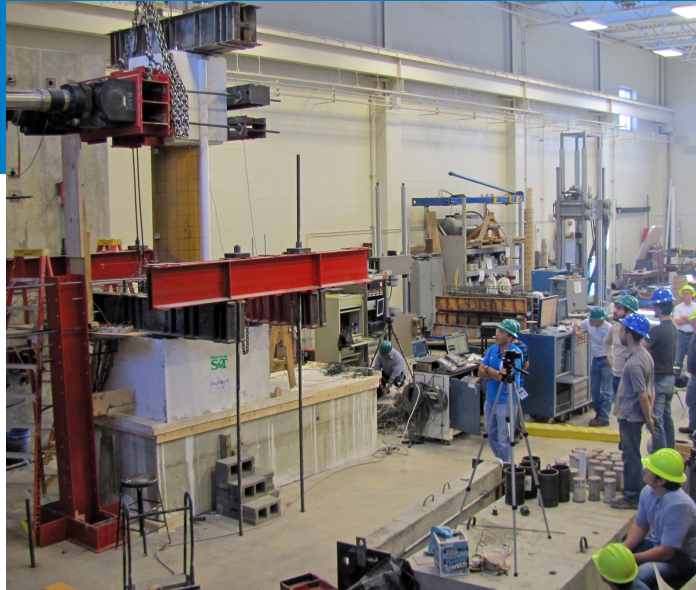


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Structural Performance of Concrete
Columns Incorporating Advanced
Materials and Structural Systems

SP-341

Editor:
Mohamed A. ElGawady



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Structural Performance of Concrete Columns Incorporating Advanced Materials and Structural Systems

Sponsored by
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PREFACE

Columns are crucial structural elements in buildings and bridges. This Special Publication of the American Concrete Institute Committees 441 (Reinforced Concrete Columns) and 341A (Earthquake-Resistant Concrete Bridge Columns) presents the state-of-the-art on the structural performance of innovative bridge columns. The performance of columns incorporating high-performance materials such as ultra-high-performance concrete (UHPC), engineered cementitious composite (ECC), high-strength concrete, high-strength steel, and shape memory alloys is presented in this document. These materials are used in combination with conventional or advanced construction systems, such as using grouted rebar couplers, multi-hinge, and cross spirals. Such a combination improves the resiliency of reinforced concrete columns against natural and man-made disasters such as earthquakes and blast.

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Behavior of High-Strength Concrete and Normal-Strength Concrete Columns under Blast Loading

Amer Hammoud and Hassan Aoude

Synopsis: This paper presents the results from tests examining the performance of high-strength concrete (HSC) and normal-strength concrete (NSC) columns subjected to blast loading. As part of the study six columns built with varying concrete strengths were tested under simulated blast loads using a shock-tube. In addition to the effect of concrete strength, the effects of longitudinal steel ratio and transverse steel detailing were also investigated. The experimental results demonstrate that the HSC and NSC columns showed similar blast performance in terms of overall displacement response, blast capacity, damage and failure mode. However, when considering the results at equivalent blasts, doubling the concrete strength from 40 MPa to 80 MPa (6 to 12 ksi) resulted in 10%-20% reductions in maximum displacements. On the other hand, increasing the longitudinal steel ratio from $\rho = 1.7\%$ to 3.4% was found to increase blast capacity, while also reducing maximum displacements by 40-50%. The results also show that decreasing the tie spacing (from $d/2$ to $d/4$, where d is the section depth) improved blast performance by reducing peak displacements by 20-40% at equivalent blasts. The use of seismic ties also prevented bar buckling and reduced the extent of damage at failure. As part of the analytical study the response of the HSC columns was predicted using single-degree-of-freedom (SDOF) analysis. The resistance functions were developed using dynamic material properties, sectional analysis and a lumped inelasticity approach. The SDOF procedure was able to predict the blast response of HSC columns with reasonable accuracy, with an average error of 14%. A numerical parametric study examining the effects of concrete strength, steel ratio and tie spacing in larger-scale columns with 350 mm x 350 mm (14 in. x 14 in.) section was also conducted. The results of the numerical study confirm the conclusions from the experiments but indicate the need for further blast research on the effect of transverse steel detailing in larger-scale HSC columns.

Keywords: High-strength concrete, Columns, Blast, Shock Tube.

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INTRODUCTION

The use of high-strength concrete (HSC) in structures has gradually increased over the past few decades. The American Concrete Institute (ACI) defines HSC as concrete that has a compressive strength which exceeds 55 MPa (8 ksi), however concretes with strengths of 80-100 MPa (12-15 ksi) are now regularly used in practice ¹. One of the established applications of HSC is in the lower storey columns of high-rise buildings, where its use can allow for reduced cross-section sizes and more efficient use of floor space ². As noted in ACI 363R-10: "*the largest application of HSC has been in the columns of high-rise buildings*" and "*all of the tallest buildings constructed in the past 10 years have some structural contribution from HSC in vertical column elements*" ¹.

Over the years, extensive studies have examined the behavior of HSC columns under pure axial compression and simulated earthquake loading. A comprehensive review of this research is summarized in ACI 441R-96: Report on High-strength Concrete Columns ². As a result of this important research, design requirements for the structural use of high-strength concrete in columns are now well established in several codes worldwide ³. Comparatively very limited research exists on the blast performance of such columns. Given the critical role of ground-story columns in the overall blast resistance and stability of multi-story buildings ⁴, there exist an important need for research data that can allow for better understanding of the blast behavior of HSC columns.

Accordingly, this paper examines the effect of increased concrete strength on the blast behavior of a series of columns tested under simulated blasts using a shock-tube. In addition to the effect of concrete strength, the effects of longitudinal steel ratio and seismic detailing are also investigated. As part of the analytical study the blast behavior of the test columns is predicted using dynamic inelastic single-degree of freedom (SDOF) analysis.

REVIEW OF PREVIOUS BLAST RESEARCH ON RC COLUMNS

Important research has investigated the behavior of reinforced concrete columns under extreme dynamic loading, including impact and blast. The following sections provide a brief review of this research.

Previous Impact Studies on Reinforced Concrete Columns

The impact behavior of reinforced columns has been investigated by a number of early researchers, including Reinschmidt et al. ⁵ and others. Louw et al. ⁶ conducted a more recent study on the impact response of reinforced concrete columns. A total of 28 cantilever columns having dimensions of 4 m × 300 mm × 300 mm (13 ft × 12 in. × 12 in.) were tested under drop-weight impact loads, with an additional 8 specimens tested under quasi-static conditions. Varying strain-rates were obtained by dropping a striker with varying masses (650-1450 kg [1430-3200 lb]) from 2.7 m (8.9 ft), resulting in impact velocities of 6.4-7.1 m/s (21-23 ft/s). The columns were subdivided into 8 groups based on the percentage of steel, concrete strength ($f'_c = 19\text{-}37$ MPa [2.8-5.4 ksi]) and amount of axial load. The main conclusion of this study was that concrete strength and shear reinforcement had a greater influence on the shear resistance of the columns, while flexural strength was governed by the longitudinal reinforcement ratio. For shear-critical columns, doubling the concrete strength was found to increase the impact strength by 33%, compared to an increase of 17% under static conditions.

Remennikov and Kaewunruen ⁷ conducted another series of impact tests on five quarter-scale reinforced concrete columns ($f'_c = 32$ MPa [4.6 ksi]) having dimensions of 1 m × 100 mm × 100 mm (40 in. × 4 in. × 4 in.) using a drop hammer test rig. The main parameters investigated were the drop-height (1.2-1.9 m [47-75 in.]) of the 160 kg (352 lb) mass, and the column shear-bending capacity ratios (0.63-0.93). An additional 2 columns were tested statically. Since the shear/bending capacity ratios were less than 1, all impact-tested columns failed in shear, with diagonal cracks developing in a more severe manner when compared to static conditions. While the study found the dynamic reaction force can be enhanced by a factor of up to 2.0 under impact, the authors noted that flexural behavior under impact can only be developed if the shear capacity significantly exceeds bending resistance due to amplification of the reaction force under dynamic loading. The study further noted that columns with poorly confined concrete require an enhancement to their ductility to adequately resist impact loading.

Research on the impact response of HSC columns is scarce, however Huynh et al. ⁸ investigated the impact response of a series of sixteen 2 m × 250 mm × 250 mm (80 in. × 9.8 in. × 9.8 in.) high-strength concrete and reactive powder concrete (RPC) beams-columns ($f'_c = 100\text{-}160$ MPa [15-23 ksi]) under low and medium-velocity impact loading. The effects of axial force, load eccentricity, and RPC as a replacement for HSC (in the cover/shell

region or the entire section) were investigated. The results showed that both axial load and load eccentricity had important effects on impact resistance and failure mode, while the use of fiber-reinforced RPC was found to improve the impact performance of the columns when compared to HSC. Under static testing the HSC columns failed in flexure, whereas they experienced flexure-shear or shear failures under impact, with the authors noting that the change in failure mode was more noticeable for the HSC columns when compared to the RPC columns.

Previous blast studies on reinforced concrete columns

Given the critical role of columns in the blast performance of buildings, a significant number of studies have focused on studying the blast behavior of reinforced concrete columns, both numerically and experimentally. Saatcioglu et al.⁹ numerically studied the effect of seismic design on the blast response of RC frames using dynamic inelastic analysis. The authors concluded that seismic detailing improved the overall blast performance and ductility of the buildings, while noting that such detailing can reduce the potential of progressive collapse. Similar conclusions were reported by Hayes et al.¹⁰, where strengthening of perimeter column elements using modern seismic detailing requirements was found to reduce blast damage and prevent progressive collapse. Similarly, Bao and Li¹¹ used finite element modeling to examine the effects of transverse reinforcement spacing on the blast response of columns and noted that the use of seismic detailing improved control of deflections and increased residual capacity. Similar results have been reported by Kyei and Braimah¹² and others.

Experimental research on the blast behavior of conventional reinforced concrete columns has primarily focused on bridge type elements subjected to near-field or close-in blasts. Fujikura and Bruneau¹³ conducted an important series of field tests on quarter-scale seismically-detailed bridge columns as well as non-ductile columns retrofitted with steel-jacketing. The NSC columns ($f'_c = 42$ MPa [6 ksi]) had 300 mm (12 in.) circular cross-sections and were tested under close-in blasts, without axial loads. Converse to the above cited numerical research, the study found that the seismically-detailed and retrofitted columns did not show ductile response, and suffered brittle direct shear failures under blast loading. In a further series of companion tests, improved performance was obtained in concrete-filled circular steel tube (CFST) bridge piers designed using a multi-hazard concept¹⁴.

Williamson et al.¹⁵ conducted another important series of tests on ten half-scale NSC ($f'_c = 28$ MPa [4 ksi]) bridge columns, also under close-in blast loads. A large set of parameters were investigated, including the effect of standoff distance, column depth ratio, transverse steel detailing and splice location. Increasing the transverse steel ratio was found to improve the blast behavior of the columns, with the best performance obtained when using “blast” and “seismic” detailing when compared to “typical” design detailing. On the other hand, the failure mode under intense close-in blasts was also observed to be direct shear at the base.

Experimental studies on the behavior of RC columns subjected to far-field blasts are limited. Lloyd¹⁶ studied the blast response of a series of normal-strength concrete columns ($f'_c = 46$ -58 MPa [6.7-8.4 ksi]) having dimensions of 2.4 m × 0.1 m × 0.15 m (95 in. × 4 in. × 6 in.) and reinforced with varying steel ratios under simulated blast loads using a shock-tube. The study reported that the use of increased longitudinal steel ratio improved the displacement response of the columns, however the use of seismic vs conventional detailing was found to be insignificant. Conversely, Burrell et al.¹⁷ reported that seismic detailing in self-consolidating concrete (SCC) columns ($f'_c = 50$ MPa [7.3 ksi]) with larger 0.15m × 0.15m (6 in. × 6 in.) cross-sections reduced displacements, prevented bar buckling and increased damage tolerance.

To the knowledge of the authors there exists no published studies on the blast behavior of conventional HSC columns, whether under close-in or far-field explosions. Nonetheless, a few studies in the literature have focused on more advanced UHPC (ultra-high performance concrete). For example, Astarlioglu and Krauthammer¹⁸ numerically studied the blast response of NSC and UHPC columns using SDOF analysis and noted that the use of UHPC improved control of displacements and significantly increased the impulse required to cause failure. Similar conclusions related to the benefits of using UHPC in columns have been reported in the experimental studies conducted by Xu et al.¹⁹ (near-field blasts) and Aoude et al.²⁰ (far-field, simulated blasts).

In addition to new structures, several researchers have also studied the potential of various retrofits to increase the blast resistance of existing RC columns. Among them, an important number of studies have focused on the effectiveness of using fiber-reinforced plastics (FRP) to improve the blast behavior of columns (for a review, see Crawford²¹). Other retrofit techniques considered in the literature include the use of steel-jackets¹³ and steel-reinforced polymer (SRP) wraps²².

RESEARCH SIGNIFICANCE

The previous sections provided an overview of published studies which have focused on the impact and blast behavior of reinforced concrete columns. While important tests and conclusions have been reported in the literature, the blast behavior of high-strength concrete columns has not been specifically studied by other researchers. Previous research is also conflicting about the effectiveness of seismic detailing to improve the blast

behavior of reinforced concrete columns. Experimental studies on the blast behavior of columns subjected to far-field blasts also remain relatively limited. Accordingly, this paper aims to fill this gap in research and increase the understanding of the effect of increased concrete strength on the blast behavior of columns, with a particular focus on columns subjected to far-field type blasts. In addition to examining the effect of concrete strength, the tests provide insights into the effects of seismic detailing and longitudinal steel ratio on the blast behavior of the HSC columns. To further examine the influence of increased concrete strength, the response of the HSC columns is compared to ultra-high strength UHPC columns previously tested by the authors.

EXPERIMENTAL INVESTIGATION

Description of specimens

A total of six columns were built and tested in this research program. The tests included two columns cast with normal-strength concrete (NSC) and four columns built with high-strength concrete (HSC). The HSC series included specimens with and without seismic detailing, while the effect of longitudinal steel ratio was investigated in both concrete series. **Table 1** and **Fig. 1** summarize the design details of the test specimens.

