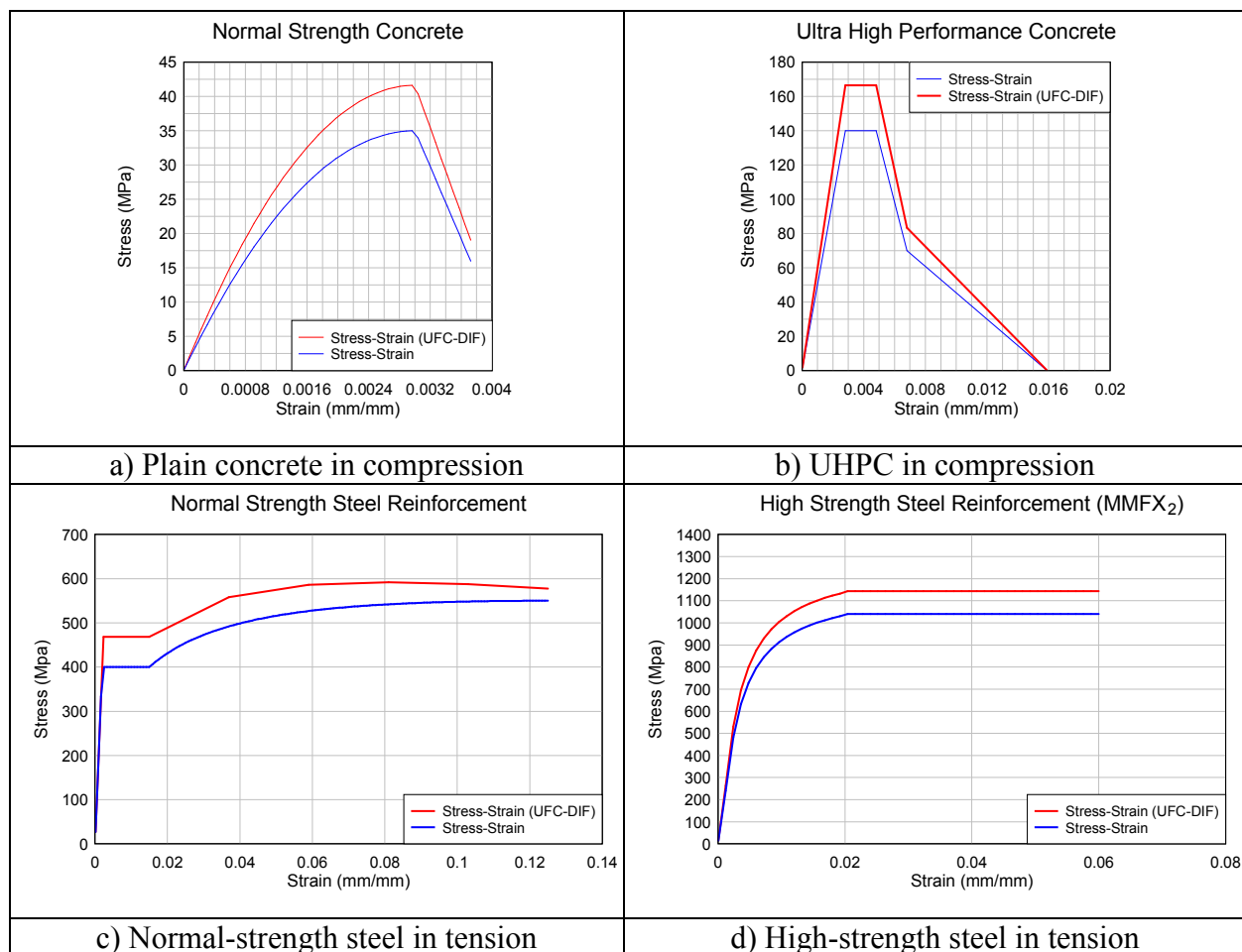


Figure 8-7 Sample Stress-strain Curves with and without Dynamic Effects

8.4. Member and Dynamic Analysis

The analysis conducted in this research program was implemented in a computer program named *UO Resistance*, developed during this master's research. The VBA code enables the user to input a beam's geometric properties and select pre-defined constitutive material stress-strain relationships and DIF models to perform sectional analysis which are then used to obtain the resistance curve of the beam. *UO Resistance* is also capable of performing single degree of freedom dynamic analysis as further explained in the sections that follow.

8.4.1. Sectional Analysis & Resistance Curve

Based on cross-section geometry, reinforcement properties and material model selection, the sectional analysis in *UO Resistance* constructs the moment-curvature relationship. The analysis is conducted in 50 strain increments; the increments are chosen by gradually increasing the strain at the peak compressive fiber until the ultimate compression strain of concrete is reached. Starting with the smallest compressive strain, the sectional analysis algorithm iterates for the neutral axis depth using the numerical secant method. The neutral axis depth (NA) is determined by plotting the stress and strain distributions along the section and ensuring that the sum of forces on the cross-section considering all material contributions and imposed axial force is zero. The resultant forces are obtained by numerical integration (trapezium method) of the stresses distributed across the member's cross-section (Figure 8-8). It is noted that *UO Resistance* discretizes the element's cross-sectional height into 100 equal strips, for which the stress experienced at a specific location is assigned based on the given linear strain profile. Once equilibrium between compressive and tensile force components is obtained, the internal moment resistance and corresponding curvature is computed and stored as a data point within the moment-curvature relationship. This process is repeated until the ultimate compression strain is reached or until tensile rupture occurs in the longitudinal steel reinforcement in which case it becomes impossible to balance the compressive force component and the moment resistance rapidly descends to zero (Figure 8-8). Figure 8-10 shows a sample moment-curvature relationship as obtained using the program.

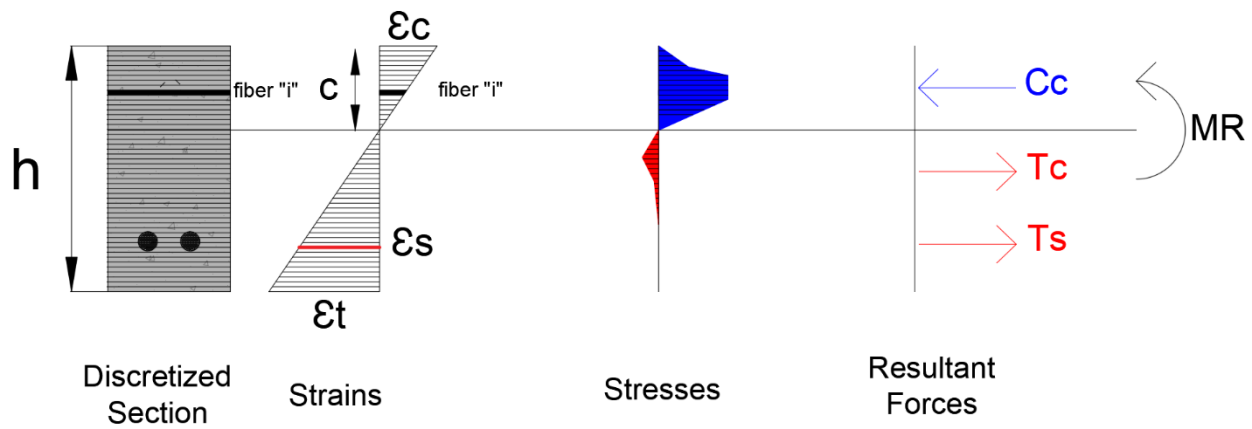


Figure 8-8 Sectional Analysis

With the sectional response defined, *UO Resistance* proceeds to compute the element member resistance curve. For the pre-yield range when the curvature (ϕ) is equal or inferior to the yield curvature ($\phi \leq \phi_y$) the displacements are obtained by means of integration of curvature along the beam length using the moment-area method as depicted in Equation 8.36 and Figure 8-9(a). The plastic post-yield ($\phi > \phi_y$) displacements are calculated using plastic analysis. The plastic analysis assumes that post-yield displacements are concentrated at the location of the plastic hinge formation over a length referred to as a plastic hinge length (l_p), which the user can define by selecting pre-defined models or specifying a custom value; more details on the plastic hinge length models are presented in section 8.4.2. The post-yield displacement is then obtained by summing the plastic displacement at the plastic hinge and yield displacement as defined in Equation 8.37 and Figure 8-9(b). Sample moment-curvature and load-deflection curves obtained using this procedure are shown in Figure 8-10.

$$(\phi \leq \phi_y) \quad \Delta = \int_0^{\frac{L}{2}} \phi * x \, dx \quad (8-36)$$

$$(\phi > \phi_y) \quad \Delta = \Delta_y + l_p * (\phi - \phi_y) * \left(\frac{L}{2} - \frac{l_p}{2}\right) \quad (8-37)$$

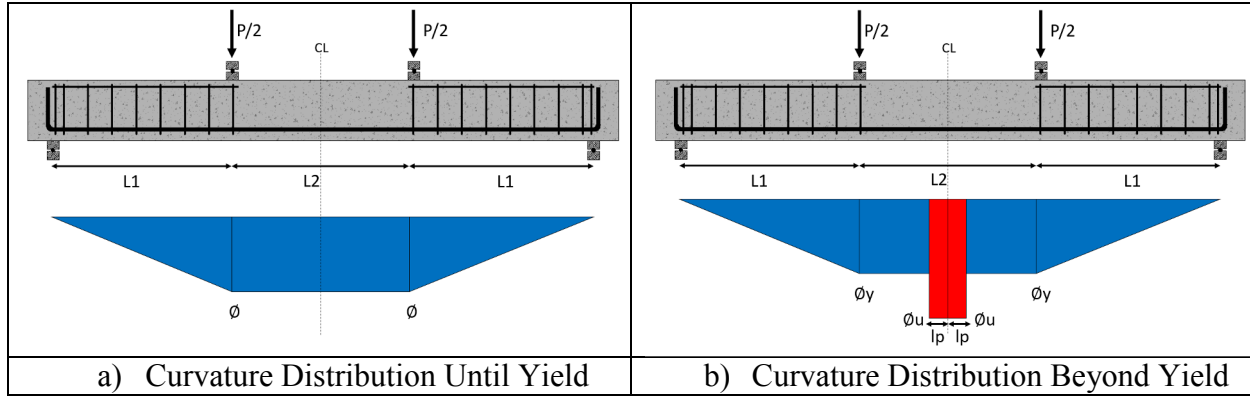


Figure 8-9 Curvature Distribution

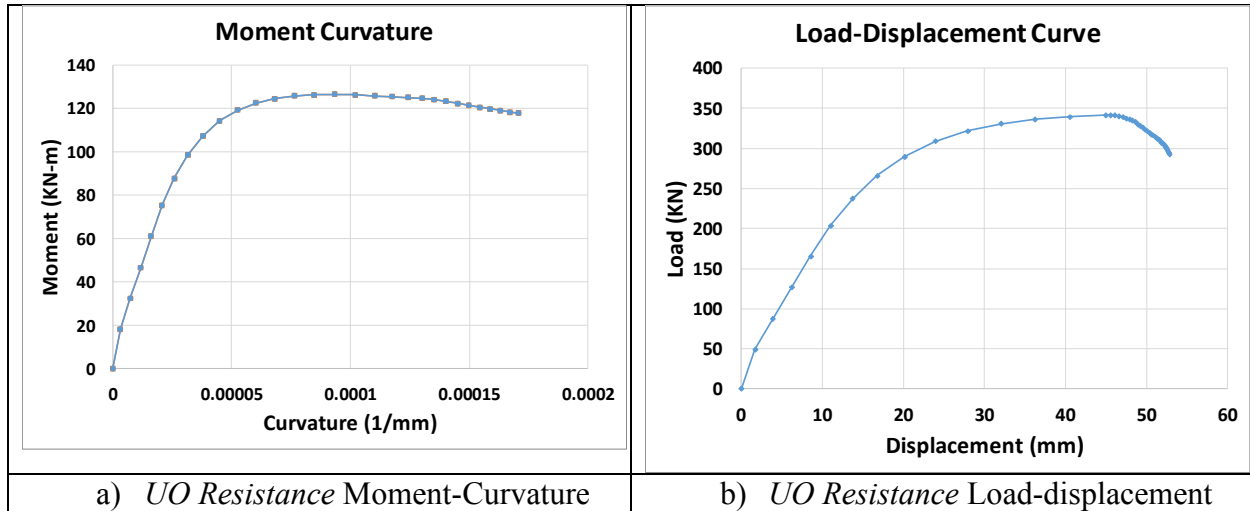


Figure 8-10 UO Resistance Sample Moment-Curvature and Load-Displacement Relationships

8.4.2. Plastic Hinge Length Models

Comprehensive research has been conducted since the mid 20th century on the plastic hinge length in reinforced concrete flexural elements. Thus, several models have been developed and proposed in the literature for determining the plastic hinge length based on certain parameters such as: concrete strength, shear span, reinforcement type and moment gradient. Park & Paulay (1975) have reported that unconfined concrete elements subjected to flexural loading can have plastic hinge lengths ranging between 40% and 240% of their cross-section effective depth. In the analysis of conventional reinforced concrete flexure members, the member's effective depth gives a reasonably good approximation of the plastic hinge length and can be used for simplicity of analysis (Caldwell, 2011). Some researchers have however noted that the plastic hinge length may be reduced for UHPC members (Burrell, 2012). In the current study, several cases of plastic hinge length are considered in the sensitivity analysis presented in the next section, including: 1.0d and 0.3d. Additionally, the following models were also considered.