As shown in **Fig. 1**, all columns had a total height of 2.44 m (8 ft) and cross-sectional dimensions of 152 mm $\times$ 152 mm (6 in. $\times$ 6 in.). Longitudinal reinforcement consisted of 4 – 10M, 4 – No.4, or 4 – 15M bars (bar diameters = 9.5, 11.3, 16 mm [0.38, 0.44, 0.63 in.], with bar areas = 71, 100, 200 mm² [0.11, 0.16, 0.31 in²]), resulting in longitudinal steel ratios of 1.7%, 2.2% and 3.4%. Transverse reinforcement in all “non-seismic” columns consisted of 6.3 mm (0.25 in.) diameter ties made of non-deformed steel wire, spaced at $s = 75$ mm (3 in.), conforming to the requirements of *moderately ductile* columns in the CSA A23.3 Standard (ductility-related factor, $R_d = 2.5$)²³. This spacing was reduced to 38 mm (1.5 in.) in one “seismic” HSC column, following the requirements of *ductile* columns in the same standard ($R_d = 4$). In all cases, the tie spacing was kept constant over the column height.

Specimen nomenclature in **Table 1** reflects the three variables in the experimental program, namely: the concrete type (HSC vs. NSC), longitudinal bar size (10M, No.4 or 15M bars), and transverse steel detailing level (with “S” indicating use of seismic ties). For example, column NSC-10M was built with normal-strength concrete, 4-10M bars and non-seismic ties spaced at 75 mm (3 in.), while HSC-10M-S represents the companion column built with HSC and seismic ties spaced at 38 mm (1.5 in.)

Material properties

Table 2 lists the properties of the normal-strength and high-strength concrete mixes used in this study. The NSC mix had a specified strength of 35 MPa (5.1 ksi), with a maximum aggregate size of 19 mm (0.75 in.) and contained no admixtures. The high-strength concrete mix had a target strength of 80 MPa (11.6 ksi), and its constituents included Portland cement, slag, silica fume, sand, coarse aggregate (with a maximum size of 9.5 mm [3/8 in.]) and liquid admixtures (a water reducer and set retarder). In all cases the concrete was mixed using a 0.225 m³ (8 ft³) capacity forced-action pan mixer at the University of Ottawa. The average properties of the concrete in each column in terms of compressive strength were obtained by testing three 100 mm $\times$ 200 mm (4 in. $\times$ 8 in.) cylinders on the day of testing in accordance with the ASTM C39 standard (see **Table 1**). The properties of the steel reinforcement were obtained from tension tests on a minimum of 3 samples for each bar type in accordance with the ASTM 370 standard²⁴ (see **Table 3**). Based on these tests, the average yield strengths of the 10M, No.4, 15M bars and 6 mm wire were determined to be 474, 464, 449 and 542 MPa (69, 67, 65 and 79 ksi). **Fig. 2** shows typical stress-strain curves for the NSC, HSC and steel reinforcement.

University of Ottawa Shock Tube

The University of Ottawa Blast Research Laboratory is equipped with a shock-tube that can simulate the shockwaves generated by the hemispherical free air surface burst of high explosives²⁵. As shown in **Fig. 3a**, the shock tube consists of four main components, including: (1) a variable length driver section which generates the shockwave energy using compressed air, (2) a spool section which controls the firing of the shockwave using a differential pressure diaphragm, (3) and an expansion section which ends with (4) a 2 m $\times$ 2 m (80 in. $\times$ 80 in.) rigid end test frame. For non-planar elements such as columns, a load transfer device (LTD) is used to collect and transfer the pressure at the end-frame opening onto the specimens. Varying shockwave parameters (pressure, duration and impulse) can be obtained by adjusting the shock-tube driver length and driver pressure.

Test setup and procedure

Fig. 3b shows a typical column prior to testing. All columns were tested under combined axial and blast loads. For all tests the axial load at the start of the tests was 294 kN (66 kip), equivalent to $\sim 30\%$ of the pure axial load capacity of the control normal-strength concrete specimens. In this study the LTD consisted of a light gauge steel metal sheet connected to a series of steel beams which resulted in the transfer of the blast pressure as a uniformly

distributed load along the tension face of the columns. The clear span of the columns between supports was 1980 mm (6.5 ft), with the columns connected to the shock-tube frame using partially-fixed supports. Complete displacement-time histories were recorded using two linear variable differential transducers (LVDT) placed at mid-height and at 1/3rd distance along the clear span as shown in **Fig. 3b**. Pressure sensors near the load transfer device were used to record complete reflected pressure time histories for each test. A high speed camera also recorded the response of the columns response at a frame rate of 500 frames per second.

Each column was tested with gradually increasing blast pressures to test the specimens under elastic, yield and ultimate conditions. Sample shockwaves corresponding to Blasts 1 to 4 are shown in **Fig. 4a**. In this study, the driver length was kept constant at 2743 mm (9 ft) while the driver pressures varied between 17 psi (117 kPa), 35 psi (240 kPa), 80 psi (550 kPa) and 100 psi (690 kPa) for Blasts 1 to 4, respectively. The resulting average reflected pressure (P_r), positive phase duration (t_d), and reflected impulse (I_r) for these blasts are summarized in **Table 4**.

EXPERIMENTAL RESULTS

Summary of results

Table 5 summarizes the detailed experimental results for the columns in terms of measured shockwave parameters (P_r , t_d , and I_r) and column responses (maximum midspan displacements, residual midspan displacements and support rotations: d_{max} , d_{res} and θ_{max}). **Fig. 4b** shows typical blast and response time histories for one column. **Fig. 5-7** show photos of the columns illustrating damage, and **Fig. 8-11** compare the column displacement responses at selected blasts. It is noted that displacements d_{max} and d_{res} (and the results in the figures) represent “net” values (i.e. displacements were zeroed before each test). For reference, d_{max}^c and d_{res}^c represent the cumulative displacements, which include the permanent deflections from previous shots (see **Table 5**). **Table 5** also compares the net rotations with the response limits in the CSA S850 blast standard³⁶. These response limits (B1-B4) correspond to specific values of support rotation (θ_{max}) or ductility ratio (μ_{max}). In this study, response limits B1, B2, B3 and B4 were defined using $\mu_{max} = 1$, $\theta_{max} = 4^\circ$, $\theta_{max} = 6^\circ$ and $\theta_{max} = 10^\circ$, respectively. These limits in turn define component damage levels: “Blowout” (greater than B4), “Hazardous failure” (between B4 and B3); “Heavy” (between B3 and B2); “Moderate” (between B2 and B1) and “Superficial” (less than B1). In this study, Blast 1 tested the columns under elastic conditions ($\mu_{max} \leq 1$) and aimed at causing “superficial” damage, while Blast 2 was intended to bring the longitudinal steel in most columns near yielding and cause “moderate” damage ($\theta_{max} \leq 4^\circ$). Blasts 3 and 4 were intended to cause “heavy” ($4^\circ < \theta_{max} \leq 6^\circ$) and “hazardous” damage ($6^\circ < \theta_{max} \leq 10^\circ$), respectively.

Effect of parameters on midspan displacements

In this section the results from Blasts 1-4 are compared to investigate the effects of the test parameters on maximum and residual displacements, support rotations, blast capacity and failure mode.

Effect of concrete strength — The effect of concrete strength can be examined by comparing the response of the following two sets of specimens which were built with high-strength concrete and normal-strength concrete, respectively: 1) HSC-#4 vs. NSC-#4 and 2) HSC-15M vs. NSC-15M. It is noted that all four columns had non-seismic ties, with 4-No.4 and 4-15M bars used in Sets 1 and 2, respectively. Comparative displacement results for these columns are shown in **Fig. 8**.

In the No.4 set the two companion columns recorded similar displacements at Blasts 1 and 2 (17 and 35 psi), with support rotations of $\theta_{max} \approx 0.5^\circ$ and $\theta_{max} \approx 1.2^\circ$. Damage was limited to hairline cracks at Blast 1 and further crack opening at Blast 2, which can indeed be qualified as “superficial” and “moderate”. The low residual displacements also confirm that Blast 1 kept the columns in the elastic range, while the permanent deformations at Blast 2 confirm that this shot brought the columns to inelastic conditions. Both columns sustained large displacements and significant damage at Blast 3 (80 psi), however reductions of 6% and 14% in maximum and residual displacements were recorded for the HSC column when compared to its NSC companion (**Fig. 8a**). The support rotations for these columns reached $\theta_{max} = 4.8^\circ$ and 5° , with signs of concrete crushing and the formation of plastic hinges at midspan and at the column ends (**Figs. 5a & 5e**), which can indeed be qualified as “heavy” damage. Only column HSC-#4 was tested under a fourth shot (Blast 4 – 100 psi), which resulted in a large support rotation of $\theta_{max} = 12^\circ$, corresponding to “blowout”. This intense blast caused significant concrete damage and cover spalling, with the generation of significant blast fragments at failure (**Fig. 6b**).

The effect of concrete strength can next be examined in the 15M set by comparing the results of columns HSC-15M and NSC-15M. Beginning with Blast 1, the HSC column showed reduced maximum deformation (7 mm [0.3 in.]) when compared to NSC-15M (10 mm [0.4 in.]), with similar support rotations of 0.4° and 0.6° . The HSC column also had reductions of 20% and 49% in maximum and residual deformations at Blast 2 (35 psi) (see **Fig. 8b**). Both columns showed relatively low damage after this shot, with support rotations of $\theta_{max} = 0.9^\circ$ and 1.2°

(“superficial” and “moderate”, respectively). The use of high-strength concrete also resulted in improved control of deformations at Blasts 3-4 (100 psi), with reductions of 20% and 14% in maximum displacements (**Fig. 8c-d**). The HSC and NSC columns underwent support rotations of $\theta_{max} = 4.5^\circ$ vs. 4.8° at Blast 3, with both columns experiencing concrete crushing and the formation of plastic hinges at midspan (“heavy” damage). Likewise the final shot resulted in large rotations of $\theta_{max} = 8.1^\circ$ vs. 9.3° , with significant crack propagation, crushing and cover spalling, which can indeed be qualified as “hazardous” damage (**Figs. 6a & 6c**).

In summary, the results show that the response limits in modern blast codes can be used to assess the blast behavior of both high-strength concrete and normal-strength concrete columns. The results also show that the columns in both sets showed similar response characteristics, regardless of concrete type. Indeed, increasing the concrete strength (f'_c) did not lead to more brittle failures or a reduction in ductility. In fact, moderate enhancements in displacement control (in the range of 10-20%) were observed at most test shots as the concrete strength was increased. To further demonstrate this point, **Fig. 8e** shows the blast responses of companion beams with varying concrete strengths tested by Li. et al. ²⁶. The 125 x 250 x 2400 mm (5 x 10 x 95 in.) beams were singly-reinforced with 2-No.4 bars, with $f'_c = 58$ and 108 MPa (8.4 and 15.7 ksi) for beams C50-No.4-S and C100-No.5-S. Testing was conducted under third-point bending over a span of 2232 mm (88 in.) using a shock-tube. It can be observed that the beams show similar displacement histories, with lower displacements for the beam with 108 MPa concrete.

On the other hand, the results from Blast 4 show that HSC columns can generate significant blast fragments at failure (see high-speed stills in **Fig. 7a** and damage photos in **Fig. 6d**). Previous researchers report that the use of fibers can increase the fragmentation resistance of HSC under blast loads. For example, Luccioni et al. ²⁷ found that fibers improved the spalling resistance of HSC slabs subjected to contact explosions. Similarly, Algassem et al. ²⁸ reported that fibers increased the damage tolerance of HSC beams under blast loads (see **Fig. 7b** which shows the ability of fibers to control crushing and spalling). Likewise, Burrell et al. ¹⁷ reported that fibers were effective in reducing secondary blast fragments in normal-strength SCC columns (see **Fig. 7c** which shows the ability of fibers to reduce concrete damage in the column midspan region). Further research examining the benefits of implementing fibers in HSC columns tested under blast loads is recommended.

Effect of seismic detailing — As noted in the literature survey, previous research is conflicting on the effectiveness of seismic detailing to improve the blast behavior of reinforced concrete columns. The effect of this parameter in HSC columns is investigated by comparing the results of columns HSC-10M and HSC-10M-S which were built with ties spaced at $d/2$ (75 mm [3 in.]) and $d/4$ (38 mm [1.5 in.]), representing non-seismic and seismic designs, respectively.

The displacement response of the columns at Blast 1 to 3 can be observed in **Table 5** and **Fig. 9**. Blast 1 (17 psi) tested both columns within the elastic range and therefore both specimens showed similar responses with support rotations of $\sim 0.5^\circ$. The beneficial effect of seismic design can be observed at Blast 2 (35 psi) which brought the columns into the inelastic range. As shown in **Fig. 9a**, column HSC-10M-S showed significant reductions of 38% and 69% in maximum and residual deformations at this shot, with maximum support rotations of 1.2° vs. 1.9° recorded for the specimens with seismic and nominal details. Similar enhancements occurred at Blast 3 (80 psi), where the use of seismic ties reduced maximum and residual deformations by 20% and 27% (see **Fig. 9b**). Both columns showed spalling and formation of secondary fragments at failure, however damage of core concrete was relatively better controlled in the seismic specimen, with buckling of the compression bars prevented. The seismic and non-seismic columns recorded support rotations of $\sim 6^\circ$ and 7.6° which qualify as “heavy” and “hazardous”.

In summary, the results show that seismic detailing can improve the blast behavior of HSC columns by reducing displacements at equivalent blasts. Similar observations have previously been reported by Burrell et al. ¹⁷ in normal-strength concrete columns. It should be noted that the columns in this study had relatively small cross-sections, with testing conducted under relatively low axial loads (recall that the axial load was chosen based on the nominal axial capacity of the control NSC columns). Further blast research on the effect of seismic detailing in HSC columns having large cross-sections is recommended.

Effect of increased longitudinal steel ratio — Increasing the longitudinal steel ratio has been reported by several researchers as an effective means to enhance the impact and blast response of flexural dominant reinforced concrete elements ²⁶. The effect of this parameter in the HSC columns was studied by comparing the results of specimens HSC-10M, HSC-#4 and HSC-15M which had reinforcement ratios of $\rho = 1.7\%$, 2.2% and 3.4% , respectively. All columns in this set were built with HSC and non-seismic ties. As shown in **Fig. 10** doubling the steel ratio from 1.7% (4-10Mbars) to 3.4% (4-15M bars) resulted in improved control of displacements in the HSC columns. All columns showed similar results at Blast 1 (17 psi), however column HSC-15M reduced maximum displacement by $\sim 50\%$ when compared to HSC-10M at Blast 2 (35 psi), with support rotations of 0.9° and 1.9° (“superficial” vs. “moderate” damage). The benefit of increased steel ratio can also be observed at Blast 3 (80 psi), with a reduction of 41% in maximum deformations when transitioning from 10M to 15M bars, with corresponding support rotations of 7.6° and 4.5° (“hazardous” vs. “heavy” damage). More importantly, this shot

caused failure of column HSC-10M. In comparison the companion with 15M bars survived this shot, with failure delayed to Blast 4 (100 psi). While noting that the columns failed at different blast intensities, more significant damage occurred in the 15M specimen when compared to the companion with 10M bars, especially in terms of the amount of spalling (**Fig. 6d**). Examining the cracking patterns, it can also be observed that more extensive diagonal cracks occurred in the column with 15M bars, which indicates an increase in shear demands. Li. et al.²⁶ previously reported that increasing the steel ratio enhanced the blast resistance of HSC beams, but also increased damage (amount of crushing and spalling). On the other hand, Algassem et al.²⁸ reported that the use of fibers was effective in reducing spalling and controlling crushing in HSC beams, regardless of steel ratio. It is also well known that fibers increase the diagonal tension capacity of concrete, which can counter increased shear demands. Further research examining the benefits of fibers in HSC columns with increased steel ratios is recommended.

Comparing the responses of HSC and UHPC columns — As noted in the literature survey, several recent studies have examined the blast behavior of columns designed with more advanced ultra-high performance concrete (UHPC). Since UHPC shows very high compressive strength it is of interest to compare the blast behavior of HSC and UHPC columns. **Fig. 11** compares the responses of columns HSC-10M and HSC-15M from the current study, with columns UHPC-10M and UHPC-15M from Aoude et al.²⁰ at Blasts 2-3-4. These columns had the same setup, dimensions and reinforcement details (4-10M or 4-15M bars), but were built with fiber-reinforced UHPC having $f'_c = 138$ MPa. The fibers in the UHPC had a length of 12 mm (0.5 in.), diameter of 0.2 mm (0.008 in.) and strength of 2000 MPa (290 ksi) and were added at a volumetric ratio of 2% (160 kg/m³).

The results in **Fig. 11a-b** confirm the superior blast behavior of UHPC when compared to conventional high-strength concrete. In the 10M set, column UHPC-10M showed reductions of 48%-55% and 70%-80% in maximum and residual deformations at Blasts 2-3, with significant reductions in maximum support rotations ($\theta_{max} = 1^\circ$ vs. 2° and 3.5° vs. 7.6°), reduced damage, and an ability to sustain one more blast before failure. Indeed, the UHPC column showed no visible damage prior to failure at Blast 4, which occurred due to tension bar rupture at the critical crack (see **Fig. 11e**). The difference in failure mode can be explained by the very high toughness of UHPC in compression, coupled with its high bond capacity, which transfers failure from concrete in compression to steel in tension. Despite the brittle failure, no secondary fragments were formed at UHPC column failure.

The same trend of improved response is observed when comparing the responses of columns UHPC-15M and HSC-15M in **Fig. 11c-d**. While both columns showed similar results early on at Blast 2, the UHPC column showed reductions of 33% and 47% in maximum deformations and 70% and 67% in residual displacements at Blasts 3 and 4, with support rotations of $\theta_{max} = 3^\circ$ vs. 4.5° and 4.3° vs. 8.1° . Increasing the tension steel ratio prevented bar rupture in the UHPC column at Blast 4 (**Fig. 11f**), allowing for better utilization of UHPC material capacity.

In summary, the results show that the use of UHPC results in improved blast behavior when compared to conventional HSC columns. In addition to its high-strength, UHPC contains steel fibers which provide the material with exceptional tensile resistance, toughness and damage tolerance, which translates into superior blast performance. Further research on HSC columns reinforced with steel fibers is recommended.

ANALYTICAL INVESTIGATION

Single-degree-of-freedom (SDOF) modelling is commonly used in practice to assess the blast performance of normal-strength reinforced concrete columns⁴. In this study, the ability of SDOF analysis to predict the blast response of HSC columns is investigated. In this study, the dynamic resistance functions were developed using non-linear sectional analysis incorporating dynamic material properties, as described in the following sections.

Material models

The blast response of the columns was predicted using SDOF analysis after defining the material response for concrete and steel reinforcement at high-strain rates. A summary of the models is shown in **Table 6** and **Fig. 12**.

Concrete in compression — The response of confined concrete in compression was modelled using the stress-strain relationship proposed by Légeron and Paultre²⁹. This model accounts for important parameters affecting the performance of confined concrete, including the spacing, configuration and yield stress of the transverse steel, bar arrangement, and concrete strength. In this model, confinement is taken into account using a "confinement index" ($I_E = f_{le}/f'_{co}$), where the effective confining pressure (f_{le}) is calculated using the nominal confining pressure provided by the steel ties (f_l), and an "effective confinement coefficient" (K_e) which accounts for the arching action which occurs between the levels of the transverse steel in rectangular columns:

$$f_{le} = K_e f_l \quad (1)$$

$$K_e = \frac{\left(1 - \frac{\sum w_i^2}{6c_x c_y}\right) \left(1 - \frac{s'}{2c_x}\right) \left(1 - \frac{s'}{2c_y}\right)}{1 - \rho_c} \quad (2)$$

In the above equation, s' , c_x , c_y , $\sum w_i^2$ and ρ_c represent the clear spacing between ties, the width of the concrete core in the x and y directions, the sum of the squares of the clear spacing between adjacent longitudinal bars and the ratio of longitudinal steel in the core region, respectively. The model defines the stress-strain curve using the point of peak confined strain and stress (ϵ'_{cc} , f'_{cc}) and the strain at 50% drop in peak stress (ϵ_{cc50} , $0.5f'_{cc}$), with an exponential descending branch, as described in **Table 6**.

The response of unconfined confined concrete in the cover region was simulated using the model proposed by Popovics³⁰. The equations used in this model are summarized in **Table 6**. To account for spalling, the cover response was assumed to drop to 40% f'_{co} at a strain of 0.003, with zero stress at a strain of 0.0035.

Previous research indicates that concrete experiences an increase in strength under dynamic loading. These studies shows that compressive strength, aggregate stiffness and specimen size are factors that affect the dynamic properties of concrete (Soroushian et al.³¹; Bischoff and Perry³²; Elfahal and Krauthammer³³). Guo et al.³⁴ examined the dynamic compressive properties of three high-strength concretes with static strengths of 60, 80 and 110 MPa using a Split Hopkinson Pressure Bar (SHPB) device. Rate dependence was quantified using the dynamic increase factor (DIF), which can be defined as the ratio of dynamic to static strength. All three concretes exhibited significant rate sensitivity, with an increase in strength under dynamic loading. Samples with C60 and C80 concrete showed similar DIF, with lower values for the C110 concrete.

In the current study the CEB³⁵ equations were used to estimate the rate sensitivity of concrete since this model accounts for the influence of increased compressive strength on dynamic resistance (see **Table 6**). Using this model, and a strain-rate of 1 s^{-1} for the shock-tube tests, DIFs of 1.30 and 1.15 were determined for NSC and HSC, respectively. **Fig. 12a-b** show typical stress-strain curves for confined and unconfined concrete, with and without DIF applied. To examine the effect of DIF model, two additional analysis cases were conducted using design DIF factors for reinforced concrete columns in the far design range³⁶ (Case 2: DIF = 1.19), and ignoring dynamic effects (Case 3: DIF = 0).

Longitudinal steel in tension and compression — The response of the longitudinal steel reinforcement in tension was modelled using the relationships proposed by Jacques et al.³⁷. As shown in **Table 6**, this model consisted of two linear segments to describe elastic and post-yield behavior, and a parabolic function to describe the effect of strain-hardening. Previous research indicates that reinforcing steel experiences an increase in strength under dynamic loading, with the increase in stress being greater at yield than ultimate³⁸. To account for the effects of high-strain rates, the Saatcioglu et al.³⁹ expressions (see **Table 6**) were used to determine DIFs of 1.30 and 1.1 for steel at yield and ultimate stress (Case 1). Sample stress-strain curves with and without DIF are shown in **Fig. 12c**. Two additional analysis cases were considered, including Design DIFs of 1.17 & 1.05 at yield & ultimate³⁶ (Case 2), and ignoring dynamic effects (Case 3: DIF = 0).

Bar buckling in compression was modeled using the relationships proposed by Yalcin and Saatcioglu⁴⁰, with the stability of the compression bars expressed as a function of bar aspect ratio (s/d_b), defined as the ratio of the unsupported bar length between adjacent ties and the longitudinal bar diameter (see **Fig. 12d**).

SDOF analysis

After defining the constitutive material models, the dynamic response of the columns was predicted using software *RCBlast*⁴¹ by solving the equation of motion shown below:

$$K_{LM}\ddot{u}(t) + R(u(t)) = AP_r(t) \quad (3)$$

where $\ddot{u}(t)$ and $u(t)$ are the acceleration and displacement at column mid-height, $R(u(t))$ is the column resistance as a function of displacement, and K_{LM} is a load-mass factor used to transform the column into an equivalent SDOF system⁴². In the analysis, the mass m was taken to be 315 kg (694 lb.), equivalent to the total mass of the column and load-transfer device. The loading function was defined using the time-variant blast pressure $P_r(t)$ and the loaded area, A , taken as area of the shock-tube end-frame opening (4.12 m^2 [44 ft^2]). An equivalent triangular blast load with the same peak reflected pressure and impulse recorded in the experiments was used in the analysis (see **Table 7**).

Analysis using *RCBlast* began by defining the cross-sectional properties of the columns and constitutive models for the concrete and steel reinforcement as described above. Using this data, the program computed the moment-curvature relationship of the column using sectional analysis. With the moment-curvature relationship and a user-defined plastic hinge length, the resistance functions were then computed using the lumped inelasticity approach (**Fig. 12e-f**). Detailed description of this method is provided in Jacques et al.³⁷. The plastic hinge length at midspan and at the supports was taken to be equal to the effective depth, defined as the distance between the extreme compression fiber and tension steel bars. During testing the columns experienced a reduction in axial load during maximum response due to column shortening and associated horizontal deformations under lateral blast pressures³⁷. To account for variable axial load, composite resistance functions were determined using 6 axial load steps

from 0 to 294 kN (66 kips) using an available function in *RCBlast*⁴¹. The load-deformation characteristics generated by the program and the previously defined idealized triangular blast loads were then used to solve the equation of motion using the average acceleration method.

SDOF analysis results

Fig. 13a shows a sample analysis result, while **Fig. 13b** and **Table 7** compare the predicted results (d_{anls}) and experimental maximum displacements (d_{exp}), as well as their ratio (d_{anls}/d_{exp}) at Blasts 1-4. Considering all columns and all blast scenarios, the average d_{anls}/d_{exp} ratio was found to be 1.04 with an average absolute error of 14%, and a coefficient of variation of 15.1%. Overall, the results show reasonable agreement between the SDOF analysis predictions and the experimental results for most of the HSC and NSC columns. Considering the individual blasts, it can be observed that the analysis under-predicted the displacements at Blast 3 (average d_{anls}/d_{exp} ratios of 1.17, 1.03 and 0.94 at Blasts 1, 2 and 3, respectively), and this can be explained by the effect of accumulated damage from repeated testing which was not considered in the analysis.

The effect of DIF model is illustrated in **Fig. 13c**, where it can be seen that ignoring strain-rate effects (DIF = 0, Case 3) and the use of constant design DIF values (Case 2) resulted in higher d_{anls}/d_{exp} ratios when compared to Case 1 (strain-rate dependent DIF models). When considering all blasts, the d_{anls}/d_{exp} ratios for Cases 1, 2 and 3 were found to be 1.04, 1.08 and 1.14, which follows the expected trend. In general, the conservative results from Case 3 confirm that dynamic material properties should be considered in the blast analysis of HSC and NSC columns. On the other hand, the use of design DIF models resulted in reasonably accurate predictions.

Parametric study on larger-scale columns

After validation of the analysis methodology, the blast behavior of larger scale columns having dimensions of 3.5 m × 350 mm × 350 mm (14 in. × 14 in. × 138 in.) was analyzed using the same SDOF procedure presented in the previous section. **Fig. 14a** and **Table 8** describe the design details of the columns, which had two different cross-sections (Configurations-A and B: with 8 bars + single ties and 12 bars + double-ties), three longitudinal steel ratios (use of 15M, 20M and 30M bars), three transverse steel detailing levels ($R_d = 1.5, 2.5$ and 4.0, as defined in the CSA A23.3 standard²³), with concrete strengths of 40 MPa [6 ksi] (NSC) and 80 MPa [12 ksi] (HSC). In all cases, the columns were assumed to be fully-fixed and subjected to a constant axial load corresponding to 40% of the nominal capacity of the control 40 MPa columns. Blast loads were simulated in software *RCBlast* for a constant standoff of 20 m and varying TNT charge weights, increased in 50 kg (110 lb.) increments up to 700 kg (1550 lb.) or column failure. The results of the analysis are shown in **Fig. 14**, which shows a comparison of column displacements as a function of TNT charge weight.

Beginning with the effect of concrete strength, **Fig. 14b-c** show that the use of HSC in the columns with 15M bars and $R_d=4$ detailing reduced displacements at equivalent blasts without increasing blast resistance, which is similar to the observation in the experiments. On average, the displacements were reduced by 17% for both Configurations-A and B as the concrete strength was increased from 40 to 80 MPa.

Next, the effect of longitudinal steel ratio is examined in **Fig. 14d-e** which compares the results of the Configuration-A normal-strength and high-strength concrete columns with 15M, 20M and 30M bars ($\rho = 1.3\%, 2\%$ and 3%). It can be seen that increasing the steel ratio reduced displacements, and increased blast resistance, for both concrete types, which is similar to the observations in the experiments.

Finally, the effect of transverse steel detailing in the HSC columns can be examined in **Fig. 14f-g**. No significant differences in performance are observed when varying detailing in the HSC columns at the early blasts, however the use of “ductile” ($R_d = 4.0$) detailing increased blast capacity and delayed column failure in both Configurations-A and B. Moreover, it can be observed that improving the detailing within the cross-section (i.e. use of double ties vs. single ties [Type B vs. A]) allowed for greater blast capacity, especially for the columns with $R_d = 2.5$ detailing. As noted before, the scaled columns in the experimental study had relatively small cross-sections; based on the observations in the numerical study, further blast research examining the effect of transverse steel detailing in HSC columns have large cross-sections is recommended.