Sawyer (1964)

Sawyer proposed an equation to describe the plastic hinge length as a function of the shear span (z) and the effective depth (d) of a flexural member as presented in equation (8-38).

$$l_p = 0.075z + 0.25d \quad (8-38)$$

Mattock (1967)

Mattock provided the following plastic hinge length equation. Based on experimental results, it was found that the plastic hinge length could be represented reasonable well by equation (8-39):

$$l_p = \frac{d}{2} + 0.05z \quad (8-39)$$

8.4.3. Single Degree of Freedom Dynamic Analysis

The single degree of freedom (SDOF) dynamic analysis conducted during this study was completed using the *UO Resistance* computer program. *UO Resistance* solves the equation of motion defined in Equation (8-40). As an alternative to the exact rigorous solution which is both complex and computationally demanding, modification factors are used to transform the real structure into an equivalent SDOF structure as depicted in Figure 8-11. In Equation (8-40), K_m is a mass factor and K_L is a load factor which is used to transform the real element parameters into an equivalent SDOF system. Furthermore, M is the total mass of the equivalent system, K is the stiffness of the element, $\ddot{y}$ is the acceleration at mid-span, y is the mid-span displacement of the beam and $F(t)$ is the idealized time forcing function.

$$K_m M \ddot{y}(t) + K_L K y(t) = K_L F(t) \quad (8-40)$$

The transformation factors; K_m for uniform mass and K_L for concentrated loads are obtained as presented in Equations (8-41) & (8-42), where m is the mass per unit length, φ is a designated shape function, L is the length of the element and F_r is the applied load.

$$K_M = \frac{\int_0^L m \varphi^2(x) dx}{mL} \quad (8-41)$$

$$K_L = \frac{\sum_0^r F_r \varphi_r}{\sum_0^r F_r} \quad (8-42)$$

$$K_{LM} = \frac{K_m}{K_L} \quad (8-43)$$

To simplify the solution of the approximate equation of motion, the K_{LM} factor presented in equation (8-43) can be implement in lieu of both K_M and K_L modification factors. For the dynamic test setup used in the current study, it was assumed that the systems mass, considering the load transfer device (267 kg) and the concrete element (182 kg) acted as a single uniform mass. A complete list of modification factors are presented in (Biggs, 1964). Based on the loading and boundary conditions, the ratio of mass to load factors (K_{LM} factors) employed are of 0.6 in the elastic region and 0.56 in the plastic region (the factors correspond to the case of uniform mass and four-point bending). The time-varying forcing function ($F(t)$) applied on the beam specimens is obtained by multiplying the reflected pressure-time function by the loaded area at the shock-tube opening ($A = 3.4\text{m}^2$ for the LTD used in this study) as shown in equation (8-44). In the analysis, an idealized triangular pressure-time function, having the same peak pressure (P_r) and impulse (I_r) recorded during dynamic testing was used ($P_r(1 - \frac{t}{t'_d})$, where the idealized time $t'_d = 2 I_r / P_r$).

$$K_{LM} M \ddot{y} + K y = A P_r (1 - \frac{t}{t'_d}) \quad (8-44)$$

In the above equation, K represents the resistance of the member; where the resistance function is generated by UO Resistance as described in section 8.4. The equivalent single degree of freedom system presented in Figure 8-11 is then solved numerically using the Acceleration Impulse Extrapolation Method outlined in UFC 3-340-02 (Department of Defense, 2008). The integration time step selected is one thousandth of the positive phase duration which is considerably below the minimum suggested value of one tenth of the natural period of vibration of the element as specified in the above reference. The dynamic analysis predicts the mid-span displacement for the specified reflected pressure and impulse up to maximum displacement but does not generate the resulting rebound displacement. *UO Resistance* results were validated using program *RCBlast*, developed by Jacques (2011).

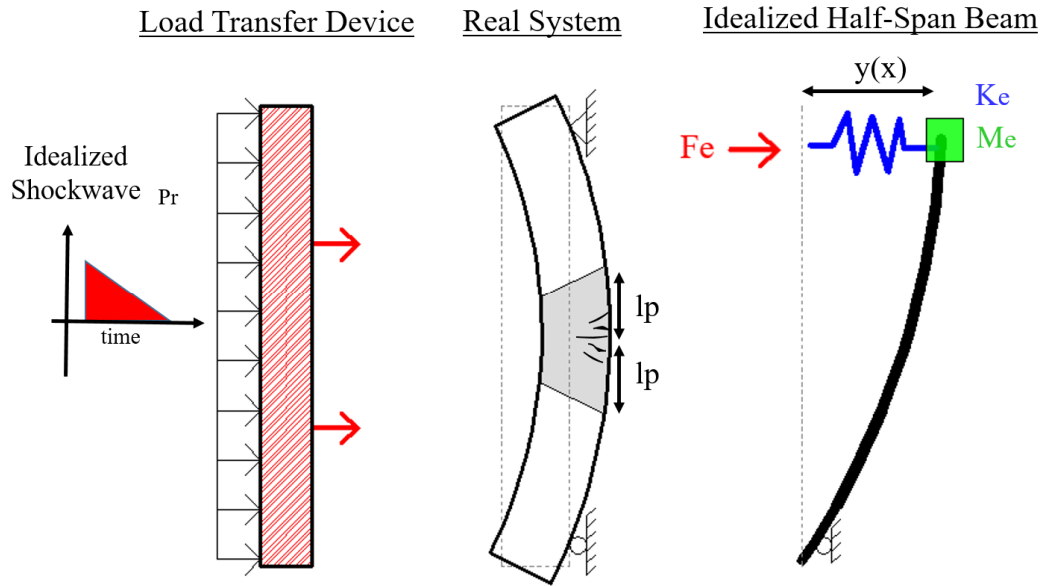


Figure 8-11 UO Resistance Equivalent SDOF System

The general analysis steps in *UO Resistance* are presented in Table 8-3:

Table 8-3 *UO Resistance* Analysis Procedure Summary

UQ Resistance																																																																																															
Step 1:	Input the element properties and cross-sectional properties using drop down lists and manual inputs.																																																																																														
Step 2:	Select the constitutive material models (concrete (compression), concrete (tension) and steel reinforcement). The user can select among the pre-defined models or input a custom stress-strain relationship in the custom input tab. Additionally, the user has the option of including dynamic increase models into the analysis based on the available models in the drop-down lists; custom models can also be introduced in the input tab.																																																																																														
<table><tr><th colspan="2">Section Properties</th></tr><tr><th colspan="2">Cross Sectional Properties</th></tr><tr><td>Width (mm)</td><td>125</td></tr><tr><td>Height (mm)</td><td>250</td></tr><tr><td>Length (mm)</td><td>2438.4</td></tr><tr><td>Un-Suported Length (mm)</td><td>2232</td></tr><tr><td>Loading Type</td><td>Four Point Bending</td></tr><tr><td>Shear Span (Z) (mm)</td><td>741</td></tr><tr><td>Cover to bar (mm)</td><td>40</td></tr><tr><td>Plastic Hinge Length (mm)</td><td>60.1425</td></tr><tr><td>Bar type</td><td>#6</td></tr><tr><td>Bar Diameter (tension) (mm)</td><td>19.05</td></tr><tr><td>Bar Area (tension) (mm²)</td><td>283.87</td></tr><tr><td>Effective Depth (tension) (mm)</td><td>200.475</td></tr><tr><td>Number of Bars</td><td>2</td></tr><tr><td>Total Reinforcement Area (mm²)</td><td>567.74</td></tr><tr><td>Resistance Curve</td><td>Full Displacements</td></tr><tr><td>Area of Compression Steel (mm²)</td><td>0</td></tr><tr><td>Effective Depth (Compression) (mm)</td><td>0</td></tr><tr><td>Axial Load "P" (kN)</td><td>0</td></tr><tr><td>Plastic Hinge Equation (mm)</td><td>0.3D</td></tr></table>		Section Properties		Cross Sectional Properties		Width (mm)	125	Height (mm)	250	Length (mm)	2438.4	Un-Suported Length (mm)	2232	Loading Type	Four Point Bending	Shear Span (Z) (mm)	741	Cover to bar (mm)	40	Plastic Hinge Length (mm)	60.1425	Bar type	#6	Bar Diameter (tension) (mm)	19.05	Bar Area (tension) (mm ²)	283.87	Effective Depth (tension) (mm)	200.475	Number of Bars	2	Total Reinforcement Area (mm ²)	567.74	Resistance Curve	Full Displacements	Area of Compression Steel (mm ²)	0	Effective Depth (Compression) (mm)	0	Axial Load "P" (kN)	0	Plastic Hinge Equation (mm)	0.3D	<table><tr><th colspan="4">Material Properties</th></tr><tr><th colspan="2">Concrete Properties</th><th colspan="2">Steel Reinforcement Properties</th></tr><tr><td>Concrete Type</td><td>UHPFRC-Hosinieh</td><td>Rebar type</td><td>MMFX</td></tr><tr><td>DIF Model</td><td>Saatcioglu-al</td><td>DIF Model</td><td>Custom-DIF</td></tr><tr><td>Tensile Stress-Strain Model</td><td>LOK-PEI</td><td>Modulus of Elasticity (nsr)</td><td>215000</td></tr><tr><td>vf (%) (i.e. 2)</td><td>2</td><td>yield strain (nsr)</td><td>0.002</td></tr><tr><td>lf (mm)</td><td>13</td><td rowspan="2">strain hardening strain (nsr)</td><td rowspan="2">0.015</td></tr><tr><td>df (mm)</td><td>0.2</td></tr><tr><td>E-fiber (Mpa)</td><td>200000</td><td>fy (Mpa) (nsr)</td><td>430</td></tr><tr><td>f'c(MPa)</td><td>140</td><td>fsh (Mpa) (nsr)</td><td>430</td></tr><tr><td>eo (NSC)</td><td>0.003</td><td>fu (Mpa) (nsr)</td><td>555</td></tr><tr><td>eu</td><td>0.0095</td><td></td><td></td></tr><tr><td>DIF Model (Tension)</td><td>Malvar-Ross</td><td></td><td></td></tr></table>		Material Properties				Concrete Properties		Steel Reinforcement Properties		Concrete Type	UHPFRC-Hosinieh	Rebar type	MMFX	DIF Model	Saatcioglu-al	DIF Model	Custom-DIF	Tensile Stress-Strain Model	LOK-PEI	Modulus of Elasticity (nsr)	215000	vf (%) (i.e. 2)	2	yield strain (nsr)	0.002	lf (mm)	13	strain hardening strain (nsr)	0.015	df (mm)	0.2	E-fiber (Mpa)	200000	fy (Mpa) (nsr)	430	f'c(MPa)	140	fsh (Mpa) (nsr)	430	eo (NSC)	0.003	fu (Mpa) (nsr)	555	eu	0.0095			DIF Model (Tension)	Malvar-Ross		
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lf (mm)	13	strain hardening strain (nsr)	0.015																																																																																												
df (mm)	0.2																																																																																														
E-fiber (Mpa)	200000	fy (Mpa) (nsr)	430																																																																																												
f'c(MPa)	140	fsh (Mpa) (nsr)	430																																																																																												
eo (NSC)	0.003	fu (Mpa) (nsr)	555																																																																																												
eu	0.0095																																																																																														
DIF Model (Tension)	Malvar-Ross																																																																																														
Step 1 (Screen Shot)		Step 2 (Screen Shot)																																																																																													

Step 3:	Once input parameters have been entered, user clicks “Clear Current Analysis”, followed by “Generate Material Stress-Strain Curves” and “Apply Dynamic Increase Factors” if required for analysis.	
	Section Properties Cross Sectional Properties Width (mm) 125 Height (mm) 250 Length (mm) 2438.4 Un-Supported Length (mm) 2232 Loading Type Four Point Bending Shear Span (Z) (mm) 741 Cover to bar (mm) 40 Plastic Hinge Length (mm) 60.1425 Bar type #6 Bar Diameter (tension) (mm) 19.05 Bar Area (tension) (mm²) 283.87 Effective Depth (tension) (mm) 200.475 Number of Bars 2 Total Reinforcement Area (mm²) 567.74 Resistance Curve Full Displacements Area of Compression Steel (mm²) 0 Effective Depth (Compression) (mm) 0 Axial Load "P" (kN) 0 Plastic Hinge Equation (mm) 0.3D Blast Parameters Peak Reflected Pressure (KPa) 97.66 Impulse (KPa-ms) 1078.44 Positive time duration (s) 0.02205603 Time step (Δt) (ms) 2.20566E-05 Area (m²) 3.4 Material Properties Concrete Properties Concrete Type UHPRC-Hosiniieh Tensile Stress-Strain Model LOK-PEI vf (%) (i.e. 2) 2 df (mm) 0.2 E-fiber (Mpa) 200000 ε_{cs} (NSCL) 0.003 DIF Model (Tension) Malvar-Ross Steel Reinforcement Properties Rebar type MMFX DIF Model Custom-DIF Modulus of Elasticity (nsr) 215000 yield strain (nsr) 0.002 strain hardening strain (nsr) 0.015 ultimate strain (nsr) 0.125 f_y (Mpa) (nsr) 430 f_{sh} (Mpa) (nsr) 430 f_u (Mpa) (nsr) 555 Generate Material Stress-Strain Curves Apply Dynamic Increase Factors Clear Current Analysis UO Resistance	
Step 4:	Next generated Stress-Strain curves are displayed.	
	Concrete Compression Stress-Strain Concrete Tension Stress-Strain Rebar Stress-Strain	
Step 5:	The moment curvature is then generated by clicking on “Generate Moment Curvature” button, the user is then prompted to input the yield curvature value in the green text box. Lastly, the load displacement curve can be obtained by clicking “Generate Load-Displacement”	
	Moment Curvature Load-Displacement Curve Generate Moment Curvature Generate Load-Displacement Curve	
Step 6:	Dynamic analysis: the user is prompted to input the blast parameters and click “Blast Parameters” button to solve for the maximum mid-span displacement of the element which is then displayed graphically.	
	Blast Parameters SDOF - Displacement Time History	

8.5. Static Analysis Results

This section presents the analysis results of seven beam specimens which were tested statically. The sectional analysis and the generation of the load-displacement curves was completed using *UO Resistance* as presented in Section 8.4. Section 8.5.1 presents a description of the modelling parameters and a summary of results. Furthermore, sections 8.5.2. and 8.5.3. present sensitivity analyses on the effect of plastic hinge length and high strength steel constitutive material model.

8.5.1. Summary of Analysis Results

Table 8-4 presents the default model combinations and plastic hinge lengths which provide the best analytical predictions when compared to experimental results for each specimen. The modelling parameters presented in Table 8-4 were selected based on the sensitivity analysis presented in sections 8.5.2. and 8.5.3. Constitutive material models for concrete in compression and tension are as presented in section 8.2. for plain concrete and UHPC. The normal-strength steel reinforcement material model is also as described in section 8.2. , while “MMFX₂ coupon” simply refers to the actual experimental stress-strain data obtained in Chapter 3. Lastly, the plastic hinge length utilized for each analysis is specified in Table 8-4. Comparisons of the analytical and experimental load-displacement relationships are plotted in Figure 8-14.