SUMMARY, FINDINGS AND CONCLUSIONS

This paper presented the experimental results from six columns built with normal-strength concrete and high-strength concrete which were tested under simulated blast loading using a shock-tube. The tests examined the effects of concrete strength, seismic detailing and longitudinal reinforcement ratio on blast response. The following conclusions can be drawn from this study:

1. The results showed that the use of high-strength concrete did not significantly affect the blast response of the columns, in terms of blast capacity or failure mode when compared to normal-strength concrete.

However, when considering the results at equivalent blasts, doubling the concrete strength from 40 MPa to 80 MPa (6 to 12 ksi) resulted in 10%-20% reductions in maximum displacements;

2. The results showed that the use of seismic detailing improved the response of the HSC columns by better controlling deformations at equivalent blasts, while also enhancing damage control in terms of core crushing and prevention of bar buckling. In this study, reducing the tie spacing from $d/2$ to $d/4$ (75 to 38 mm [3 to 1.5 in.]), led reductions of 20-40% in maximum deformations at equivalent blasts, but did not increase blast capacity;
3. Increasing the longitudinal steel ratio significantly improved the blast response of the HSC columns. In this study, increasing the steel ratio from $\rho = 1.7\%$ to 3.4% (10M vs. 15M bars) led to reductions of 40-50% in maximum deformations at equivalent blasts. In addition to reducing blast-induced displacements, doubling the steel ratio increased blast capacity, with failure delayed from Blast 3 to Blast 4. On the other hand, increased damage in terms of spalling at failure was observed as the steel ratio was increased;
4. Results from the analytical study showed that SDOF analysis was able to predict the blast response of the HSC columns with reasonable accuracy. The resistance functions were developed using dynamic material properties, sectional analysis, and a lumped inelasticity approach. Considering all columns and all blast scenarios, the average d_{anls}/d_{exp} ratio was found to be 1.04 with an average absolute error of 14%, and a coefficient of variation of 15.1%. The results were shown to be sensitive to the choice of DIF model, with more conservative results obtained when using static material properties. Use of design DIF factors were shown to result in similar predictions when compared to the strain-rate dependent models.
5. A parametric SDOF study on larger-scale columns was also presented. The results showed that increasing concrete strength from 40 to 80 MPa (6 to 12 ksi) did not affect blast capacity, but resulted in better control of displacements by an average of 17% in the larger-scale columns. On the other hand, increasing the longitudinal steel ratio from 1.3% to 3% increased blast capacity in both the NSC and HSC columns. Likewise, the numerical results showed that seismic detailing increased the blast capacity of the HSC columns.

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Table 1 – Details of column specimens

Column ID	Concrete mix	Avg. Concrete Strength MPa (ksi)	Long. Reinforcement		Transverse Reinforcement
			Configuration	Steel Ratio ρ (%)	spacing mm (in.)
NSC-#4	NSC	40.6(5.9)	4 – No.4	2.2	75 (3.0)
NSC-15M		40.6(5.9)	4 – 15M	3.4	
HSC-10M	HSC	93.9(13.5)	4 – 10M	1.7	75 (3.0)
HSC-10M-S		93.9(13.5)	4 – 10M	1.7	38 (1.5)
HSC-#4		84.7(12.6)	4 – No.4	2.2	75 (3.0)
HSC-15M		84.7(12.6)	4 – 15M	3.4	

Table 2 – NSC and HSC concrete mix details

Concrete Type	GU Cement (kg/m ³)	Slag (kg/m ³)	Silica Fume (kg/m ³)	Coarse* Aggregate (kg/m ³)	Sand (kg/m ³)	Water	WR ^a	SR ^b
NSC	300	--	--	1200	600	180	--	--
HSC	373	164	48	1120	740	176	5.85	1.93

Note: 1 kg/m³ = 0.0624 lb/ft³

*maximum aggregate size = 19 mm (3/4 in.) for NSC and 9.5 mm (3/8 in.) for HSC

^a Water-reducing admixture: BASF MasterGlenium 7500 (added to mixing water)^b Set-retarding admixture: BASF MasterSet R 100 (added to mixing water)

Table 3 – Reinforcing steel properties

Rebar Type	Diameter mm (in.)	Area mm ² (in. ²)	Yield		Ultimate	
			Stress f_y MPa (ksi)	Strain ϵ_y	Stress f_u MPa (ksi)	Strain ϵ_u
Steel wire	6.3 (0.25)	31 (0.05)	542(78.2)	0.0028	594(85.7)	594(85.7)
10M	11.3(0.45)	71 (0.11)	474(68.4)	0.0026	582(84.0)	582(84.0)
No.4	12.7(0.51)	100 (0.16)	464(67.0)	0.0027	695(100.3)	695(100.3)
15M	16.0(0.64)	200 (0.31)	449(64.8)	0.0026	596(86.0)	596(86.0)

* yield point determined 0.2% offset method

Table 4 – Shocktube driver parameters and average shockwave properties

Blast ID	Driver Pressure P_d kPa (Psi)	Driver Length L_d mm (ft)	Avg. Reflected Pressure P_r kPa (Psi)	Avg. Reflected Impulse I_r kPa*msec (Psi *msec)	Avg. Positive Phase Duration t_d msec
Blast 1	117 (17)	2743 (9)	22.1(3.2)	214.5(31.1)	22.4
Blast 2	241 (35)	2743 (9)	40.7(5.9)	422.1(61.2)	24.1
Blast 3	552 (80)	2743 (9)	72.9(10.6)	796.7(115.5)	26.9
Blast 4	690 (100)	2743 (9)	87.8(12.7)	1069.3(155.0)	31.0

Table 5 – Experimental results: shockwave properties and midspan displacements

Column ID	Blast ID	Shockwave Properties ¹			Net midspan Displacements and Support Rotations ²			Cumulative Displacements ³		CSA S850 Response limits and Component damage ⁴	
		P_r (kPa)	t_d (msec)	I_r (kPa*msec)	d_{max} (mm)	d_{res} (mm)	θ_{max} (°)	d_{max}^c (mm)	d_{res}^c (mm)	Response Limit	Damage level
NSC-#4	Blast 1	22.1	23.6	242	10.7	-	0.6	10.7	0	<B1	Superficial
	Blast 2	40.3	24.4	464	20.5	5.5	1.2	20.5	5.5	B1-B2	Moderate
	Blast 3	65.1	26.8	854	87.4	51.3	5.0	92.9	56.8	B2-B3	Heavy
NSC-15M	Blast 1	22.2	22.6	193	7.4	1.3	0.4	7.4	1.3	<B1	Superficial
	Blast 2	38.9	23.0	428	20.0	6.7	1.2	21.3	8	B1-B2	Moderate
	Blast 3	71.9	26.6	846	84.0	54.8	4.8	92	62.8	B2-B3	Heavy
	Blast 4	83.2	32.2	1098	162.8	136.6	9.3	225.6	199.4	B3-B4	Hazardous
HSC-10M	Blast 1	23.0	23.2	233	8.4	1.3	0.5	8.4	1.3	<B1	Superficial
	Blast 2	45.1	26.4	429	33.6	14.7	1.9	34.9	16	B1-B2	Moderate
	Blast 3	75.4	25.0	779	132.6	118.7	7.6	148.6	134.7	B3-B4	Hazardous
HSC-10M-S	Blast 1	19.8	21.6	211	9.5	2.4	0.5	9.5	2.4	<B1	Superficial
	Blast 2	41.2	24.6	378	20.8	4.6	1.2	23.2	7	B1-B2	Moderate
	Blast 3	74.1	26.4	731	106.1	87.1	6.0	113.1	94.1	B3-B4	Heavy
HSC-#4	Blast 1	24.4	21.0	181	7.2	2.0	0.4	7.2	2	<B1	Superficial
	Blast 2	39.6	23.3	462	20.1	7.8	1.2	22.1	9.8	B1-B2	Moderate
	Blast 3	78.3	31.8	875	82.4	44.3	4.8	92.2	54.1	B2-B3	Heavy
	Blast 4	92.3	27.8	1112	210.6	173.5	12.0	264.7	227.6	>B4	Blowout
HSC-15M	Blast 1	21.3	22.4	226	7.6	1.7	0.4	7.6	1.7	<B1	Superficial
	Blast 2	39.0	23.0	372	16.0	3.4	0.9	17.7	5.1	<B1	Superficial
	Blast 3	72.8	24.6	696	77.7	54.5	4.5	82.8	59.6	B2-B3	Heavy
	Blast 4	87.8	33.0	998	141.1	117.6	8.1	200.7	177.2	B3-B4	Hazardous

Note: 1 kPa = 0.145 Psi, 1 mm = 0.0394 in.

¹ P_r = reflected pressure; I_r = reflected impulse, t_d = positive phase duration;

² d_{max} , d_{res} and θ_{max} = net maximum midspan displacement, net residual midspan displacement and net support rotation (non-cumulative);

³ d_{max}^c and d_{res}^c = cumulative maximum and residual displacements (includes permanent deformations from previous shots);

⁴: Component damage and response limits: “Blowout” (Component is overwhelmed by blast load causing debris with significant velocities) = response greater than B4; “Hazardous failure” (component has failed with no significant velocities) = response in between B4 and B3; “Heavy damage” (Component has not failed but has significant permanent deflections causing it to be unrepairable) = response in between B3 and B2; “Moderate damage” (Component has permanent deflections but is repairable) = response in between B2 and B1; “Superficial damage” (component has no visible permanent damage) = response less than B1. In this study, limits B1-B4 are taken as $\mu_{max} = 1$, $\theta_{max} = 4^\circ$, $\theta_{max} = 6^\circ$ and $\theta_{max} = 10^\circ$.

Table 6 – Constitutive material and dynamic increase factor models

	Material model	DIF model
Confined concrete in compression	<u>Légeron and Paultre [29]:</u> Stress-strain relationship: $f_{cc} = \begin{cases} \frac{f'_{cc} \times \left(k \times \frac{\epsilon_{cc}}{\epsilon'_{cc}}\right)}{k - 1 + \left(\frac{\epsilon_{cc}}{\epsilon'_{cc}}\right)^k} & \text{for } \epsilon_{cc} \leq \epsilon'_{cc} \\ f'_{cc} \times \exp(k_1(\epsilon_{cc} - \epsilon'_{cc})^{k_2}) & \text{for } \epsilon_{cc} > \epsilon'_{cc} \end{cases}$ Where: $f'_{cc} = f'_{co} \left[1 + 2.4 \left(\frac{f_{le}}{f'_{co}}\right)^{0.7}\right]$; $\epsilon'_{cc} = \epsilon'_{co} \left[1 + 35 \left(\frac{f_{le}}{f'_{co}}\right)^{1.2}\right]$ and: f'_{co}, ϵ'_{co} = peak unconfined stress and strain f_{le} = effective confining pressure provided by transverse steel k = parameter affecting slope of ascending branch k_1, k_2 = parameters affecting shape of descending branch	<u>CEB model [35]:</u> $DIF_c = \begin{cases} \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^{1.026\alpha} & \text{for } \dot{\epsilon} \leq 30s^{-1} \\ \gamma \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^{1/3} & \text{for } \dot{\epsilon} > 30s^{-1} \end{cases}$ Where: $\log \gamma = 6.156\alpha - 2$; $\alpha = \frac{1}{(5 + 9f'_{cc}/10)}$; $\dot{\epsilon}_s = 30 \cdot 10^{-6}s^{-1}$
Unconfined concrete in compression	<u>Popovics [30]:</u> Stress-strain relationship: $f_c = \frac{f'_{co} \beta_c \left(\frac{\epsilon_c}{\epsilon'_{co}}\right)}{\beta_c - 1 + \left(\frac{\epsilon_c}{\epsilon'_{co}}\right)^{\beta_c k_o}}$ Where: $\beta_c = 0.8 + \frac{f'_c}{17}$; $E_c = 3320\sqrt{f'_c} + 6900$; $\epsilon_{c0} = \frac{f'_c}{E_c} * \frac{\beta_c}{\beta_c - 1}$ $k_o = \begin{cases} 1.0 & \text{Ascending} \\ 0.67 + \frac{f'_c}{62} & \text{Descending} \end{cases}$	<u>CEB model [35]:</u> (same as above, with f'_{co} when determining α)
Steel in tension	<u>Jacques et al. [37]:</u> $f_s = \begin{cases} E_s \epsilon_s & \text{for } \epsilon_s \leq \epsilon_y \\ f_y + (\epsilon_s - \epsilon_y) \left(\frac{f_{sh} - f_y}{\epsilon_{sh} - \epsilon_y}\right) & \text{for } \epsilon_y < \epsilon_s \leq \epsilon_{sh} \\ f_y + (f_u - f_y) \left[2 \left(\frac{\epsilon_s - \epsilon_{sh}}{\epsilon_u - \epsilon_{sh}}\right) - \left(\frac{\epsilon_s - \epsilon_{sh}}{\epsilon_u - \epsilon_{sh}}\right)^2\right] & \text{for } \epsilon_{sh} < \epsilon_s \leq \epsilon_u \end{cases}$	<u>Saatcioglu et al. [39]:</u> At yield $\rightarrow DIF_y = 0.034 \ln(\dot{\epsilon}) + 1.30 \geq 1.0$ At ultimate $\rightarrow DIF_u = 0.0101 \ln(\dot{\epsilon}) + 1.10 \geq 1.0$

Note: f_{cc} – confined concrete stress at strain ϵ_{cc} ; f'_{cc} , ϵ'_{cc} – peak confined concrete stress and strain; f_c – unconfined concrete stress at strain ϵ_c ; f'_{co} , ϵ'_{co} – peak unconfined concrete stress and strain; E_c – concrete elastic modulus; f_s – steel stress at strain ϵ_s ; f_y – steel yield stress; ϵ_y – steel yield strain; f_{sh} – steel hardening stress; ϵ_{sh} – steel hardening strain; f_u – steel ultimate stress; ϵ_u – steel ultimate strain; $\dot{\epsilon}_s$ – static strain rate; $\dot{\epsilon}$ – dynamic strain rate; DIF_c , DIF_y & DIF_u : Dynamic increase factors for concrete in compression. and steel at yield & ultimate.

Table 7 – Blast analysis results

Column ID	Shot #	Blast ID ^a	Idealized Shockwave Properties ^{1a}			Midspan Displacements ²			
			P _r (kPa)	I _r (kPa·msec)	t _{di} (msec)	d _{exp} (mm)	d _{anls} (mm)	d _{anls} /d _{exp}	Absolute % Error
NSC- #4	1	Blast 1	22.1	242	21.9	10.70	11.55	1.08	7.9%
	2	Blast 2	40.3	464	23.0	20.47	22.87	1.12	11.7%
	3	Blast 3	65.1	853	26.2	87.38	78.27	0.90	10.4%
HSC-10M	1	Blast 1	23.0	233	20.3	8.38	10.84	1.29	29.4%
	2	Blast 2	45.1	429	19.0	33.64	27.45	0.82	18.4%
	3	Blast 3	75.4	779	20.7	132.57	106.52	0.80	19.6%
HSC-10M-S	1	Blast 1	19.8	211	21.3	9.49	9.86	1.04	3.9%
	2	Blast 2	41.2	378	18.4	20.81	24.42	1.17	17.3%
	3	Blast 3	74.1	731	19.7	106.13	98.98	0.93	6.7%
HSC-#4	1	Blast 1	24.4	181	14.8	7.21	9.01	1.25	25.0%
	2	Blast 2	39.6	462	23.4	20.07	20.45	1.02	1.9%
	3	Blast 3	78.3	875	22.3	82.35	92.36	1.12	12.2%
	4	Blast 4	92.3	1112	24.1	210.58	-- ^b	-- ^b	-- ^b
Statistical analysis			d _{anls} /d _{exp}			Overall	Blast 1	Blast 2	Blast 3
			Mean			1.04	1.17	1.03	0.94
			Standard deviation			0.158	0.123	0.155	0.134
			Coefficient of variation			15.1%	10.6%	15.0%	14.3%
			Max.			1.29	1.29	1.17	1.12
			Min.			0.8	1.04	0.82	0.80

¹ Idealized triangular pressure-time history with same reflected pressure and impulse observed in experiments.

^a P_r = Reflected pressure; I_r = Reflected impulse; t_{di} = Idealized positive phase duration = 2×I_r / P_r;

² d_{exp} = experimental mid-span displacement; d_{anls} = analytical SDOF mid-span displacement.

^b analysis predicts blowout failure

Table 8 – Parametric study column design details

Configuration	Cross-section size (mm x mm)	Longitudinal steel Number of bars & Reinforcement ratio (ρ)			Transverse Steel Spacing & Ratio (ρ _v)		
		15M	20M	30M	R _d = 4.0	R _d = 2.5	R _d = 1.5
A	350 x 350	8 bars 1.31%	8 bars 1.96%	8 bars 2.94%	40 mm 0.76%	125 mm 0.46%	255 mm 0.22%
B	350 x 350	12 bars 1.96%	12 bars 2.94%	12 bars 6.86%	85 mm 1.34%	125 mm 0.91%	255 mm 0.45%

Note: 1 mm = 0.0394 in.

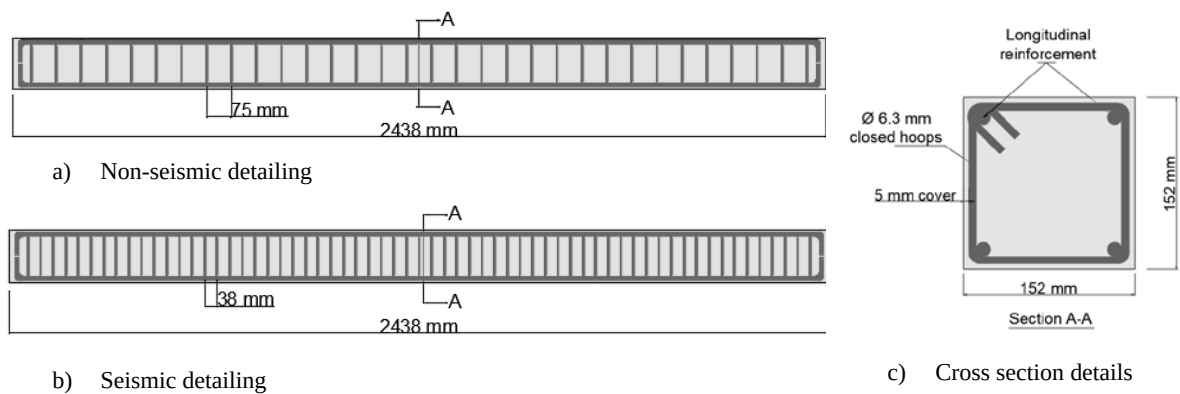


Fig. 1 – Column dimensions and reinforcement details.

Note: 1 mm = 0.0394 in.

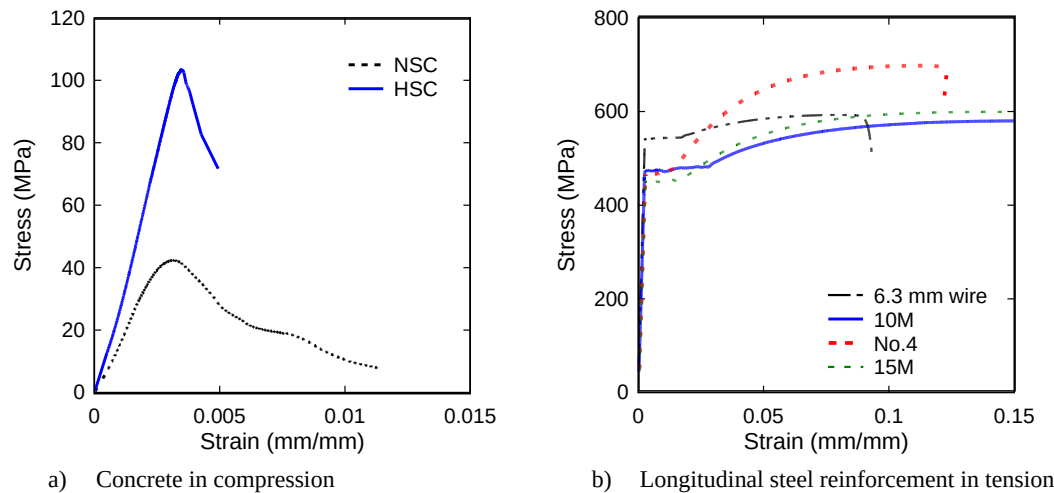


Fig. 2 – Typical stress-strain curves for concrete and steel reinforcement.

Note: 1 MPa = 145 Psi

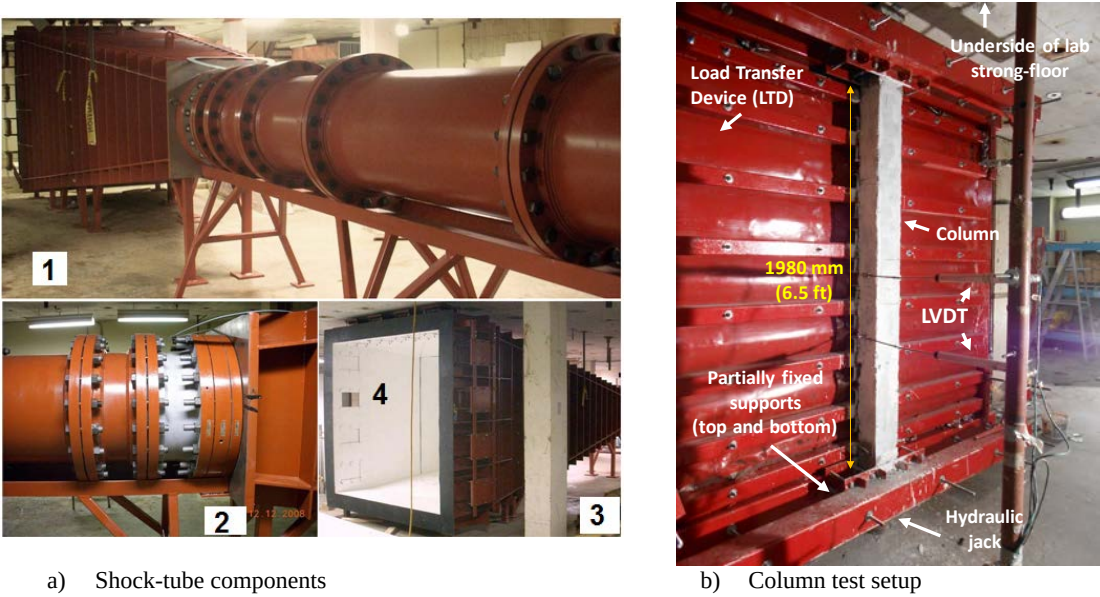


Fig. 3 – Shocktube and column test setup.

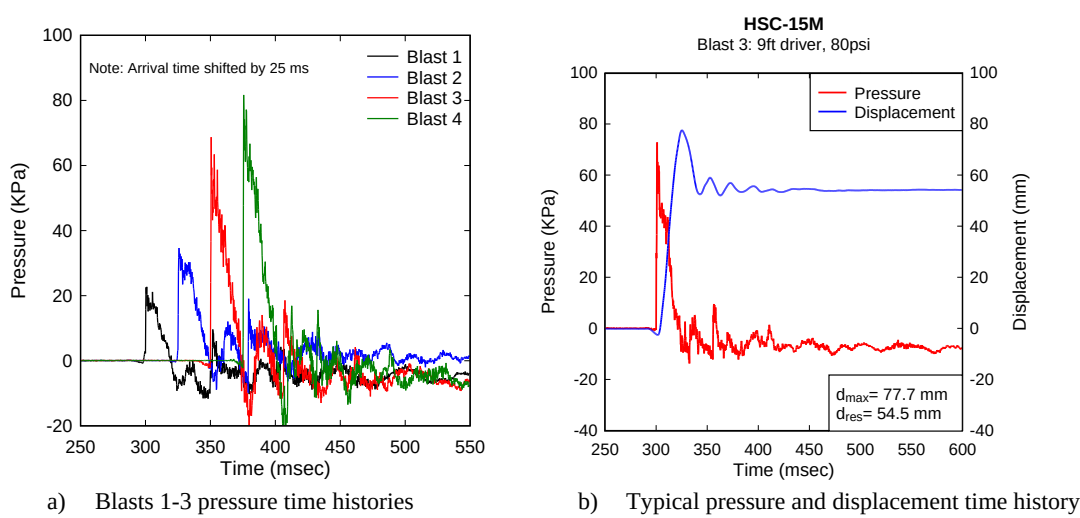


Fig. 4 – Sample pressure and displacement time histories.

Note: 1 kPa = 0.145 Psi, 1 mm = 0.0394 in.

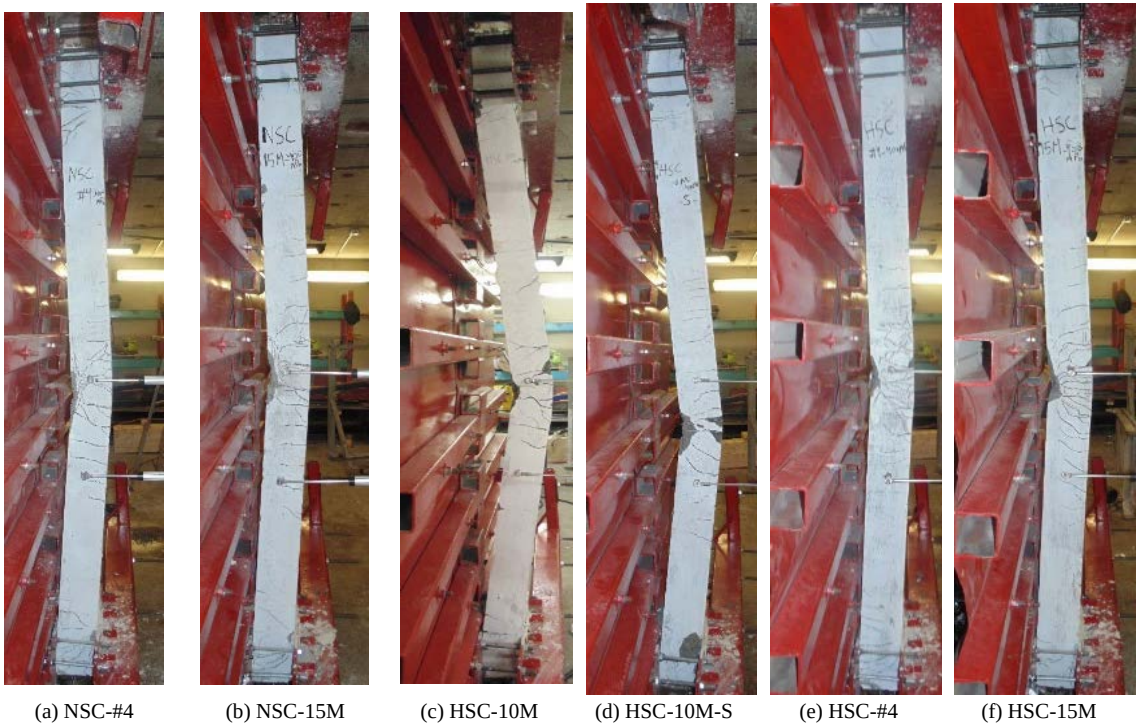


Fig. 5 – Columns after Blast 3 (80 psi).