Table 8-4 Optimal Static Analysis Model Combinations

Specimen I.D	Concrete Models		Steel Model	Plastic Hinge Length [mm]
	Compression	Tension		
NSC-0%-15M-S	Hognestad	Linear	Park & Paulay	D [202]
NSC-0%-20M-S				D [200]
CRC-2%A-#4-S	Hosinieh	Lok & Pei	Park & Paulay	0.3D [61]
CRC-2%A-15M-S				0.3D [61]
CRC-2%A-20M-S				0.3D [60]
CRC-2%A-#4-HSR-S			MMFX ₂ Coupon	0.3D [61]
CRC-2%A-#6-HSR-S				0.3D [61]

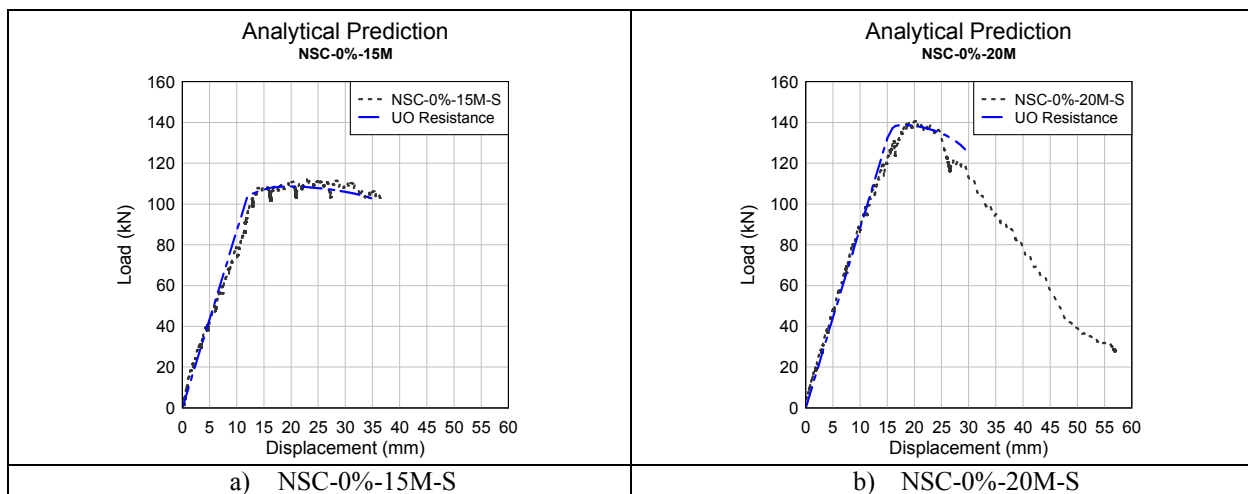


Figure 8-12 Static Analytical Result Summary (1/3): Control specimens

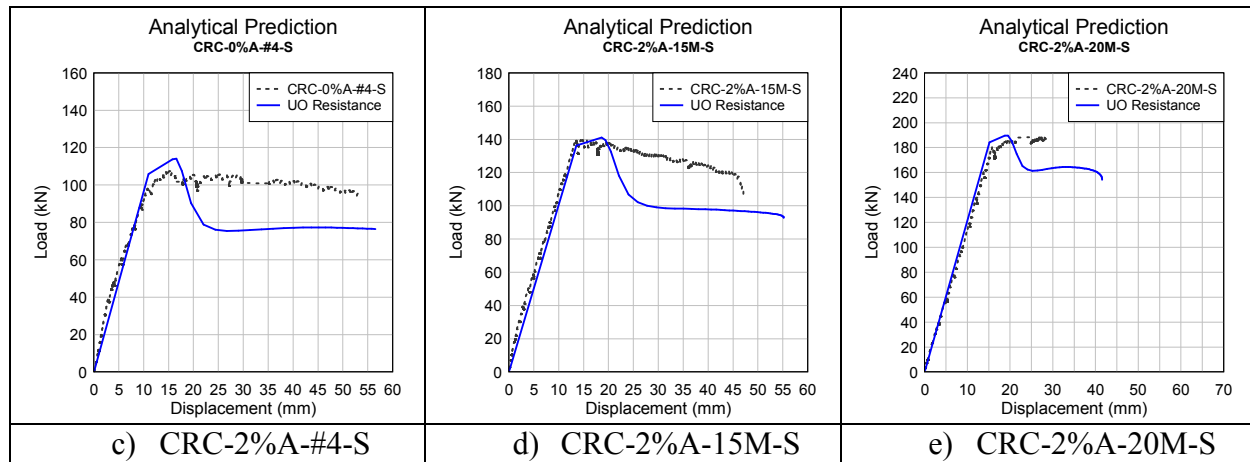


Figure 8-13 Static Analytical Result Summary (2/3): UHPC & NSR specimens

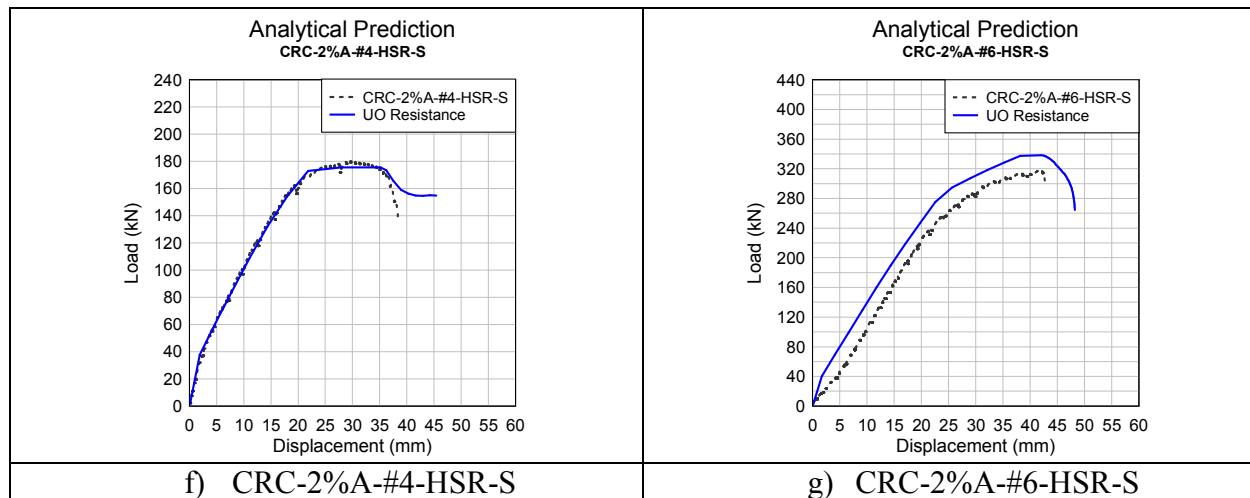


Figure 8-14 Static Analytical Result Summary (3/3): UHPC & HSR specimens

Statistical analysis results are presented in Table 8-5 and Table 8-6 present the ratios of analytical to experimental peak load ($\frac{P_{anls}}{P_{exp}}$) and analytical to experimental failure displacements ($\frac{\Delta_{anls}}{\Delta_{exp}}$). Table 8-5 and Table 8-6 also present the averages and standard deviations of the maximum load and failure displacement ratios.

Table 8-5 Static Analysis Results (Control Series)

Specimen I.D	Peak Load Ratio	Failure Disp. Ratio
NSC-0%-15M-S	96.8%	96.1%
NSC-0%-20M-S	98.6%	111.2%
Average [S.D.]	97.7% [0.9%]	103.6% [7.5%]

Table 8-6 Static Analysis Results (UHPC Series)

Specimen I.D	Peak Load Ratio	Failure Disp. Ratio
CRC-2%A-#4-S	106.2%	105%
CRC-2%A-15M-S	101.0%	118.3%
CRC-2%A-20M-S	100.2%	148.5%
CRC-2%A-#4-HSR-S	96.6%	117.7%
CRC-2%A-#6-HSR-S	106.5%	113.8%
Average [S.D.]	<u>102.1%</u> [3.7%]	<u>120.6%</u> [14%]

8.5.2. Sensitivity Analysis – High Strength Steel Model

The effect of the tension steel model on the predictions of the UHPC beams with HSR is examined by comparing the results using two high strength steel stress-strain relationships. Case 1 uses the ACI ITG-6 constitutive model while Case 2 uses the experimental MMFX₂ stress-strain data obtained from the coupons. The concrete compression and tension models remain constants as depicted in Table 8-7. The resulting resistance curves from case 1 and case 2 are plotted in Figure 8-15, followed by a summary of the peak load and failure displacement ratios in Table 8-8. Discussion on the effect of this parameter is provided in Section 8.5.4.

Table 8-7: Static Sensitivity Analysis: High Strength Reinforcement Models

	Concrete Compression Model	Concrete Tension Model	HSR Model
Case 1	Hosinieh	Lok & Pei	ACI ITG-6
Case 2			MMFX ₂ Coupon

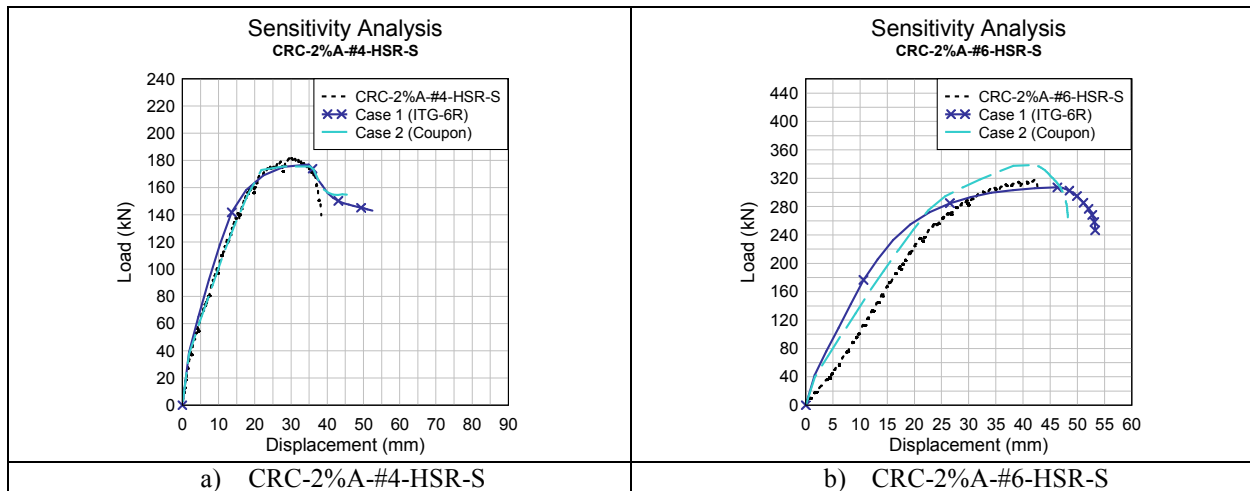


Figure 8-15 Static Sensitivity Analysis Results: High Strength Reinforcement Model (Series 1)

Table 8-8 Static Sensitivity Analysis Results: High Strength Reinforcement Model (Series 1)

Specimen I.D	Peak Load Ratio	Failure Disp. Ratio	Peak Load Ratio	Failure Disp. Ratio
	Case 1		Case 2	
CRC-2%A-#4-HSR-S	0.97	1.35	0.96	1.17
CRC-2%A-#6-HSR-S	0.97	1.25	1.06	1.13
Average	<u>0.97</u>	<u>1.30</u>	<u>1.01</u>	<u>1.15</u>

8.5.3. Sensitivity Analysis – Plastic Hinge Length

With the objective of optimizing the analytical results, a sensitivity analysis on the effect of plastic hinge length was conducted. For the control series beams, 3 cases were considered; case 1 uses the effective depth for the plastic hinge length ($l_p = D$), case 2 utilizes the plastic hinge length relation proposed by Mattock (1967) and case 3 the relation proposed by Sawyer (1964). For the UHPC beams, the same three cases were studied, with the addition of case 4 which consists of a plastic hinge length equal to 30%D. The concrete compression and tension models remain constant as depicted in Table 8-9 and Table 8-11. The sensitivity analysis results for the control series and UHPC series beams are presented graphically in Figure 8-18 through Figure 8-22. The resulting peak load and displacement ratios are presented in Table 8-10 and Table 8-12 for the control and UHPC series, respectively. Discussion on the effect of this parameter is provided in Section 8.5.4.

Table 8-9 Static Sensitivity Analysis: Plastic Hinge Length Models (Series 1)

	Concrete Compression Model	Concrete Tension Model	Steel Reinforcement Model	Plastic Hinge Length
Case 1*	Hognestad	Linear	Park & Paulay (1975)	D
Case 2				Mattock (0.7D)
Case 3				Sawyer (0.5D)

*Default case presented in Section 8.5.1

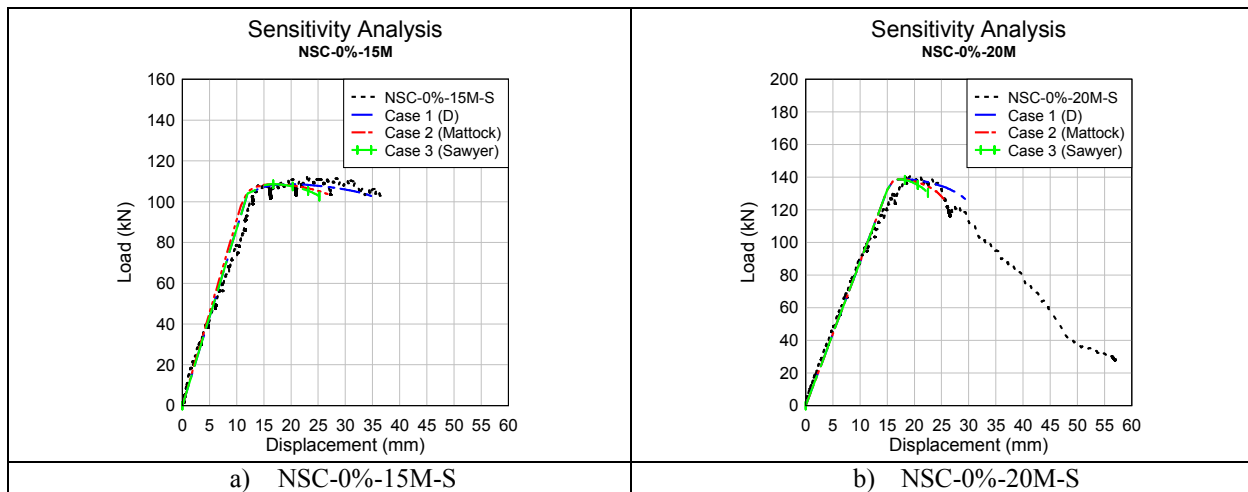


Figure 8-16 Static Sensitivity Analysis Results: Plastic Hinge Length (Series 1: Control beams)

Table 8-10 Static Sensitivity Analysis Results: Plastic Hinge Length (Series 1: Control beams)

Specimen I.D	Case 1		Case 2		Case 3	
	Peak Load Ratio	Failure Disp. Ratio	Peak Load Ratio	Failure Disp. Ratio	Peak Load Ratio	Failure Disp. Ratio
NSC-0%-15M-S	0.97	0.96	0.97	0.76	0.97	0.70
NSC-0%-20M-S	0.99	1.11	0.99	0.96	0.99	0.88
Average:	<u>0.98</u>	<u>1.04</u>	<u>0.98</u>	<u>0.86</u>	<u>0.98</u>	<u>0.79</u>

Table 8-11 Static Sensitivity Analysis: Plastic Hinge Length Models (Series 2 & 3: UHPC beams)

	Concrete Compression Model	Concrete Tension Model	Steel Model (NSR/MMFX)	Plastic Hinge Length
Case 1	Hosinieh	Lok & Pei	Park & Paulay / ITG-6	D
Case 2				Mattock (0.7D)
Case 3				Sawyer (0.5D)
Case 4*				0.3D