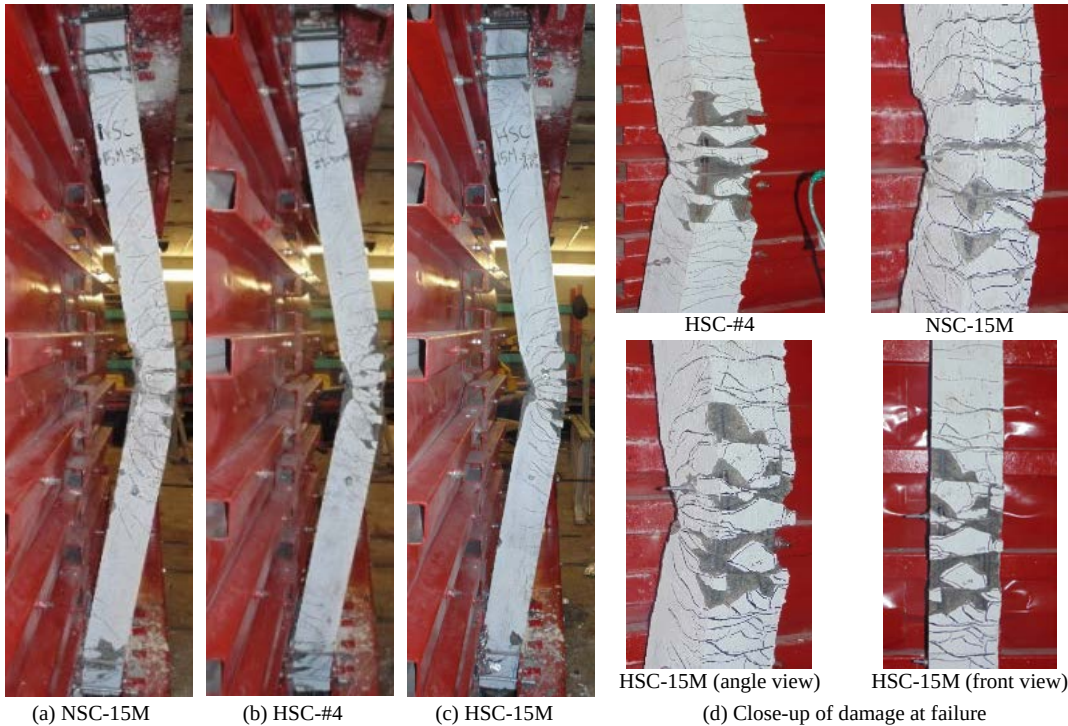
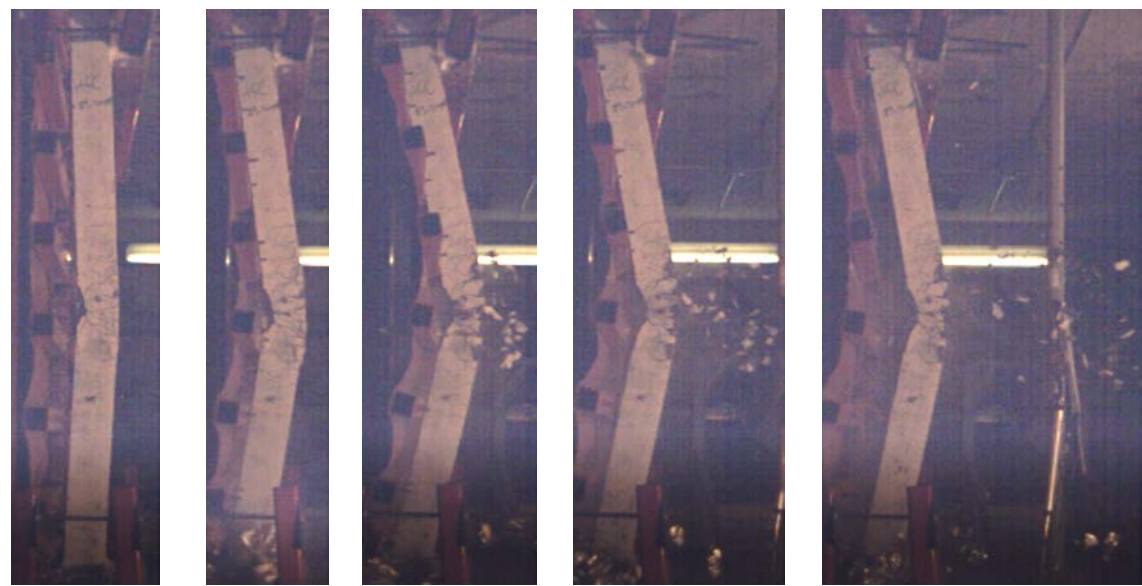
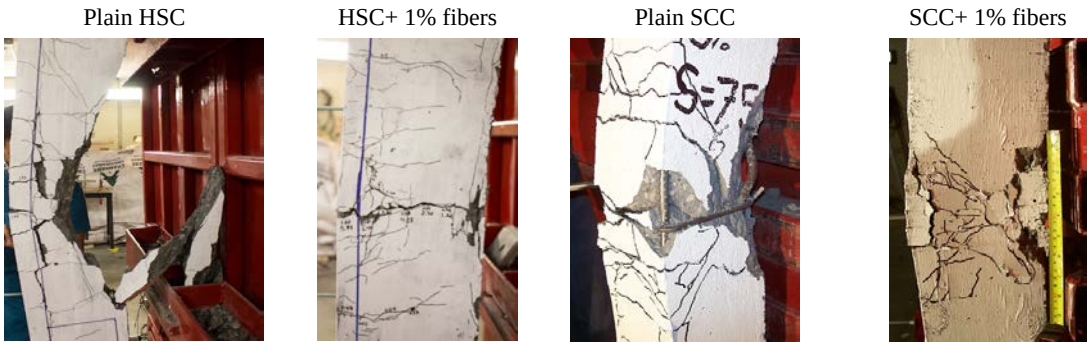


Fig. 6 – Columns after Blast 4 (100 psi).



(a) High speed video still showing secondary blast (HSC-15M at Blast 4)



(b) Ability of fibers to reduce crushing and spalling in HSC beams (from ref [28])

(c) Ability of fibers to reduce damage in normal-strength SCC columns (from ref [17])

Fig. 7 – High speed video still showing secondary blast fragments in HSC columns and examples from previous research showing the ability of fibers to control concrete damage and limit fragments

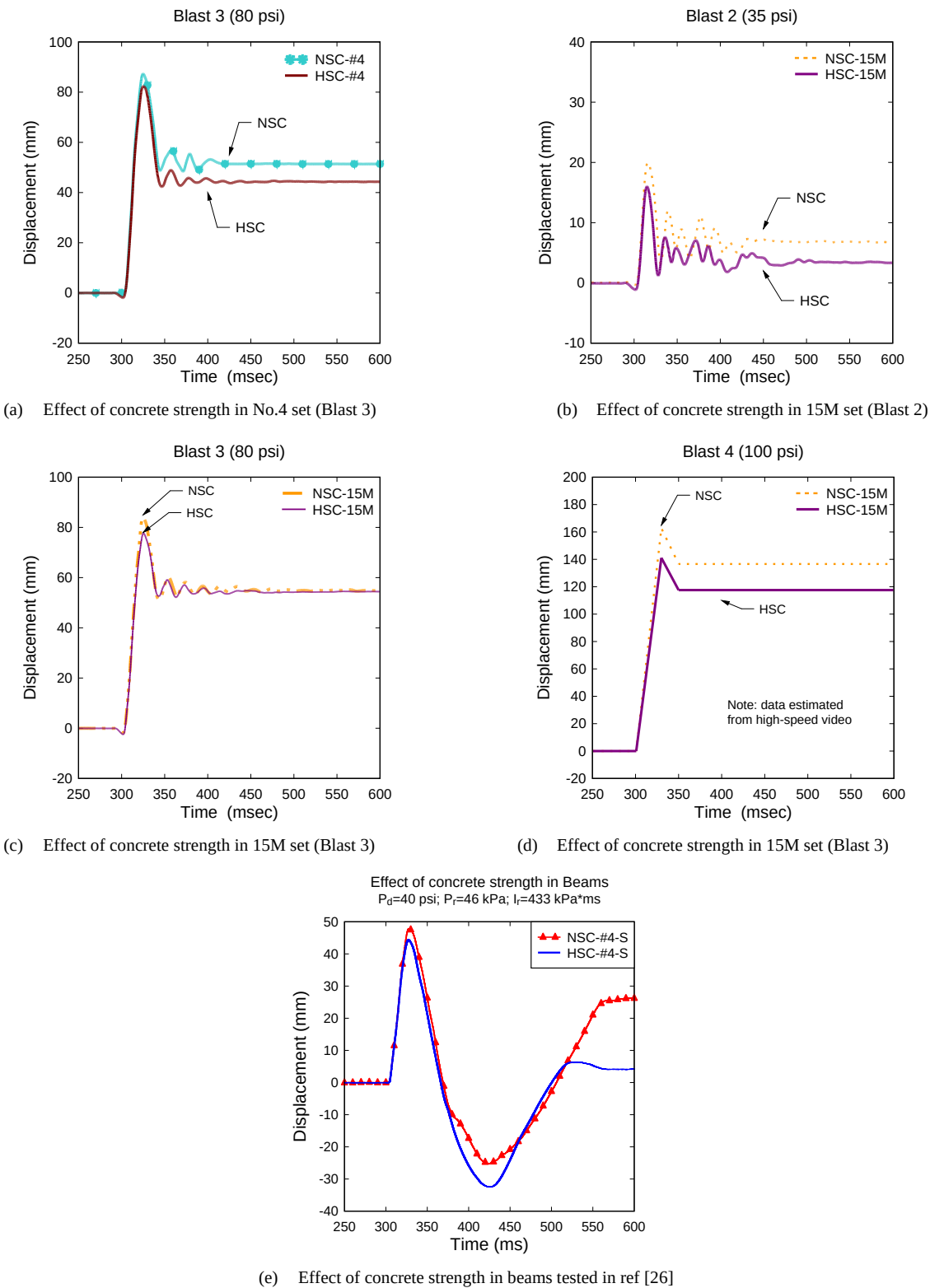


Fig. 8 – Effect of concrete strength in test columns (and beams tested by Li et al. ²⁶)
Note: 1 mm = 0.0394 in.

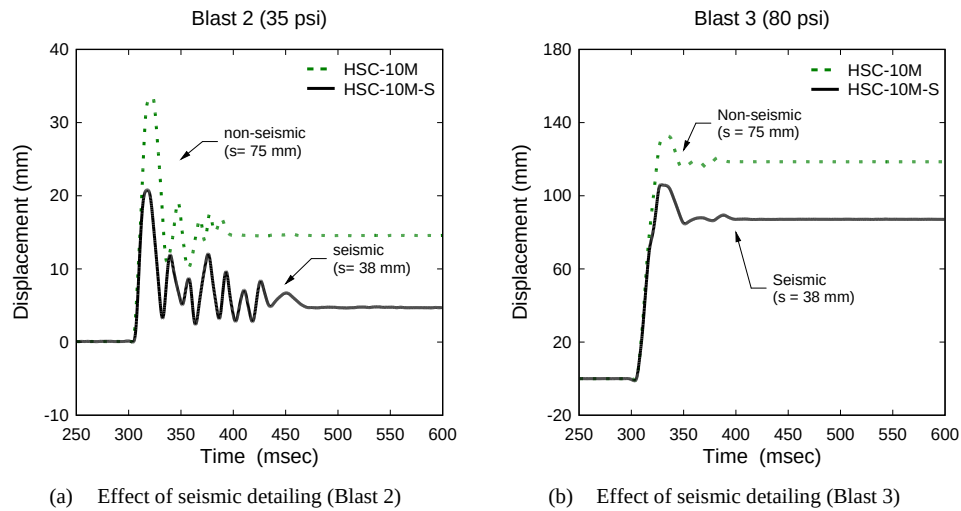


Fig. 9 – Effect of seismic detailing in HSC columns at Blasts 2-3

Note: 1 mm = 0.0394 in.

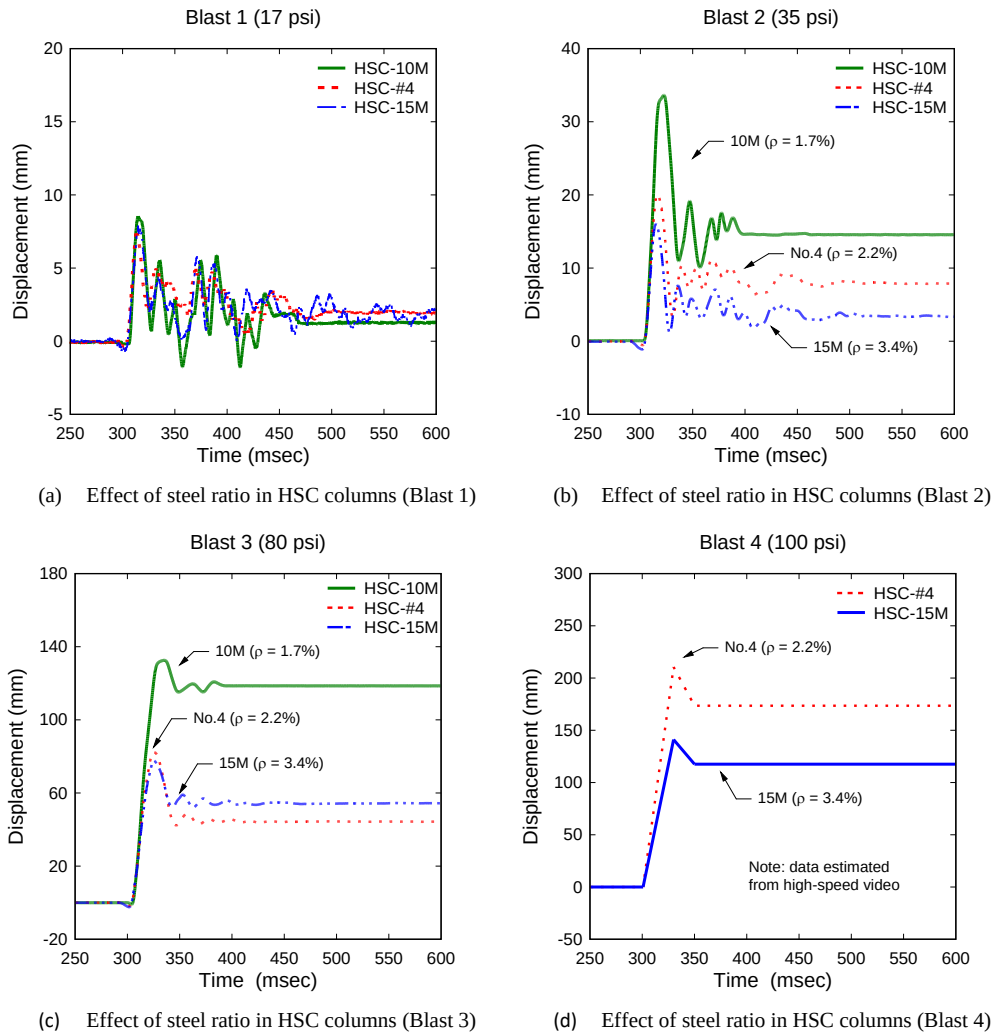
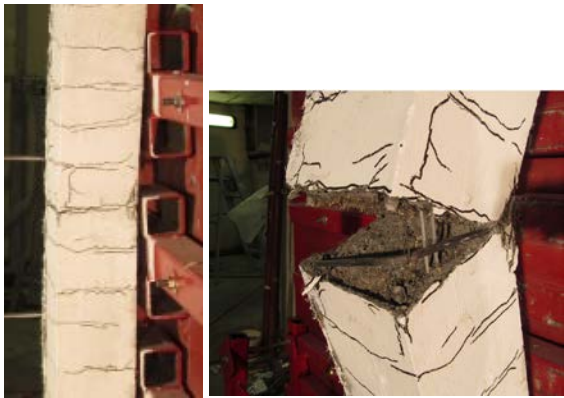
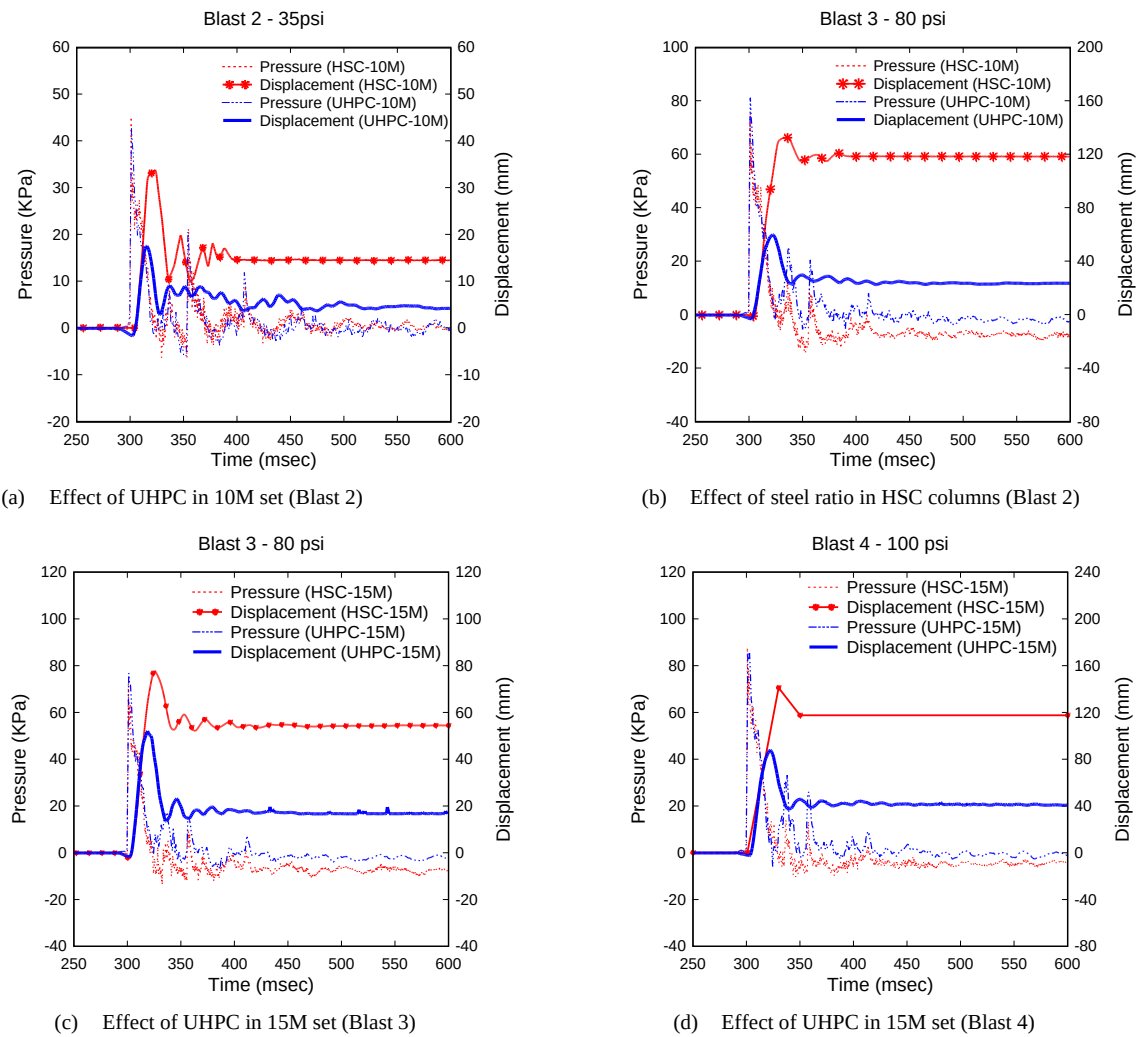
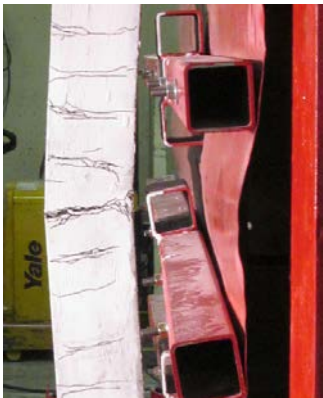


Fig. 10 – Effect of increased longitudinal steel ratio in HSC columns at Blasts 1-4

Note: 1 mm = 0.0394 in.

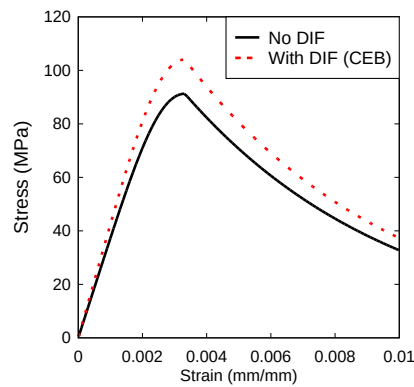


(e) UHPC-10M after Blasts 3 (left) and Blast 4 (right)



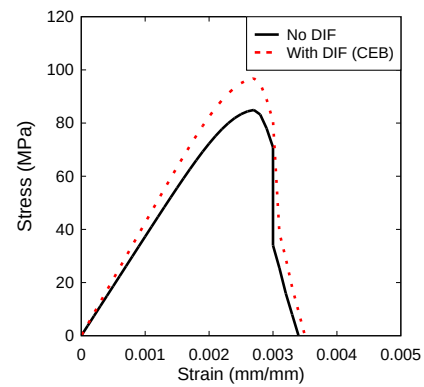
(f) UHPC-15M after Blast 4

Fig. 11 – Effect of concrete type: UHPC vs. HSC columns
Note: 1 mm = 0.0394 in.



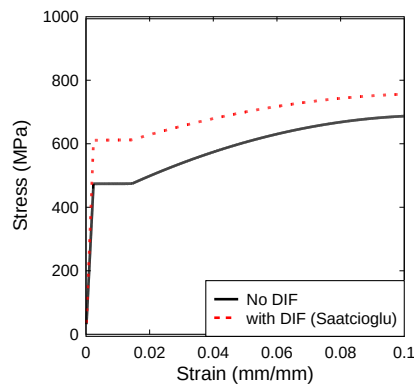
(a) Confined concrete

[Légeron and Paultre + DIF = CEB]



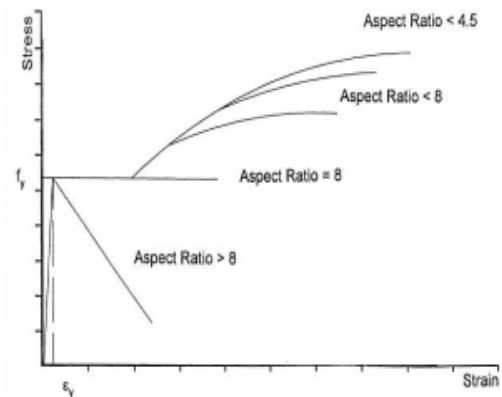
(b) Unconfined concrete

[Popovics + DIF = CEB]



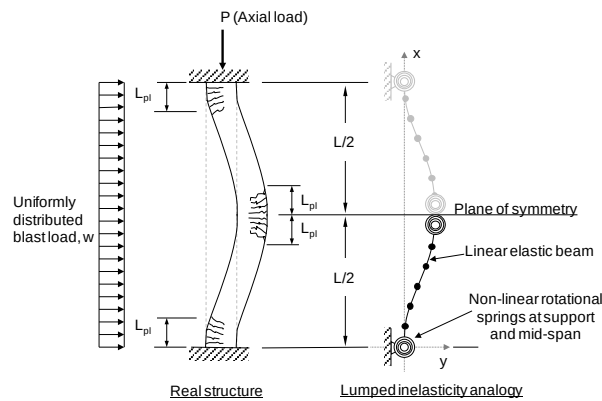
(c) Longitudinal steel in tension

[Jacques et al. + DIF = Saatcioglu]

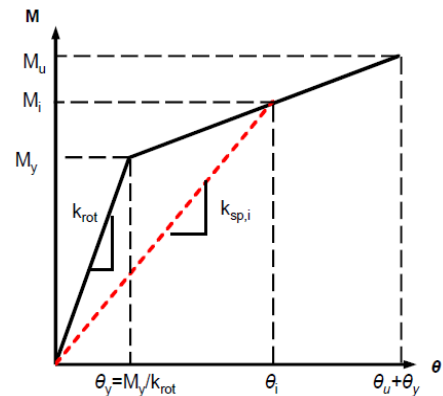


(d) Longitudinal steel in compression

[Yalcin and Saatcioglu]



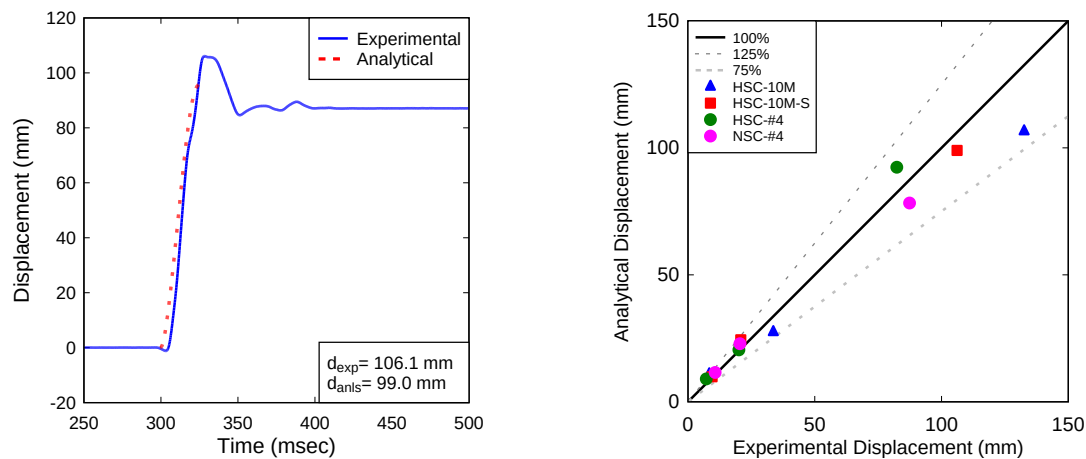
(e) Lumped inelasticity model



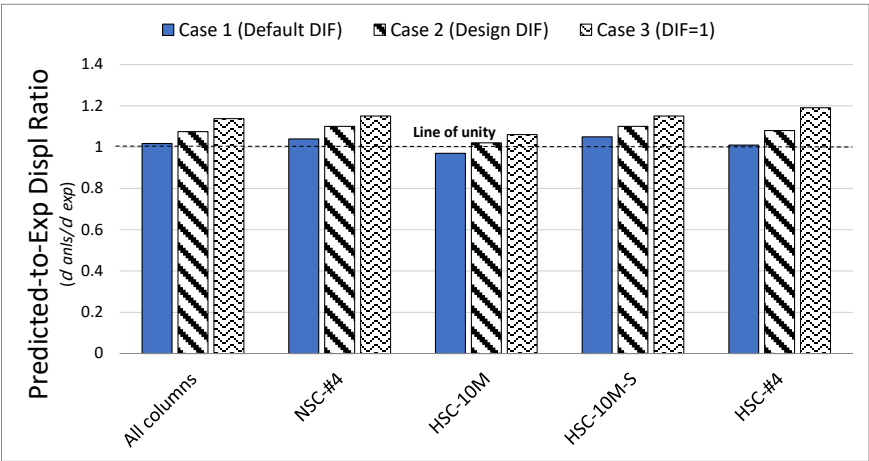
(f) Idealized moment-rotation relationship used in derivation of rotational spring properties

Fig. 12 – Material constitutive models and lumped inelasticity model as defined by Jacques et al. (2012).

Note: 1 MPa = 145 Psi

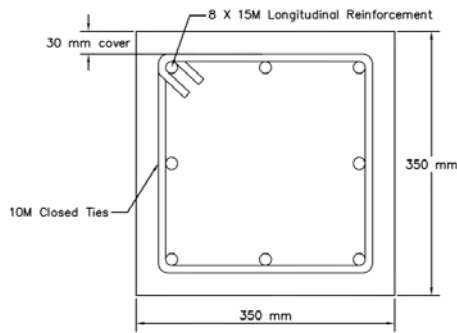


(a) Sample analysis prediction result (HSC-10M-S at Blast 3) (b) Predicted vs. experimental maximum displacements (Blasts 1-3)

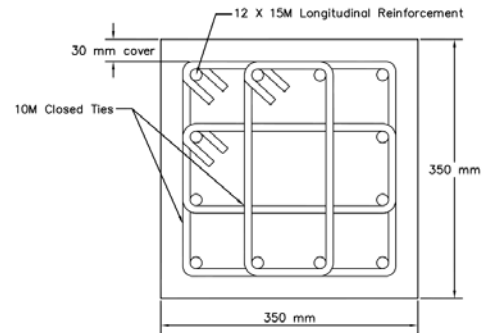


(c) Effect of DIF model on the predicted-to-experimental displacement ratios.

Fig. 13 – Analysis results and effect of DIF model.
Note: 1 mm = 0.0394 in.