*Default case presented in Section 8.5.1

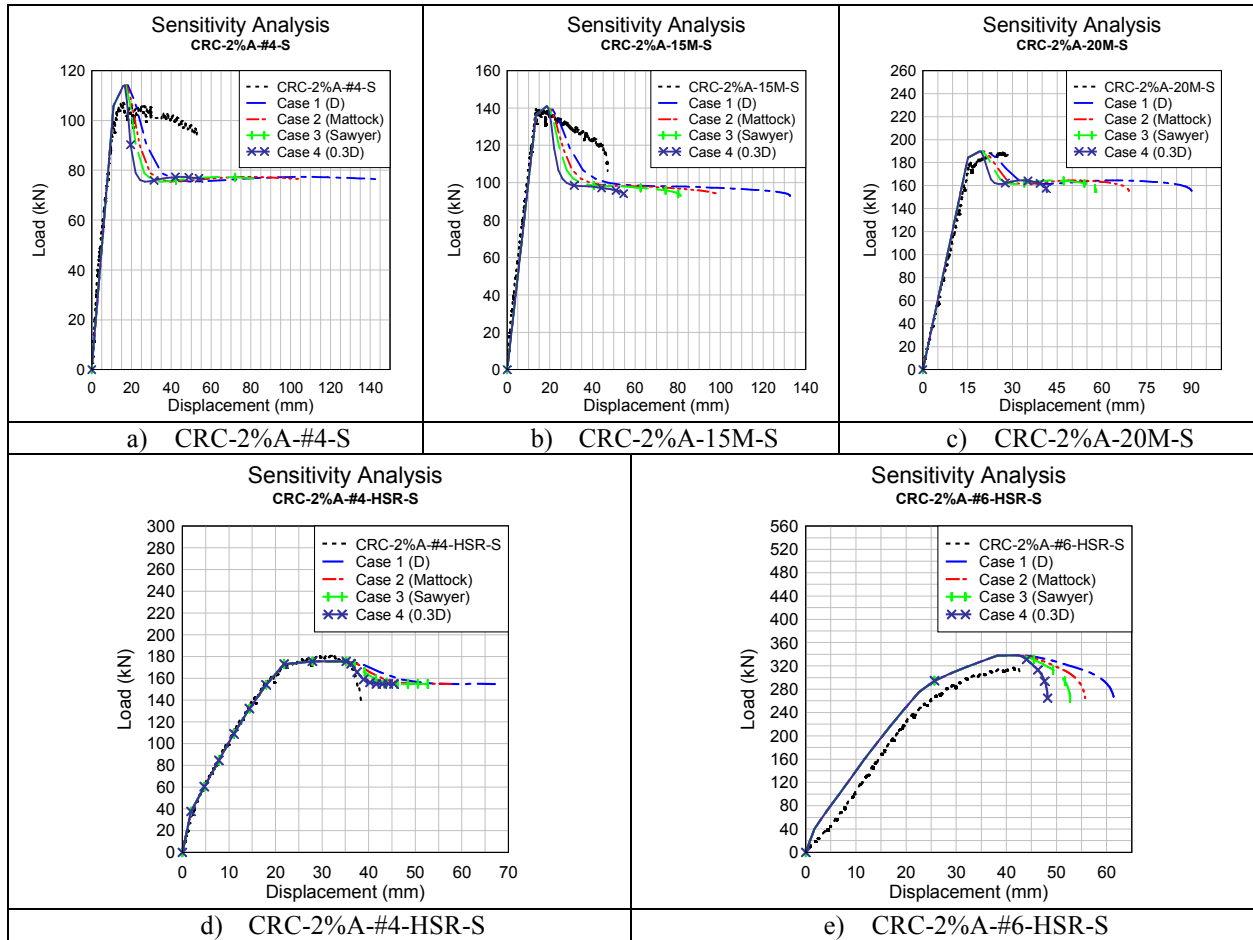


Figure 8-17 Static Sensitivity Analysis Results: Plastic Hinge Length (Series 2 & 3: UHPC beams)

Table 8-12 Static Sensitivity Analysis Results: Plastic Hinge Length (Series 2 & 3)

Specimen I.D	Case 1		Case 2		Case 3		Case 4	
	Peak Load Ratio	Failure Disp. Ratio	Peak Load Ratio	Failure Disp. Ratio	Peak Load Ratio	Failure Disp. Ratio	Peak Load Ratio	Failure Disp. Ratio
CRC-2%A-#4-S	1.06	2.63	1.06	1.93	0.60	1.52	1.06	1.05
CRC-2%A-15M-S	1.01	2.81	1.01	2.10	1.01	1.72	1.01	1.18
CRC-2%A-20M-S	1.00	3.15	1.00	2.46	1.00	2.05	1.00	1.49
CRC-2%A-#4-HSR-S	0.97	1.74	0.97	1.47	0.97	1.37	0.97	1.18
CRC-2%A-#6-HSR-S	1.07	1.45	1.07	1.31	1.07	1.24	1.07	1.14
Average:	<u>1.02</u>	<u>2.36</u>	<u>1.02</u>	<u>1.86</u>	<u>1.02</u>	<u>1.58</u>	<u>1.02</u>	<u>1.21</u>

8.5.4. Discussion of Static Analysis Results

Control Beams

The analysis results for the control series specimens (NSC-0%-15M-S and NSC-0%-20M-S), based on the constitutive material models and plastic hinge lengths presented in Table 8-4, prove to generate good predictions of the response of the beams with conventional concrete and reinforcement. By visual inspection of Figure 8-13, the analysis closely predicts the actual stiffness and peak loads of the beams. The analytical post-peak response of specimen NSC-0%-15M-S follows an almost identical trajectory to failure displacement when compared to the test data. Similarly, for specimen NSC-0%-20M-S, the peak load is well predicted and the analysis captures the post-peak descending branch with reasonable accuracy when compared to the test data. The average peak load ratio ($\frac{P_{anls}}{P_{exp}}$) and failure displacement ratio ($\frac{\Delta_{anls}}{\Delta_{exp}}$) for the two control specimens are 97.7% and 103.6%. As depicted in Table 8-9, three plastic hinge length cases were considered for series 1 specimens and by simple visual inspection of the resulting resistance curves obtained in Figure 8-16 it is clear that utilizing the effective depth ($l_p = D$) is most appropriate. The plastic hinge length has a more critical effect on the post-yield displacement, such that using $l_p = D$ (Case 1) yields an average displacement ratio of 104% whereas the plastic hinge length relationships proposed by Mattock (Case 2: 0.7D) and Sawyer (Case 3: 0.5D) tend to underestimate the failure displacement, resulting in average displacement ratios of 86% and 79%, respectively.

UHPC Beams

Analytical results for beams designed with UHPC and normal strength reinforcement (series 2: CRC-2%A-#4-0, CRC-2%A-15M-S, CRC-2%A-20M-S) prove to also be representative of the experimental load-displacement relationships. By visual inspection of Figure 8-13, the series 2 analytical stiffness' are almost identical to the experimental results. Moreover, the peak load resisted and corresponding displacement are also well predicted. However, for all three series 2 specimens, the post-peak displacement plateau obtained in the analysis is inferior to that of the experimental results by a margin of approximately 30%. This difference can be attributed to the rapidly descending post-peak branches found in both the Hosinieh (2015) concrete compression model and the Lok & Pei (1998) concrete tension model. Analytical results for series 3 specimens (CRC-2%A-#4-S and CRC-2%A-#6-S), also correlate well with the experimental results as shown in Figure 8-14. The resistance curve for specimen CRC-2%A-#4-S is practically a mirror image of the experimental load-displacement curve as depicted in Figure 8-14. The peak load and failure displacement are also representative of actual behavior for the CRC-2%A-#6-S specimen, however there is a slight over-prediction in peak resisted load. Considering all five UHPC in series 2& 3, the average peak load ratio ($\frac{P_{anls}}{P_{exp}}$) is found to be 102%, with a S.D. of 3.7% (Table 8-6).

Two sensitivity analyses were considered for the UHPC beams. The first sensitivity study examined the effect of high strength steel model in series 3. Examining the results in Figure 8-15 and Table 8-8 Static Sensitivity Analysis Results: High Strength Reinforcement Model (Series 1), it can be seen that the MMFX₂ coupon stress-strain data (Case 2) results in predictions that are slightly more representative than those obtained using the ACI ITG-6 model (Case 1). Nonetheless, the differences are minor and the ITG-6 model provides adequate predictions indicating that this

model is suitable for beams designed with MMFX reinforcement. The minor advantage of the MMFX₂ coupon data was a more representative stiffness and improved prediction of peak load and failure displacement, with analytical-to-experimental ratios of 1.01 and 1.15 when compared to the ITG-6 model results of 0.97 and 1.30, respectively.

The second sensitivity study examined the effect of plastic hinge length on the predictions of the series 2 and 3 UHPC beams. Four cases of hinge length were considered ranging from D to $0.3D$ (refer to Table 8-11). The results in Figure 8-17 and Table 8-12 establish that using 30% of the effective depth yields the most accurate results in terms of the prediction of failure displacement (peak loads are not affected). In fact, all other plastic hinge length models including; using the effective depth ($l_p = D$), Mattock and Sawyer's proposed models ($l_p = 0.7D$ and $0.5D$) result in significant over-prediction of the maximum (failure) displacements. Using $l_p = 0.3D$ (Case 4: default) yielded an average displacement ratio of 1.21, while the ratios are 1.52 for Sawyer's model (Case 2), 1.86 for Mattock's model (Case 3) and 2.36 when using $l_p = D$ (Case 1).

8.6. Dynamic Analysis Results

This section presents the dynamic analysis results obtained using *UO Resistance* and the procedure described in section 8.4. The analysis results are summarized in three categories based on the beam specimen series (Control, UHPC & NSR, UHPC & HSR). Results are compared in terms of maximum mid-span displacement at each blast. Table 8-13 and Table 8-14 summarize the default constitutive material model combinations and plastic hinge length which were selected for the analysis of each specimen, based on the sensitivity analysis conducted in Section 8.7.

The analysis results are summarized in tabular and graphical format. The tables summarize the shockwave parameters utilized in the analysis, including the peak reflected pressure (P_r), the reflected impulse (I_r) and time duration of the idealized triangular blast wave (t_p). The tables also report the experimental maximum mid-span displacement (δ_{exp}), the analytical maximum mid-span displacement (δ_{anls}) as well as the analytical-to-experimental mid-span displacement ratio ($\square_{anls} / \square_{exp}$) and resulting error (% Error). Moreover, statistical results of the displacement ratios are also displayed (average ratio, standard deviation, coefficient of variance and error considering all blasts and individual blast results). Sample comparison of analytical resistance curves versus experimental resistance curves (as obtained from the dynamic load cells) are plotted, in addition to sample displacement time-histories for each beam series.

Table 8-13 Material Model Combinations and Dynamic Increase Factors (1/2): Series 1

Specimen I.D	Concrete Models		Steel Model [DIF _y /DIF _u]	Plastic Hinge Length [mm]
	Compression [DIF]	Tension [DIF]		
NSC-0%-#4-S	Hognestad CEB [1.34]	Linear Malvar & Ross [1.52]	Park & Paulay Saatcioglu et al. [1.3/1.1]	D [204]
NSC-0%-15M-S				D [202]
NSC-0%-20M-S				D [200]

Table 8-14 Material Model Combinations and Dynamic Increase Factors (2/2): Series 2 and 3

Specimen I.D	Concrete Models		Steel Model [DIF _y /DIF _u]	Plastic Hinge Length [mm]
	Compression [DIF]	Tension [DIF]		
CRC-2%A-#4-S	Hosinieh Zhang & Mindess [1.20]	Lok & Pei Malvar & Ross [1.13]	Park & Paulay Saatcioglu et al. [1.3/1.1]	0.3D [61]
CRC-2%A-15M-S				0.3D [61]
CRC-1%B-15M-0				0.3D [61]
CRC-2%B-15M-0				0.3D [61]
CRC-3%B-15M-0				0.3D [61]
CRC-2%A-20M-S				0.3D [60]
CRC-2%A-#4-HSR-S	Hosinieh [1.0]	Lok & Pei [1.0]	MMFX ₂ Coupon [1.0]	0.3D [61]
CRC-2%A-#5-HSR-S				0.3D [61]
CRC-2%A-#6-HSR-S				0.3D [60]

8.6.1. Dynamic Analysis Results – Control (Series 1)

The dynamic analysis results for the Series 1 (Control) specimens are presented in Table 8-15 alongside the analytical-to-experimental mid-span displacement ratios ($\square_{\text{anls}} / \square_{\text{exp}}$) and corresponding percentage of error. Table 8-16 presents various statistics relating to the predictions for all blasts combined as well as each individual blast number. The analytical to experimental mid-span displacement ratios are graphically presented in Figure 8-18. Finally, sample comparisons of analytical and experimental dynamic resistance curves and displacement time-histories are plotted in Figure 8-19.

The results displayed in Table 8-15 and Figure 8-18 suggest a general agreement between the analytical and experimental results. Based on sensitivity analysis the optimized model combinations used in the analysis include: Hognestad (1951) in combination with the CEB DIF model ($DIF_c=1.34$) for concrete in compression; linear tension model (with cracking stress = $0.33\sqrt{f'_c}$) complimented by the Malvar & Ross DIF model ($DIF_t=1.52$) for concrete in tension; Park & Paulay (1975) tensile steel reinforcement model alongside the Saatcioglu et al. DIF model ($DIF_y=1.3$; $DIF_u=1.1$) for the longitudinal steel reinforcement. Lastly, a plastic hinge length equivalent to the effective depth (D) of the section was found to be most accurate for the control series specimens.

The overall average mid-span displacement ratio for all blasts is 0.77 (see Table 8-16), which is reasonable considering the assumptions and simplifications of the SDOF analysis. However, a closer assessment of individual blast results displays an interesting trend; it is found that the mean ratio is $\frac{\delta_{anls}}{\delta_{exp}} = 0.95$ for blast 1 and $\frac{\delta_{anls}}{\delta_{exp}} = 0.81$ for blast 2, however, as the sequential tests continue and the number of repeated blast tests increase towards blast 3 and 4, the accuracy of the analytical results significantly decreases, with mid-span displacement ratios of 0.68 and 0.31, respectively. The results show that the analysis is over-predicting the resistance of the beams at these blasts, resulting in underestimation of the maximum mid-span displacements. This can be explained by the fact that the beams crack and get damaged after each blast, and this accumulation of damage and resulting stiffness degradation is not accounted for in the analysis procedure which does not consider the effects of repeated blast loading. This explains why the initial predictions (blast 1 and 2) are better, while the accuracy drops as the number of imposed shockwaves increases (e.g. at blast 4).

This can further be validated by comparing the sample experimental resistance curves in Figure 8-19(a), which display reduced stiffness when comparing blast 3 vs. blast 2. It can also be seen that the analytical dynamic resistance curve generated using *UO Resistance* corresponds quite well with the experimental load-displacement relationship experienced during blast 2, but is significantly stiffer than that for blast 3. This underlines the need for a hysteretic model capable of incorporating the stiffness degradation due to accumulated damage in the analysis of beams tested under repeated blast loading. Comparison of the displacement time-histories in Figure 8-19 (b)-(d) also show the improved degree of analytical accuracy for the first two shockwaves, followed by a considerable under-estimation of mid-span displacement for later blasts.

Table 8-15 Summary of Dynamic Analysis (Series 1)

Beams	Idealized Shockwave Properties			Maximum Mid-Span Displacement			
ID	Blast	P _r	I _r	δ_{exp}	δ_{anls}	$\bar{\delta}_{anls} / \bar{\delta}_{exp}$	% Error
	#	(kPa)	(kPa-ms)	(mm)	(mm)		
NSC-0%-#4-0	1	21.1	240.12	11.84	11.46	0.97	-3.21%
	2	35.4	364.72	27.39	21.02	0.77	-23.26%
	3	46.92	653.71	67.72	43.82	0.65	-35.29%
NSC-0%-15M-S	1	19.84	232.58	10.04	10.5	1.05	4.58%
	2	35.39	354.06	19.32	16.39	0.85	-15.17%
	3	51.53	486.21	42.05	28.31	0.67	-32.68%
NSC-0%-20M-S	1	24.98	224.38	10.77	9.11	0.85	-15.41%
	2	36.39	371.39	17.34	14.17	0.82	-18.28%
	3	46.7	524.38	29.54	21.61	0.73	-26.84%
	4	71.8	652.18	108.69	34.1*	0.31	-68.63%

*Maximum resistance curve displacement exceeded (blow out failure).

Table 8-16 Analysis Statistics (Series 1)

Blast #	Mean ratio ($\bar{\delta}_{anls} / \bar{\delta}_{exp}$)	standard deviation	Coefficient of Variance	% Error
1 - 4	0.77	0.20	0.26	-23.42%
1	0.95	0.10	0.11	-4.68%
2	0.81	0.04	0.05	-18.90%
3	0.68	0.04	0.06	-31.60%
4	0.31	n/a	n/a	-68.63%

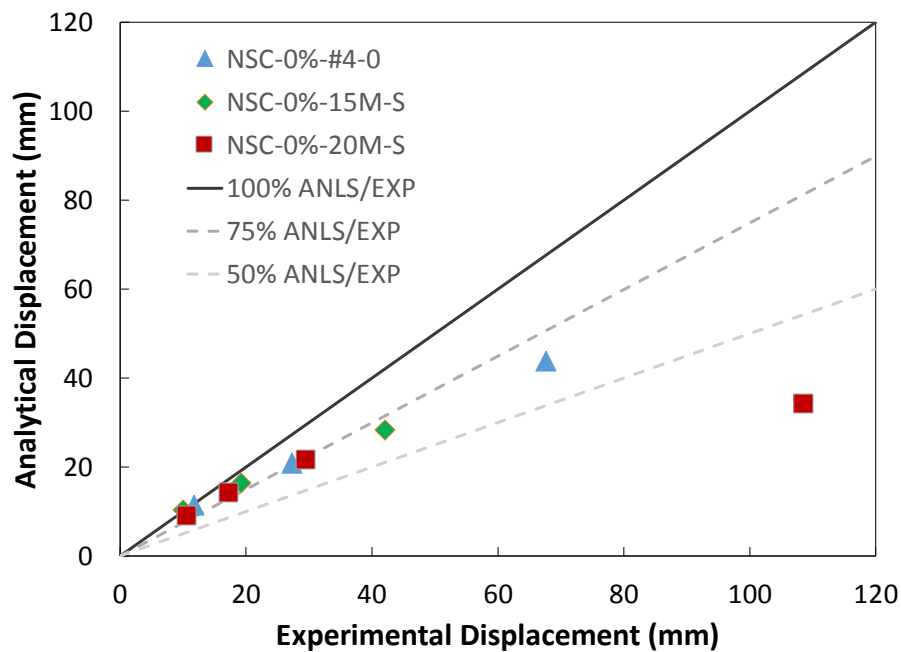


Figure 8-18 Experimental vs. Analytical Displacements (Series 1)

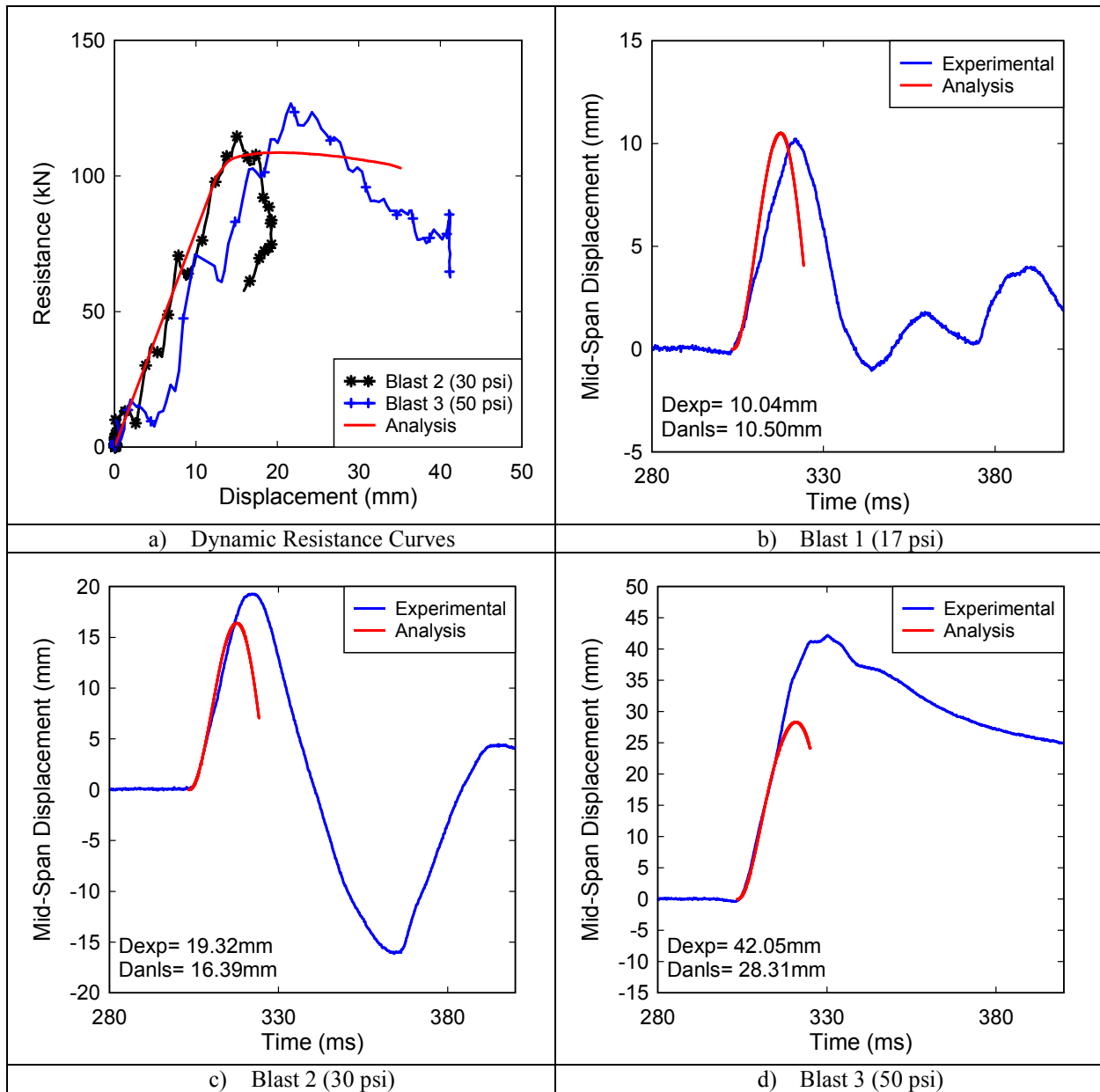


Figure 8-19 Sample Resistance Curves and Displacement Time-Histories (NSC-0%-15M-S)

8.6.2. Dynamic Analysis Results – UHPC & NSR (Series 2)

The dynamic analysis results generated for series 2 (UHPC and normal strength reinforcement) are presented in Table 8-17 alongside the analytical-to-experimental mid-span displacement ratios ($\square_{anls} / \square_{exp}$) and corresponding errors. Table 8-18 also presents various statistics relating to all blasts combined as well as each individual blast. The mid-span displacement ratios are presented graphically in Figure 8-20. Finally, sample comparisons of the analytical and experimental resistance curves and displacement time-histories are plotted in Figure 8-21. The experimental resistance curve was obtained from the dynamic load-cells during testing.

The results displayed in Table 8-17 and Figure 8-20 display a general agreement between the analytical and experimental results. Based on sensitivity analysis the optimized model combinations used in the analysis included: Hosinieh (2015) relationship in combination with the Zhang & Mindess dynamic increase factor model ($DIF=1.20$ for UHPC in compression; Lok & Pei (1998) model complimented by the Malvar & Ross DIF model ($DIF=1.13$) for UHPC in tension; and the Park & Paulay (1975) steel model alongside the Saatcioglu et al. DIF model ($DIF_y=1.3$; $DIF_u=1.1$) for longitudinal reinforcement. Lastly, a plastic hinge length equivalent to 30% of the effective depth (D) of the section was found to be most effective for the ultra-high performance concrete specimens.

The overall average mid-span displacement ratio for all blasts is of 0.94 (see Table 8-18), demonstrating that the models implemented and analysis procedure generate reasonable predictions of dynamic response. A more detailed assessment of individual blast 1 through 5 displays similar results to that observed in the control series, where the accuracy reduces at later blasts. The mean ratios for blast 1, 2 and 3 are $\frac{\delta_{anls}}{\delta_{exp}} = 1.20, 1.00$ and 0.94, respectively. However, as the sequential tests continue and the number of repeated blast tests increase towards blast 4 and 5, the analytical accuracy significantly decreases with mid-span displacement ratios of 0.78 and 0.6, respectively. Once again, the analytical results show that the analysis is over-predicting beam resistance, which results in an underestimation of the maximum mid-span displacement experienced by the beam specimens. The consistency in the trends observed in series 1 and series 2 further indicate that the repeated blast testing results in reduced stiffness which is not captured in the analysis. Analytical predictions for blasts 4 and 5 should therefore be conducted using a hysteretic model which accounts for stiffness degradation due to repeated loading.

A comparison of analytical and experimental resistance curves for specimen CRC-2%A-20M-0 is displayed in Figure 8-21 (a). The experimental resistance curve recorded at blast 5 shows reduced stiffness when compared to the one predicted by the analysis. It can also be seen that the dynamic resistance curve generated by *UO Resistance* corresponds very well to that of blast 3 before the stiffness degradation occurs. The effects of repeated loading can also be seen through comparison of the displacement time-histories presented in (b) through (f) as the analytically predicted maximum mid-span displacements show greater inaccuracies with increasing blast loads.

Table 8-17 Summary of Dynamic Analysis (Series 2)

Beams	Idealized Shockwave Properties			Maximum Mid-Span Displacement			
ID	Blast	P _r	I _r	δ_{exp}	δ_{anls}	$\bar{\delta}_{anls} / \bar{\delta}_{exp}$	% Error
	#	(kPa)	(kPa-ms)	(mm)	(mm)		
CRC-2%A-#4-0	1	28.23	244.87	5.7	8.64	1.52	51.6%
	2	35.65	377.56	12.65	13.51	1.07	6.8%
	3	53.71	534.98	26.36	21.38	0.81	-18.9%
	4	63.4	683.39	118.05	43.13	0.37	-63.5%
CRC-2%A-20M-0	1	20.74	274.55	7.73	8.23	1.06	6.5%
	2	36.02	365.19	11.41	12.81	1.12	12.3%
	3	47.77	547.7	21.48	18.52	0.86	-13.8%
	4	61.41	710.4	34.06	24.72	0.73	-27.4%
	5	83.18	859.01	56.67	34.28	0.60	-39.5%
CRC-2%A-15M-S	1	23.64	230.74	10.22	8.14	0.80	-20.4%
	2	34.54	346.64	12.89	12.67	0.98	-1.7%
	3	47.13	530.72	22.07	19.76	0.90	-10.5%
	4	60.42	673.49	41.97	35.69	0.85	-15.0%
CRC-2%A-20M-S	1	23.56	234.27	4.55	8.23	1.81	80.9%
	2	36.64	367.66	10.79	12.81	1.19	18.7%
	3	54.98	549.82	16.81	18.52	1.10	10.2%
	4	65.61	708.12	25.86	24.72	0.96	-4.4%
	5	77.24	831.8	48.48	34.28	0.71	-29.3%
CRC-1%B-15M-0	1	21.8	236.39	8.02	7.95	0.99	-0.9%
	2	34.54	366.43	14.25	12.8	0.90	-10.2%
	3	48.84	509.54	21.06	24.87	1.18	18.1%
	4	61.66	668.55	40.84	45.85	1.12	12.3%
	5	75.51	824.38	247.11	120.99	0.49	-51.0%
CRC-2%B-15M-0	1	23.14	229.32	8.94	8.95	1.00	0.1%
	2	N/A	N/A	18.52	13.94	0.75	-24.7%
	3	53.07	566.78	29.54	23.55	0.80	-20.3%
	4	68.58	695.75	62.04	41.66	0.67	-32.8%
CRC-3%B-15M-0	1	23.49	223.67	6.25	7.82	1.25	25.1%
	2	37.13	372.61	13.26	12.17	0.92	-8.2%
	3	55.4	530.74	20.01	19.05	0.95	-4.8%
	4	70.24	706.71	45.65	34.78	0.76	-23.8%

Table 8-18 Analysis Statistics (Series 2)

Blast #	Mean ratio ($\bar{\delta}_{anls} / \bar{\delta}_{exp}$)	Standard deviation	Coefficient of Variance	% Error
1 - 5	0.94	0.29	0.31	-6.18%
1	1.20	0.27	0.22	19.64%
2	1.00	0.27	0.27	0.20%
3	0.94	0.28	0.29	-5.86%
4	0.78	0.26	0.33	-21.81%
5	0.60	0.10	0.18	-39.95%

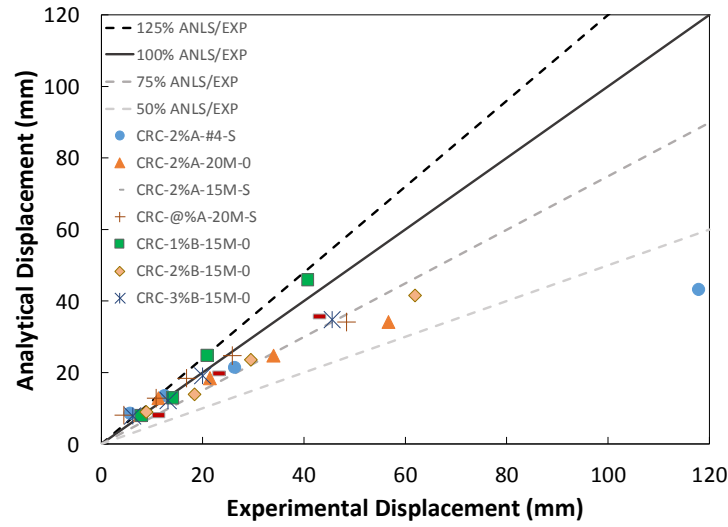


Figure 8-20 Experimental vs. Analytical Displacements (Series 2)

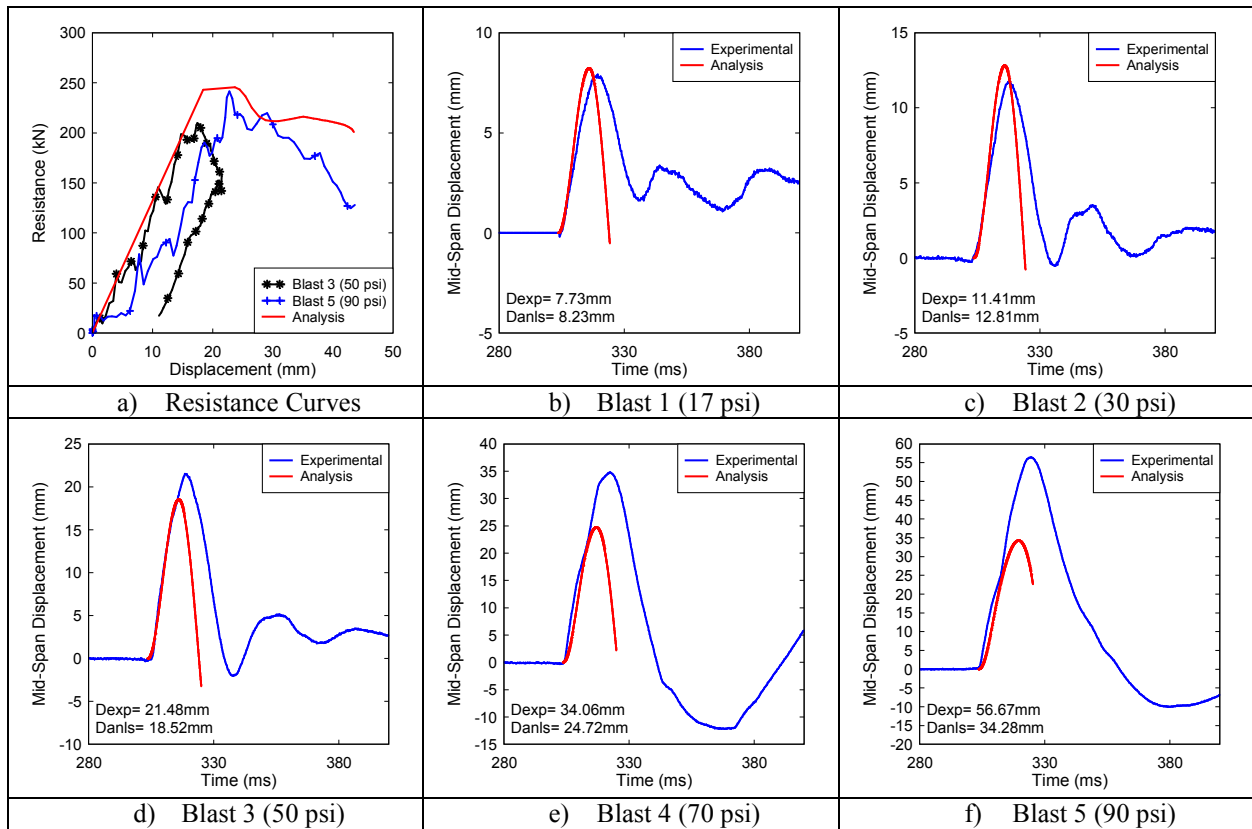


Figure 8-21 Sample Resistance Curves and Displacement Time-Histories (CRC-2%A-20M-0)

8.6.3. Dynamic Analysis Results – UHPC & HSR (Series 3)

The dynamic analysis results generated for the Series 3 specimens (UHPC and high strength reinforcement) are presented in Table 8-19 alongside the analytical-to-experimental mid-span displacement ratios ($\delta_{anls} / \delta_{exp}$) and corresponding error. Table 8-20 also presents various statistics relating to the analysis for all blasts combined as well as each individual blast. The displacement ratios are presented graphically in Figure 8-20. Finally, sample comparisons of the analytical and experimental resistance curves and displacement time-histories are plotted in Figure 8-23. The experimental resistance curve was obtained as described in Section 8.6.1.

The results displayed in Table 8-19 and Figure 8-22 display a general agreement between the analytical experimental results. As discussed later in the sensitivity study, it was found that applying dynamic increase factors of 1.0 to all material models provided the most accurate predictions. The material constitutive models for UHPC in compression and UHPC in tension remain as presented previously (Hosinieh and Lok & Pei), while the high strength steel reinforcement was modeled using the experimentally obtained MMFX₂ stress-strain relationships. Lastly, consistent with Series 2, the plastic hinge length was taken equal to 30% of the effective depth (D) for the ultra-high performance concrete beams in Series 3.

The overall average mid-span displacement ratio for all blasts is found to be 0.85 (see Table 8-20), an indicator that the models implemented and analysis procedure generate reasonable dynamic predictions. A more detailed assessment of individual blast 1 through 7 displays similar trends to those observed in Series 1 and 2. It is found that the mean displacement ratios for blasts 1, 2 and 3 are $\frac{\delta_{anls}}{\delta_{exp}} = 1.11, 0.91 \text{ \& } 0.94$, respectively which are quite acceptable, however, as the sequential tests continue and the number of repeated blast tests increase towards blast 4, 5, 6 and 7 the accuracy of the analysis significantly decreases, with mid-span displacement ratios of 0.65, 0.82, 0.74 and 0.42, respectively. Once again, as the stiffness of the beam specimens softens due to repeated loads, the analysis begins to underestimate the maximum mid-span displacements observed in the experiments. The consistency of the trends in the results of series 1, 2 and 3 further demonstrates that the analysis was not able to capture the degradation in stiffness caused by accumulated damage.

Sample dynamic experimental resistance curves for specimen CRC-2%A-#5-HSR-S are displayed in Figure 8-23(a), where the beam shows reduced stiffness when comparing blast 5 and 4. It can also be seen that the stiffness of dynamic resistance curve generated by *UO Resistance* corresponds reasonably well to that of blast 4 before the stiffness degradation occurs at blast 5. The effects of accumulated damage can also be seen through the displacement time-histories presented in Figure 8-23(b) through (f), where greater inaccuracies are observed with increasing number of blasts.

Table 8-19 Summary of Dynamic Analysis (Series 3)

Beams	Shockwave Properties			Maximum Mid-Span Displacement			
ID	Blast	P _r (kPa)	I _r (kPa-ms)	δ_{exp} (mm)	δ_{anls} (mm)	$\delta_{anls} / \delta_{exp}$	% Error
CRC-2%A-#4-HSR-S	1	22.29	226.5	5.05	7.33	1.45	45.15%
	2	36.51	413.42	14.01	13.18	0.94	-5.92%
	3	51.37	537.1	21.9	22.55	1.03	2.97%
	4	65.37	695.37	162.7	34.71	0.21	-78.67%
CRC-2%A-#5-HSR-S	1	20.03	215.19	6.67	6.3	0.94	-5.55%
	2	30.08	330.56	11.78	10.97	0.93	-6.88%
	3	47.56	532.86	19.15	17.7	0.92	-7.57%
	4	60.42	698.23	28.12	25.51	0.91	-9.28%
	5	73.28	863.95	40.01	34.26	0.86	-14.37%
CRC-2%A-#6-HSR-S	1	22.29	246.28	6.04	5.62	0.93	-6.95%
	2	35.65	361.48	11.04	9.58	0.87	-13.22%
	3	N/A	N/A	17	14.88	0.88	-12.47%
	4	62.2	720.49	25.01	20.56	0.82	-17.79%
	5	81.94	906	33.78	26.62	0.79	-21.20%
	6	94.2	1085.15	49.82	36.78	0.74	-26.17%
	7	97.66	1078.44	98.42	40.85*	0.42	-58.49%

*Maximum resistance curve displacement exceeded (blow out failure).

Table 8-20 Analysis Statistics (Series 3)

Blast #	Mean ratio ($\delta_{anls} / \delta_{exp}$)	standard deviation	Coefficient of Variance	% Error
1 to 7	0.85	0.26	0.31	-14.78%
1	1.11	0.22	0.20	8.16%
2	0.91	0.22	0.24	-6.51%
3	0.94	0.22	0.24	-4.27%
4	0.65	0.15	0.23	-26.44%
5	0.82	0.05	0.06	-17.78%
6	0.74	n/a	n/a	-26.17%
7	0.42	n/a	n/a	-58.49%

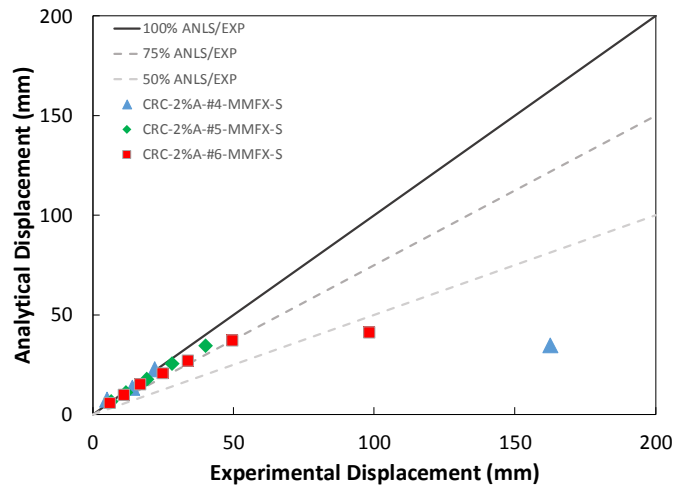


Figure 8-22 Experimental vs. Analytical Displacements (Series 3)

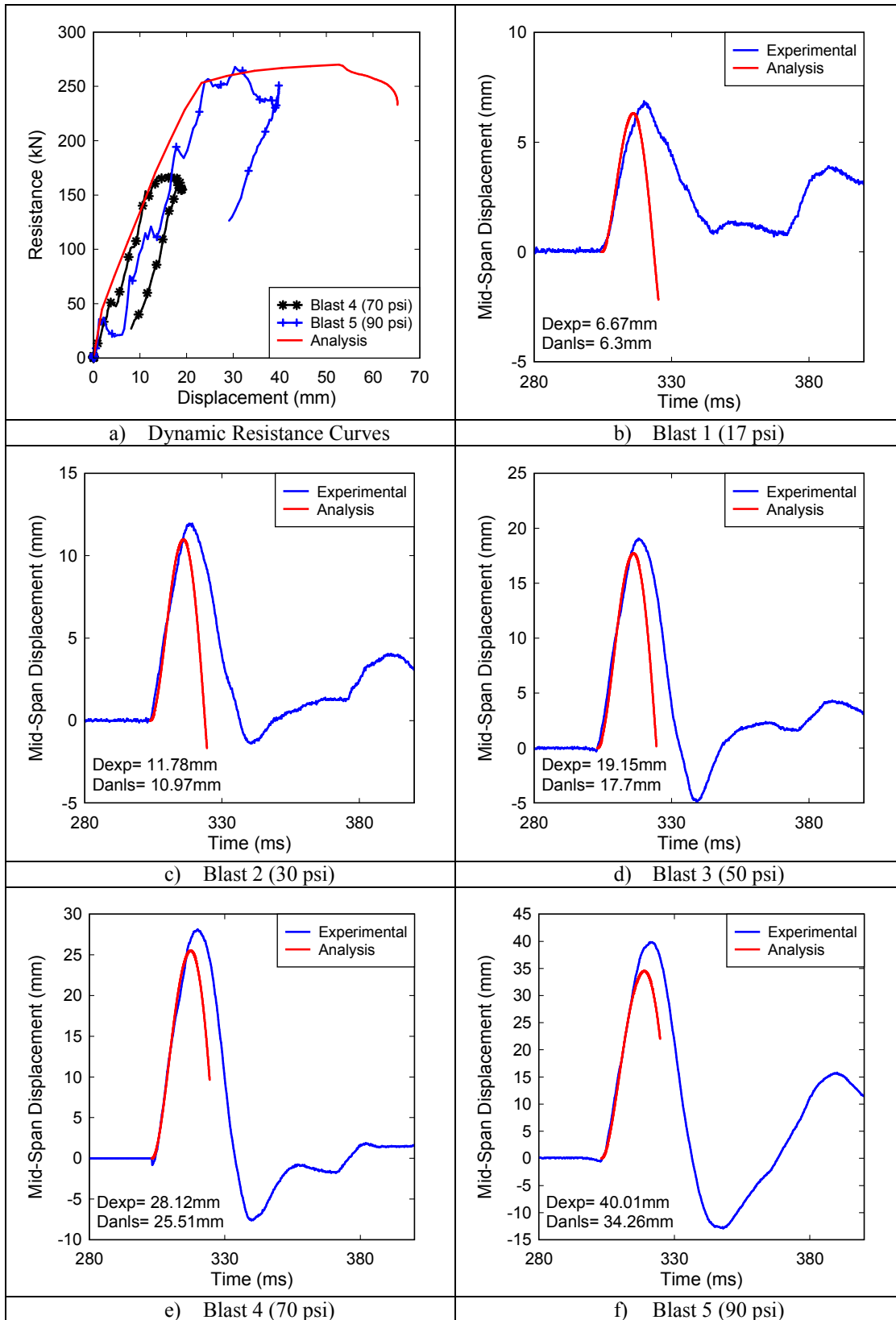


Figure 8-23 Sample Resistance Curves and Displacement Time-Histories (CRC-2%A-#5-S)

8.7. Sensitivity Study – Dynamic Analysis

This section presents the results of sensitivity analysis conducted for two key SDOF analysis parameters; dynamic increase factors and plastic hinge length. Only one parameter such as dynamic increase factor or plastic hinge length was varied per case, leaving all other parameters constant. As mentioned previously the results from the sensitivity analysis were used to define the default analysis inputs that were presented in the previous section.

8.7.1. Effect of Dynamic Increase Factors

Dynamic increase factors are utilized to effectively model the response of reinforced concrete structural elements under high strain rates. This section examines various combinations of DIF on the analytical predictions of beam response under blast loads.

Control Series

The sensitivity analysis conducted for the control series specimens (normal strength concrete and reinforcement) consisted of three distinct cases (see Table 8-21). Case 1, implements the CEB model for concrete in compression, the Malvar & Ross for concrete in tension and lastly the Saatcioglu et al. model for steel reinforcement. Case 2, is more conservative as it includes dynamic increase factors specified for design through UFC-3-340-02 (2008). Lastly, case 3 provides no increase in material strength with all dynamic increase factors set at 1.0.

Table 8-21 Dynamic Increase Factors for NSC Specimens (Series 1)

	Dynamic Increase Factors		
	Concrete in Compression (Hognestad)	Concrete in Tension (Linear)	Steel Reinforcement (Park & Paulay)
Case 1	CEB (DIF _c =1.34)	Malvar & Ross (DIF _{ct} =1.52)	Saatcioglu et al. (DIF _v =1.3 ; DIF _u =1.1)
Case 2	UFC (DIF _c =1.19)	Custom (DIF _{ct} =1.1)	UFC (DIF _v =1.17 ; DIF _u =1.05)
Case 3	(DIF _c =1.0)	(DIF _{ct} =1.0)	(DIF _v =1.0 ; DIF _u =1.0)

Figure 8-24 presents sample analytical-to-experimental mid-span displacement ratios for control specimen NSC-0%-15M-S. The results demonstrates the importance of selecting proper dynamic increase factors in blast analysis, as the results have the potential to vary significantly. Looking at case 1, the results are very precise for blast 1 and blast 2, however given the stiffness degradation and softening of the specimen due to repeated blasts, the mid-span displacement is underestimated by approximately 33% at blast 3. Although the overall results for case 2 and case 3 have analytical-to-experimental displacement ratios of 0.86 and 0.99, respectively, case 1 is considered to be the most reliable at predicting single blast dynamic response of specimens as case 2 and case 3 significantly overestimate the mid-span displacements of blast 1 and blast 2 and only become more

accurate when stiffness degradation occurs as presented in Table 8-22. The results of the analysis also follow the expected trend, with the displacement ratios increasing as the DIF factors reduce.

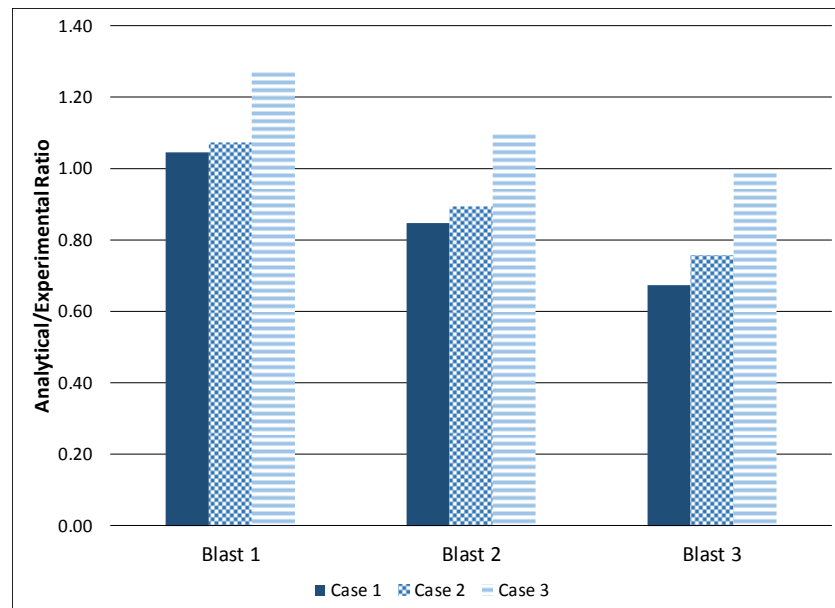


Figure 8-24 NSC-0%-15M-S Sample Sensitivity Analysis Result

Table 8-22 Sensitivity Analysis Results: Dynamic Increase Factors (Series 1)

Specimen I.D	Blast #	$\frac{\sigma_{anls}}{\sigma_{exp}}$		
		Case 1	Case 2	Case 3
NSC-0%-#4-0	1	0.97	1.01	1.17
	2	0.77	0.86	1.05
	3	0.65	0.72	0.83
NSC-0%-15M-S	1	1.05	1.07	1.27
	2	0.85	0.89	1.10
	3	0.67	0.76	0.99
NSC-0%-20M-S	1	0.85	0.87	0.97
	2	0.82	0.84	0.95
	3	0.73	0.80	0.99
	4	0.31	0.38	0.52
Average:		0.82	0.86	0.99

UHPC & NSR Series

In addition to the three DIF combinations used for the control series in the previous section, a fourth case (labelled as Case 1) was also considered as shown in Table 8-23. Case 1 consists of using the Zhang & Mindess model for concrete in compression, the Malvar & Ross model for concrete in tension and lastly the Saatcioglu et al. (2011) for steel reinforcement. Case 2 substitutes the compression DIF model with the CEB formulation. Case 3 uses the design UFC criteria for DIF, whereas Case 4 conservatively uses DIF =1 for all materials. Once again, the DIF cases range in intensity, with case 1 significantly increasing the material strength when compared to case 4 which considered no dynamic strength increase.

Table 8-23 Dynamic Increase Factors for UHPC and NSR Specimens (Series 2)

	Dynamic Increase Factors		
	Concrete in Compression (Hosinieh)	Concrete in Tension (Lok & Pei)	Steel Reinforcement (Park & Paulay)
Case 1	Zhang & Mindness (DIF _c =1.20)	Malvar & Ross (DIF _{ct} =1.13)	Saatcioglu et al. (DIF _v =1.3 ; DIF _u =1.1)
Case 2	CEB (DIF _c =1.085)	Malvar & Ross (DIF _{ct} =1.13)	Saatcioglu et al. (2011). (DIF _v =1.3 ; DIF _u =1.1)
Case 3	UFC (DIF _c =1.19)	Custom (DIF _{ct} =1.1)	UFC (DIF _v =1.17 ; DIF _u =1.05)
Case 4	(DIF _c =1.0)	(DIF _{ct} =1.0)	(DIF _v =1.0 ; DIF _u =1.0)

Figure 8-25 presents sample mid-span displacement ratio results for specimen CRC-2%A-20M-0. Results for all beams are summarized in Table 8-24. The results show that cases 1, 2 and 3 generate reasonable results having average analytical-to-experimental mid-span displacement ratios of; 0.94, 0.96 and 0.97, respectively. Cases 1 through 3 provide similar results. Nonetheless, case 1 was selected as the optimal dynamic increase factor combination for the analysis of beam specimens constructed with UHPC and normal strength reinforcement as it uses a compression DIF model which accounts for high strength concrete and fibers. It is interesting to note that the “design” Case 3 (UFC) does not result in more conservative results when compared to Cases 1 and 2 as the UFC specified factors for concrete do not account for concrete strength. As expected, the lack of DIF in case 4 results in relatively more conservative displacement predictions (larger $\frac{\delta_{anls}}{\delta_{exp}}$ ratios) when compared to all other cases. It is also interesting that the ratios are lower than 1 even for this case at Blast 3 and 4, a further indication of the need to consider accumulated damage.

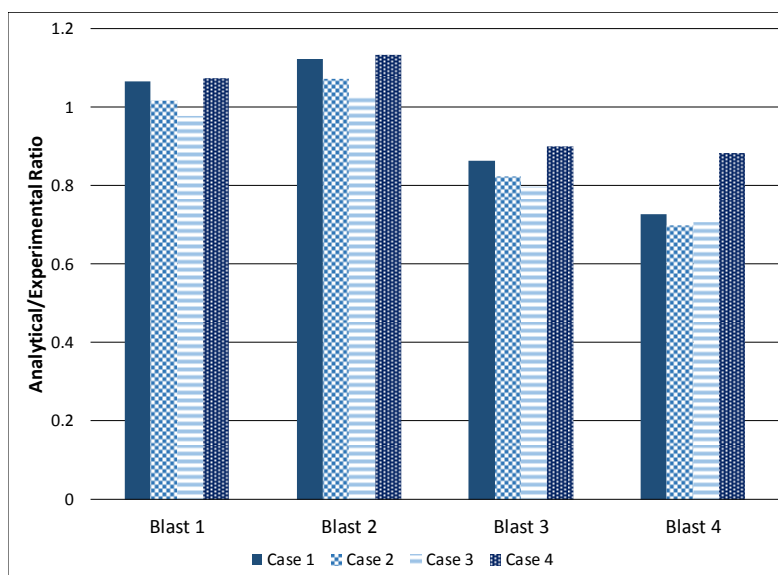


Figure 8-25 CRC-2%A-20M-0 Sample Sensitivity Analysis Result

Table 8-24 Sensitivity Analysis Results: Dynamic Increase Factors (Series 2)

Specimen I.D	Blast #	$\bar{\mu}_{anls} / \bar{\mu}_{exp}$			
		Case 1	Case 2	Case 3	Case 4
CRC-2%A-#4-0	1	1.52	1.52	1.66	1.81
	2	1.07	1.07	1.17	1.28
	3	0.81	0.83	0.92	1.21
	4	0.37	0.38	0.42	0.50
CRC-2%A-20M-0	1	1.06	1.02	0.98	1.08
	2	1.12	1.07	1.03	1.13
	3	0.86	0.82	0.80	0.90
	4	0.73	0.70	0.71	0.88
	5	0.60	0.59	0.63	1.23
CRC-2%A-15M-S	1	0.80	0.95	0.87	0.95
	2	0.98	1.17	1.08	1.19
	3	0.90	1.04	1.01	1.29
	4	0.85	0.93	1.00	1.24
CRC-2%A-20M-S	1	1.81	1.73	1.66	1.83
	2	1.19	1.13	1.09	1.20
	3	1.10	1.05	1.02	1.15
	4	0.96	0.92	0.93	1.16
	5	0.71	0.69	0.73	1.44
CRC-1%B-15M-0	1	0.99	1.00	0.95	1.15
	2	0.90	0.90	0.89	1.15
	3	1.18	1.19	1.28	1.65
	4	1.12	1.39	1.31	1.77
	5	0.49	0.72	0.50	0.58
CRC-2%B-15M-0	1	1.00	0.95	0.87	1.22
	2	0.75	0.71	0.66	0.96
	3	0.80	0.77	0.80	1.16
	4	0.67	0.66	0.71	1.00
	5	1.00	1.00	1.00	1.00
CRC-3%B-15M-0	1	1.25	1.20	1.38	1.50
	2	0.92	0.88	1.01	1.11
	3	0.95	0.92	1.08	1.38
	4	0.76	0.76	0.90	1.12
Average:		0.94	0.96	0.97	1.19

UHPC & HSR Series

The DIF model combinations for the four cases considered in series 3 are identical to those of series 2, with the exception of the DIF for the high-strength steel reinforcement. Since the high strength steel (MMFX) does not have a well-defined yield point, a constant dynamic increase factor is assigned to the entire stress-strain relationship. The various dynamic increase factor model combinations are presented in Table 8-25.

Table 8-25 Dynamic Increase Factors for UHPC and HSR Specimens (Series 3)

	Dynamic Increase Factors		
	Concrete in Compression (Hosinieh)	Concrete in Tension (Lok & Pei)	Steel Reinforcement (MMFX ₂ Coupon)
Case 1	Zhang & Mindness (DIF _c =1.20)	Malvar & Ross (DIF _{ct} =1.13)	DIF=Constant=1.1
Case 2	CEB (DIF _c =1.085)	Malvar & Ross (DIF _{ct} =1.13)	DIF=Constant=1.1
Case 3	UFC (DIF _c =1.19)	Custom (DIF _{ct} =1.1)	DIF=Constant=1.05
Case 4	(DIF _c =1.0)	(DIF _{ct} =1.0)	DIF=Constant=1.0

The results presented in Figure 8-26 and Table 8-26 differ from those obtained for series 1 and series 2 as the most conservative DIF model combination (case 4) provides significantly more accurate analytical predictions. The overall analytical-to-experimental mid-span displacement ratio for case 4 is 0.86, which is considerably more accurate than that of case 1 which results in a ratio of 0.76. This underlines the need for more research to study the blast behaviour of ultra-high performance concrete beams detailed with high strength reinforcement, with the development of appropriate material models which account for high strain rates, as the results show that existing concrete and steel reinforcement DIF models do not accurately encapsulate the response of beams built with such high strength materials. Thus, case 4 which considered no dynamic increase factor models, was selected as the most reliable analysis model combination for the current study.

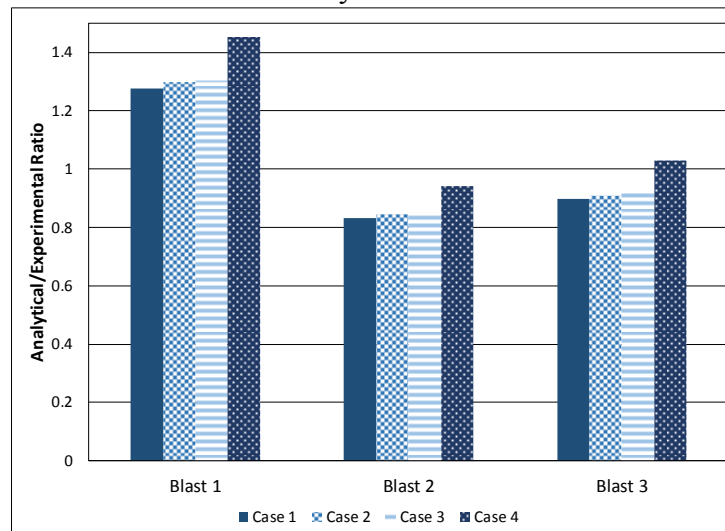


Figure 8-26 CRC-2%A-#4-HSR Sample Sensitivity Analysis Results

Table 8-26 Sensitivity Analysis Results: Dynamic Increase Factors (Series 3)

Specimen I.D	Blast #	$\bar{u}_{anis} / \bar{u}_{exp}$			
		Case 1	Case 2	Case 3	Case 4
CRC-2%A-#4-HSR-S	1	1.28	1.30	1.30	1.45
	2	0.83	0.85	0.85	0.94
	3	0.90	0.91	0.92	1.03
	4	0.18	0.19	0.19	0.21
CRC-2%A-#5-HSR-S	1	0.83	0.85	0.85	0.94
	2	0.83	0.84	0.84	0.93
	3	0.81	0.83	0.83	0.92
	4	0.79	0.80	0.81	0.91
	5	0.74	0.75	0.77	0.86
	6	1.00	1.00	1.00	1.00
CRC-2%A-#6-HSR-S	1	0.81	0.84	0.83	0.93
	2	0.76	0.79	0.78	0.87
	3	0.77	0.79	0.79	0.88
	4	0.72	0.73	0.74	0.82
	5	0.68	0.70	0.71	0.79
	6	0.62	0.64	0.65	0.74
	7	0.35	0.36	0.37	0.42
Average:		0.76	0.77	0.78	0.86

8.7.2. Effect of Plastic Hinge Length

This final section of this chapter considers the effect of plastic hinge length on the dynamic analysis results. This study is a continuation of the plastic hinge length sensitivity analysis conducted in the static series: section 8.5.3. The objective was to validate that the optimal plastic hinge length models obtained for static analysis correspond to those in the dynamic analysis.

The different plastic hinge cases for series 1, are summarized in Table 8-27, while the cases for series 2 and 3 are presented in Table 8-28. Bar charts illustrate the effect of plastic hinge model on the analytical-to-experimental displacement ratios for sample beams in series 1, 2 and 3 are in Figure 8-27. Similarly, tabulated results for series 1, 2 and 3 are presented in Table 8-29, Table 8-30 and Table 8-31, respectively.

Table 8-27 Dynamic Sensitivity Analysis Cases: Plastic Hinge Length (Series 1)

	Concrete Compression Model	Concrete Tension Model	Steel RModel	Plastic Hinge Length
Case 1	Hognestad (1951)	Linear	Park & Paulay (1975)	D
Case 2				Mattock (0.7D)
Case 3				Sawyer (0.5D)
Case 4				0.3D

Table 8-28 Dynamic Sensitivity Analysis Cases: Plastic Hinge Length (Series 2&3)

	Concrete Compression Model	Concrete Tension Model	Steel Model (NSR/MMFX)	Plastic Hinge Length
Case 1	Hosinieh	Lok & Pei	Park & Paulay/ ITG-6	D
Case 2				Mattock (0.7D)
Case 3				Sawyer (0.5D)
Case 4				0.3D

Results show that the choice of plastic hinge length does not have a major impact on the results of the single degree of freedom dynamic analyses, which was used to predict response up to maximum displacement. As depicted in the sample bar charts and tabulated results for series 1, 2 and 3, the mid-span displacement ratios are practically identical until a specimen experiences its final failure blast (although the general trend at later blasts shows an increase in $\bar{u}_{anls} / \bar{u}_{exp}$ ratios as hinge length reduces). Thus, it can be stated that plastic hinge length plays an important role in the predictions of static response, however, the effects of plastic hinge length on single degree of freedom dynamic analysis is minor.

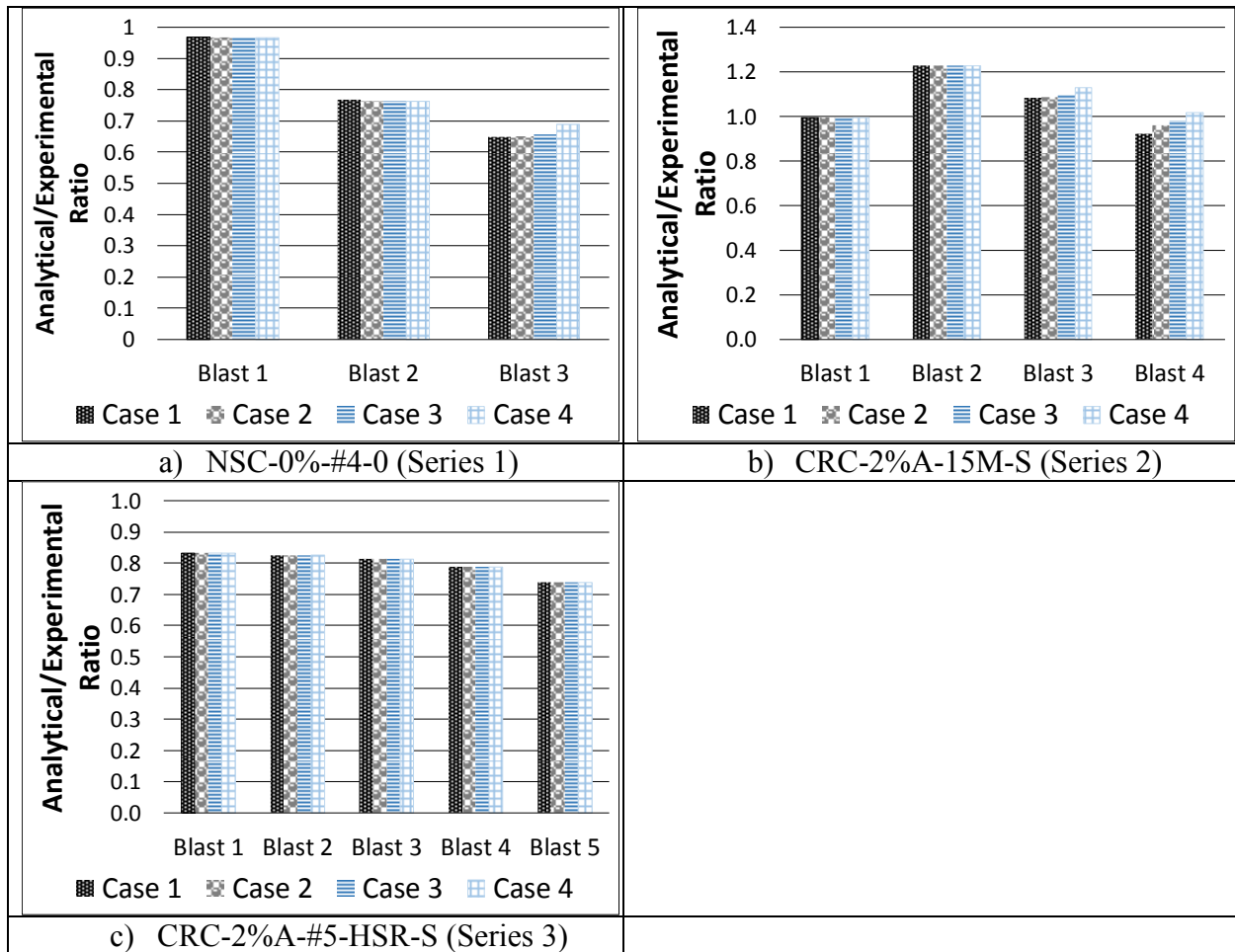


Figure 8-27 Sample Sensitivity Analysis Results: Effect of Hinge Length

Table 8-29 Sensitivity Analysis Results: Plastic Hinge Length (Series 1)

Specimen I.D	Blast #	$\bar{L}_{anls} / \bar{L}_{exp}$			
		Case 1	Case 2	Case 3	Case 4
NSC-0%-#4-0	1	0.97	0.97	0.97	0.97
	2	0.77	0.76	0.76	0.76
	3	0.65	0.65	0.66	0.69
NSC-0%-20M-S	1	0.85	0.85	0.85	0.85
	2	0.82	0.82	0.82	0.82
	3	0.73	0.73	0.73	0.74
	4	0.31	0.34	0.39	0.65
Average:		0.73	0.73	0.74	0.78

Table 8-30 Sensitivity Analysis Results: Plastic Hinge Length (Series 2)

Specimen I.D	Blast #	$\bar{L}_{anls} / \bar{L}_{exp}$			
		Case 1	Case 2	Case 3	Case 4
CRC-2%A-15M-S	1	0.99	0.99	0.99	0.99
	2	1.23	1.23	1.23	1.23
	3	1.08	1.09	1.10	1.13
	4	0.92	0.96	0.98	1.02
CRC-2%A-20M-S	1	1.81	1.81	1.81	1.81
	2	1.19	1.19	1.19	1.19
	3	1.10	1.10	1.10	1.10
	4	0.96	0.96	0.96	0.96
	5	0.68	0.69	0.70	0.71
Average:		1.11	1.11	1.12	1.13

Table 8-31 Sensitivity Analysis Results: Plastic Hinge Length (Series 3)

Specimen I.D	Blast #	$\bar{L}_{anls} / \bar{L}_{exp}$			
		Case 1	Case 2	Case 3	Case 4
CRC-2%A-#5-HSR-S	1	0.83	0.83	0.83	0.83
	2	0.83	0.83	0.83	0.83
	3	0.81	0.81	0.81	0.81
	4	0.79	0.79	0.79	0.79
	5	0.74	0.74	0.74	0.74
CRC-2%A-#6-HSR-S	1	0.81	0.81	0.81	0.81
	2	0.76	0.76	0.76	0.76
	3	0.77	0.77	0.77	0.77
	4	0.72	0.72	0.72	0.72
	5	0.68	0.68	0.68	0.68
	6	0.62	0.62	0.62	0.62
	7	0.35	0.35	0.35	0.35
Average:		0.72	0.73	0.73	0.73

8.8. Analytical Sources of Error

The analysis in this research program was conducted using a self-developed analysis program called *UO Resistance*, the results proved to be reasonably accurate when compared to experimental data. However, it is always good practice to assess and identify potential sources of error and future improvements.

Firstly, some of the differences in analytical to experimental results can be attributed to the collection of blast data using the data acquisition equipment, as peak reflected pressure reading was taken on the side of the shock-tube load transfer device, it is not conclusive that there is a uniform peak reflected pressure across the entire load transfer device area (indeed, the readings from the side and middle pressure gages were observed to be slightly different). As a result, the peak reflected pressure input into *UO Resistance* may have had some uncertainties. Moreover, it was assumed that the load transfer device is fastened by means of frictionless hinges at the top and bottom supports, however in reality there may have been some rotational resistance provided by the said load transfer device hinges.

When examining the static results, the constitutive material models implemented for concrete in compression, concrete in tension and steel reinforcement provide reasonably accurate predictions. However, as depicted in Figure 8-13 c) and d), the post peak response of the UHPC specimens is not as well estimated, hence further research should be conducted in order to refine the concrete compression and tension models such that the prediction of post-peak response is improved.

The dynamic analysis results obtained through equivalent single degree of freedom analysis agree reasonably well with the experimental results, nonetheless some modelling assumptions were made in order to facilitate the rather complex system for implementation in the SDOF model. As stated by (Biggs, 1964):

“From the viewpoint of practical design, the approximate methods presented here are extremely important. They should not be regarded as merely crude approximations, to be used for rough preliminary analysis, nor should they be regarded as methods to be used by engineers who lack the training or intellect to employ more sophisticated techniques... Complex methods of analysis are often not justified. It is a waste of time to employ methods having precision much greater than that of the input of the analysis.”

Another important assumption that was made throughout the equivalent single degree of freedom analysis was that the beam element and shock-tube load transfer device behaved as a uniform mass, with concentrate loads. However, the argument could be made that the mass of the system was a combination of both a uniform beam mass and a concentrated load transfer device mass which would require the combination of two transformation (K_{LM}) factors. In reality a multi-degree of freedom analysis should be conducted in order to get closer to the exact solution.

Finally, as noted previously another important source error is the accumulated damage from repeated loading which was not considered in the analysis.

Chapter 9. Conclusion

9.1. Conclusion

This thesis presented both an experimental and analytical research program investigating the static and blast performance of beams built with ultra high performance concrete (UHPC) in combination with normal-strength and high-strength ASTM A1035 (MMFX) longitudinal steel reinforcing bars. The principle parameters examined in this study included; concrete strength, steel reinforcement type, longitudinal steel reinforcement ratio, steel fibre type, volumetric fibre content and the effect of transverse steel reinforcement.

A total of 21 beam specimens were included in this study, fourteen of which were tested under blast loading using the University of Ottawa shock tube, with the remaining seven specimens tested under quasi-static four-point bending. The main criteria used to judge the performance of the beams subjected to blast loading were the overall blast resistance, maximum and residual displacements, failure mechanisms, crack patterns, debris generation and secondary fragmentation. The quasi-static specimen performance was evaluated based on maximum load resisted, mid-span displacement at failure, toughness, ductility and failure mechanism. The main findings from the experimental research program are summarized below:

- Increasing the longitudinal steel reinforcement ratio in the normal strength concrete control series beams increased the overall blast resistance of the specimens and delayed failure to a greater blast load. The increase in reinforcement ratio also effectively reduced the mid-span displacements under blast loading. The reinforcement ratio also impacted the failure modes experienced by the control specimens. The increase of reinforcement ratio significantly increased the load carrying capacity of the statically loaded specimens, at the expense of ductility.
- Similarly, increasing the reinforcement ratio in the specimens of series 2 (UHPC & NSR) was effective in reducing displacements at equivalent blasts, and increasing the overall blast load resistance of the UHPC beams, delaying failure to a greater blast. Moreover, increasing the reinforcement ratio altered the failure mode from rebar rupture to fiber pullout.
- As in series 1 and 2, the use of larger reinforcement ratio in series 3 (UHPC & HSR), improved blast performance by reducing displacements and increasing blast capacity. Moreover, the failure mode was altered from longitudinal rebar rupture to a combination of concrete crushing and fiber pullout when using the #6 HSR in the UHPC beams. In terms of static response, increasing the reinforcement ratio resulted in greater ultimate load resistance and prevented rebar rupture, although shear demands became greater.
- The effect of concrete type (UHPC vs. NSC) played an important role on the blast performance of specimens detailed with normal strength longitudinal reinforcement. The use of UHPC reduced displacements at equivalent blasts while also delaying the specimen failure to a greater blast load, in addition to greatly reducing the generation of debris or secondary fragmentation. However, the increased strength of UHPC altered the failure mechanism from concrete crushing in the series 1 (NSC & NSR) specimens to tensile rebar

rupture in several of the series 2 (UHPC & NSR) beams. Beam specimens subjected to static loading experienced similar results, series 2 (UHPC & NSR) specimens experienced greater maximum loads but experienced tensile rebar rupture failure mechanisms.

- The results showed that fiber type and content had a minor effect on the blast performance of the UHPC beams in series 2. In general fiber aspect ratio had a greater influence when comparing to fiber strength or content, with results show that fiber type A (aspect ratio: 62) outperforming fiber type B (aspect ratio: 43) both in terms of reduced damage (cracking, debris) and reductions in maximum and residual mid-span displacements.
- The results indicate important benefits associated with the combined use of UHPC and high-strength MMFX reinforcement in beams. Under blast loading, the use of high-strength steel improved control of maximum and residual displacements and increased overall blast capacity. However, use of high-strength bars had a tendency to present more brittle failures at extreme blast pressures. The use of large amounts of high-strength reinforcement (i.e. #6 bars in this study) allowed for the full capacity of the UHPC concrete to be utilized and prevented rebar rupture, therefore taking full advantage of the high strength materials in compression and in tension. Under static loading the combined use of UHPC and HSR increased ultimate capacity of the beams but tended to reduce ductility due to the reduced strain capacity of the high-strength bars.

The analytical component of this research program was completed using *UO Resistance*, a VBA program developed during this study. A sectional analysis of all beam specimens was conducted, from which a static resistance curve was derived for the response prediction of specimens loaded statically. Furthermore, using dynamic material models, dynamic resistance curves were developed, and dynamic analysis using a SDOF approach was used to predict the blast performance of all specimens tested using the shock-tube. The effects of several modeling parameters were also assessed in *UO Resistance* using a detailed sensitivity analysis. The principle findings from the analytical study are summarized below:

- The sectional analysis results used to obtain the resistance curve for all specimens tested under quasi-static conditions yielded accurate predictions in terms of ultimate load and displacement and specimen stiffness.
- The dynamic inelastic analysis and SDOF method was able to predict the blast response (maximum mid-span displacement) of specimens tested using the shock-tube with reasonable accuracy, especially at earlier blast load levels.
- Plastic hinge length played an important role in accurate prediction of the post-yield maximum displacement in the resistance curves. For normal strength concrete specimens, the most accurate results were obtained by using a plastic hinge length corresponding to the specimen's effective depth (D), with the relations provided by Sawyer and Mattock slightly under-estimated the maximum displacement. Specimens designed with UHPC yielded optimal prediction results when using a plastic hinge length equivalent to 30% of the specimen's effective depth ($0.3D$). Using the total effective depth (D), or the Mattock/

Sawyer relations tended to over estimate the maximum displacement. The effect of plastic hinge length was less important when considering the dynamic analysis results.

- Dynamic increase factors must be considered in order to achieve accurate dynamic predictions. Series 1 (NSC & NSR) considered three combinations of material dynamic increase factors, with the results showing that DIF combination 1 (CEB for concrete in compression, Malvar and Ross for concrete in tension and Saatcioglu et al. for steel reinforcement in tension) yielded the most accurate blast prediction. Combinations 2 and 3 underestimated the dynamic capacity of the beam specimens.
- The most accurate DIF models for series 2 specimens (UHPC & NSR) are Zhang and Mindness for concrete in compression, Malvar and Ross for concrete in tension and Saatcioglu et al. for steel reinforcement in tension. Combinations 2, 3 and 4 underestimated the dynamic capacity of the beam specimens.
- Contrary to series 1 and 2, for series 3 (UHPC & HSR) results are most accurate when no dynamic increase factors are considered. The use of dynamic increase factors tended to over-estimate the dynamic resistance of beams made of UHPC and high strength reinforcement.
- In general, the dynamic analysis predictions were more accurate at earlier blast levels when compared to later blasts. This may have been due to effect of accumulated damage which was not accounted for in the beams which were tested under repeated blast pressures.

9.2. Recommendations

The following recommendations for future research are suggested:

- Studying the performance of reinforced concrete beams under single versus multiple (repeated) blast loads. The use of single shot failure blasts to virgin beam specimens would allow for a better understanding of the actual dynamic response under blast loads due to the effects of stiffness degradation in beams tested under multiple blasts.
- Including a larger set of test specimens (with replicates) with varied steel fibre type and volumetric content in order to establish more well-defined trends.
- Assessing the close-range blast response of UHPC beam specimens, as the shock-tube setup used in the current study tested the beams under far-range shockwaves.
- Analytical sensitivity analysis on the optimal hysteretic model to predict the response of beams subjected to repeated blast loading.
- Further research to develop reliable dynamic increase factors for high strength materials such as UHPC and high-strength reinforcement.

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