(i) Configuration A



(i) Configuration B

(a) Parametric study column cross-sections

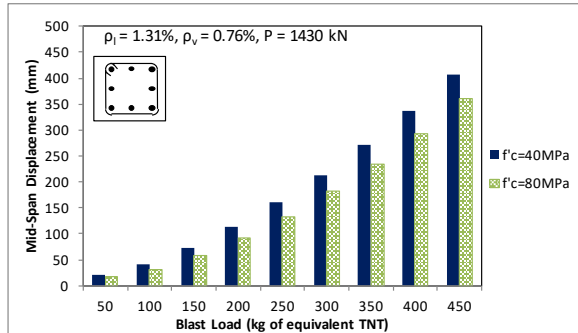
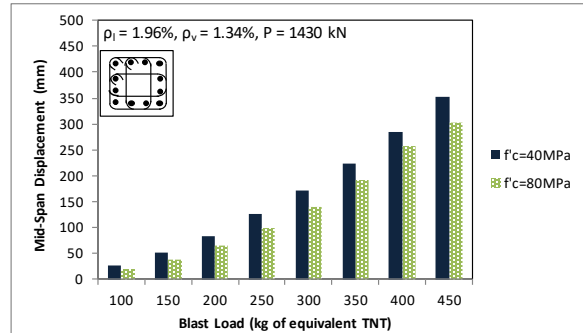
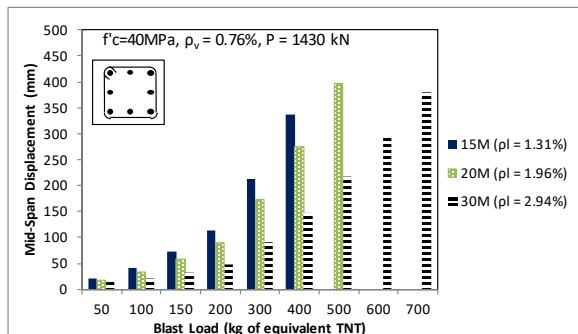
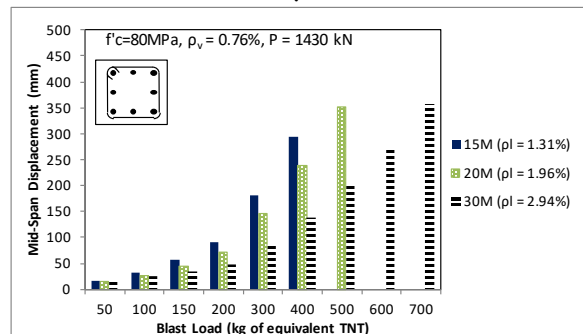
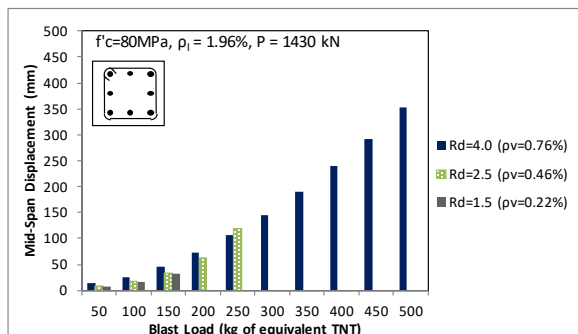
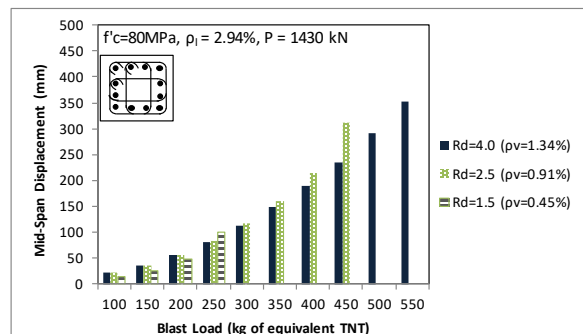

(b) Effect of concrete strength in configuration A ($R_d = 4$, 8 - 15M normal-strength bars)

(c) Effect of concrete strength in configuration B ($R_d = 4$, 12 - 15M normal-strength bars)

(d) Effect of long. steel ratio in NSC columns (Configuration A, $R_d = 4$, 15M-20M-30M bars)

(e) Effect of long. steel ratio in HSC columns (Configuration A, $R_d = 4$, 15M-20M-30M bars)

(f) Effect of transverse steel detailing ($R_d = 1.5$ to 4) in HSC columns (Configuration A, 8-20M bars)

(g) Effect of transverse steel detailing ($R_d = 1.5$ to 4) in HSC columns (Configuration B, 12-20M bars)

Fig. 14 – Parametric study column configurations and analysis results

Note: 1 mm = 0.0394 in; 1 kg = 2.2 lbs.

Combined use of UHPC and High-Performance Steel to Improve the Blast Performance of Columns with Square Cross-section

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Synopsis: This paper presents the results from tests examining the blast performance of columns constructed with ultra-high-performance concrete (UHPC) and high-performance reinforcement (high-strength steel or stainless steel). As part of the study six columns with square cross-sections were tested under simulated blast loads using a shock-tube at the University of Ottawa. Parameters investigated include the effects of concrete type, longitudinal reinforcement type and longitudinal reinforcement ratio. The results demonstrate that the use of UHPC increases the blast performance of reinforced concrete columns by increasing blast capacity and improving control of maximum and residual mid-span displacements by an average of 30% and 40%. Substitution of normal-strength bars with high-strength or stainless steel bars in the UHPC columns resulted in further reductions in displacements, which ranged between 18-43% for maximum deformations and 38-66% for residual deformations. The failure mode of all columns with low steel ratio of 1.24% (4 – No.3 bars) was tension bar rupture, regardless of steel type. Increasing the steel ratio from 1.24% to 1.84% (6 – No.3 bars) increased blast capacity and delayed failure. The use of increased amount of stainless steel bars was particularly effective, and transformed the failure mode from bar rupture to fiber pullout. The analytical study confirms that dynamic inelastic SDOF analysis can be used to reasonably predict the blast response of UHPC columns reinforced with varying steel types.

Keywords: UHPC, High-strength steel, Stainless steel, Columns, Blast, Shock Tube.

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INTRODUCTION

Recent events have highlighted the need for novel solutions to improve the blast resilience of reinforced concrete infrastructure. In the case of buildings and bridges, columns are critical elements that must be designed with sufficient strength and ductility to prevent failure and structural collapse. The use of novel construction materials such as ultra-high performance concrete (UHPC) and high-performance reinforcement can potentially enhance the blast behavior of such columns.

Compared to conventional concrete, UHPC shows superior properties which include increased compressive strength, tensile resistance, toughness and energy-absorption capacity ¹. These properties make UHPC ideal for the design of blast-resistant structures ². A promising applications of UHPC is in bridge columns, where its use can allow for more efficient designs and more durable structures. Recently the potential of combining UHPC with high-strength reinforcement in columns has also been investigated as a viable solution for the design of high-performance bridge structures ^{3,4}.

Recently, a number of experimental studies have been conducted to examine the blast behavior of UHPC elements such as slabs and columns ⁵⁻¹⁶. These studies demonstrate the superior performance of UHPC when compared to conventional concrete in terms of increased blast resistance and ability to control damage (spalling, breaching, etc.). The blast resistance of UHPC is affected by various factors, and therefore previous studies have examined the effects of such parameters as cementitious matrix strength, fiber properties (type, shape, strength) and fiber volume fraction. However, previous studies have not systematically examined the effect of reinforcement type on blast performance. Given the increased interest in the use of UHPC and high-strength reinforcement in buildings and bridges there exists a need for research on the blast behavior of structures built with these materials.

Accordingly, this research aims at examining the benefits that can be gained from the combined use of UHPC and high-performance reinforcement in building and bridge columns subjected to blast. As part of the study, six columns built with varying combinations of concrete (UHPC and conventional concrete) and steel reinforcement (conventional steel, high-strength steel and highly-ductile stainless steel) are tested under simulated blast loads using the University of Ottawa Shock-tube. As part of the analytical study the dynamic behavior of the columns is predicted using dynamic inelastic single-degree of freedom (SDOF) analysis.

REVIEW OF PREVIOUS BLAST RESEARCH OF UHPC COLUMNS

The blast behavior of UHPC structures has been studied by several researchers. Cavil et al. ⁵ conducted an early series of explosive trials on seven UHPC panels subjected to the equivalent of 6 tons of TNT at standoff distances of 30-50 m (100-164 ft) and showed that the use of UHPC resulted in high energy absorption capacity, reduced damage and better control of deformations compared to normal-strength concrete panels. Barnett et al. ⁶ presented another series of UHPC panel experiments tested under 100 kg (220 lb) of TNT and varying standoffs of 7-12 m (23-40 ft) and showed that the panels were able to sustain large blasts with small residual deformations. Similar findings related to the benefits of UHPC in panels subjected to near-field blasts have been reported by Yi et al. ⁷ (two-way slabs), Wu et al. ⁸ and Li et al. ⁹ (one-way slabs) and Li et al. ¹⁰ (one-way slabs under contact explosions). Limited studies have examined the response of UHPC panels under far-field type blasts. Ellis et al. ¹¹ studied the response of a series unreinforced UHPC slabs using a shock-tube and using finite element modelling. This study found that parameters such as fiber geometry, fiber packing and fiber content, are critical to enhancing the blast response of such unreinforced panels.

Compared to slabs and panels, limited studies have focused on the blast behavior of UHPC columns. Astarioglu and Krauthammer ¹² numerically studied the blast response of NSC and UHPC columns having varying boundary conditions and showed that the use of UHPC significantly increases the blast capacity (failure impulse) of columns.

Li et al. ¹¹ and Xu et al. ^{13,14} conducted field tests on 2.5 m × 0.2 m × 0.2 m (98 in. × 8 in. × 8 in.) columns under 1-35 kg (2.2-77 lb) of TNT equivalent at a stand-off 1.5 m (60 in.) and found that the use of UHPC columns had improved response in terms of displacement control and damage resistance when compared to high-strength reinforced concrete (HSRC). In another series of tests by the same group Li et al. ¹⁵ showed that UHPC columns had significantly improved post-blast residual capacity when compared to HSRC columns. The columns in this study were reinforced with very high-strength longitudinal bars ($f_y = 1350$ MPa [196 ksi]), however the effect of steel type was not specifically investigated.

Aoude et al.¹⁶ conducted one of the only studies on UHPC columns subjected to far-field blasts. The columns had dimensions of $2.4 \text{ m} \times 0.15 \text{ m} \times 0.15 \text{ m}$ (95 in. $\times$ 6 in. $\times$ 6 in.) and were reinforced with normal-strength bars, with blast loads applied using a shock-tube. This study found that UHPC improved column response by reducing displacements, increasing blast capacity and enhancing damage resistance when comparing to conventional concrete. However, failure of the columns was associated with rupture of the normal-strength reinforcing bars. The study recommended the use of high-performance reinforcement to improve the performance of such columns.

RESEARCH SIGNIFICANCE

Interest in the use of UHPC in building and bridge construction is growing. One of the applications of UHPC is in bridge columns where its use can allow for more efficient designs, especially when combined with high-strength reinforcement. Previous studies demonstrate the improved blast behavior of UHPC when compared to conventional concrete. However, most previous tests have focused on elements subjected to close-in or near-field blasts, with limited tests conducted on columns under far-field blasts. The effect of steel type has also not been assessed in previous studies. Given the increased interest in the use of UHPC and high-strength steels in buildings and bridges, there is need to study the blast behavior of columns built with these materials.

Accordingly, this paper provides insights into the performance enhancements that can be realized from the combined use of UHPC and high-strength or highly-ductile reinforcement in columns subjected to blasts in the far-range. Six columns are tested under shock-tube induced blast loads to examine the effect of concrete and steel type on blast resistance. The analytical study examines the ability of SDOF analysis to predict the dynamic response of UHPC columns reinforced with varying steel types.

EXPERIMENTAL INVESTIGATION

Description of test specimens

A total of six half-scale columns were tested under simulated blast loading using the University of Ottawa Shock-tube. The test program included one control column constructed with plain self-consolidating concrete (SCC) and five columns constructed with compacted reinforced composite (CRC), a proprietary ultra-high performance concrete (UHPC). To examine the effect of steel type, the UHPC specimens were built with different types of reinforcement, including: normal-strength steel, high-strength steel (HS) and stainless steel (SS).

Table 1 summarizes the properties of the various columns tested in this study. As shown in **Fig. 1**, all columns had a length of 2.468 m (8 ft) and had a 152 mm $\times$ 152 mm (6 in. $\times$ 6 in.) cross-section. It is noted that the column dimensions were selected based on the shock-tube limitations (end-frame opening, available pressure/impulse) and to allow for comparison with previous tests conducted by the authors. Longitudinal reinforcement consisted of either 4 – No. 3 or 6 – No. 3 reinforcing bars (bar diameter = 9.53 mm [0.375 in.] and bar area = 71.3 mm² [0.11 in²]), resulting in reinforcement ratios of 1.23% and 1.84%, respectively. The control SCC column was built with 4 – No. 3 normal-strength bars, while the steel type and ratio was varied in the UHPC columns. Transverse reinforcement consisted of 6.3 mm (0.25 in.) diameter hoops made of non-deformed steel wire, which were spaced at $s = 75 \text{ mm}$ (3 in.), corresponding to the requirements for “moderately ductile” columns in the CSA A23.3 Standard¹⁷. This spacing was kept constant over the entire column span as shown in **Fig. 1**. This detailing is also consistent with bridge design practice, keeping in mind that the columns were tested under idealized lab conditions.

Specimen nomenclature in **Table 1** reflects the three variables in the experimental program, namely: the concrete type (SCC vs. UHPC), longitudinal steel ratio (4 #3 or 6 #3 bars), and steel type (normal-strength, HS or SS bars). For example, column UHPC-6#3-HS was built with UHPC and 6 –No.3 high-strength bars, while SCC-4#3 contains plain SCC and 4 – No. 3 normal strength bars.

Material properties

Table 2 lists the properties of the pre-packaged SCC mix used in this study. The mix had a specified strength of 40 MPa (5.8 ksi), with a maximum aggregate size of 10 mm (0.4 in.), a sand to total aggregate ratio of 0.55, and was mixed using a water-cement ratio of 0.42. A super-plasticizer and viscosity modifying admixture (VMA) were incorporated into the blend by the manufacturer in the form of dry powder.

The UHPC specimens were constructed with compact reinforced composite (CRC) with a specified strength of 140 MPa (20.3 ksi). **Table 2** lists some of the properties of the UHPC mix as specified by the manufacturer. The high compressive strength of this UHPC is achieved through use of a low water-binder ratio (0.18 in this study), micro-silica and omission of coarse aggregates. The mix was reinforced with smooth steel fibers at a ratio of 2% by volume of UHPC (153.6 kg/m³ [9.6 lb/ft³]). These fibers had a length of 13 mm (0.51 in.), a diameter of 0.21 mm (0.08 in.) and a tensile strength of 2750 MPa (400 ksi). In all cases the concrete was mixed using a 0.225 m³ (8 ft³) capacity forced-action pan mixer.

Each column was built with one batch of concrete. The average properties of each batch in terms of compressive strength were obtained by testing 100 mm × 200 mm (4in. × 8 in.) cylinders, and are summarized in **Table 1**. **Fig. 2a** shows typical stress-strain curves for the plain SCC and UHPC. Three types of No.3 bars were used as longitudinal reinforcement, including: (1) conventional Grade 400 MPa (60 ksi) bars, (2) ASTM A1035 Grade 690 MPa (100 ksi) high-strength (HS) bars and (3) XM-28 Grade 520 MPa (75 ksi) austenitic stainless steel (SS) bars. The average tensile properties of the three steel types are summarized in **Table 3**, while typical stress-strain curves are shown in **Fig. 2b**. The steel used to construct the column ties had a yield strength of 617 MPa (89 ksi).

University of Ottawa Shock Tube

The University of Ottawa Blast Research Laboratory is equipped with a shock-tube that can simulate the shockwaves generated by the hemispherical free air surface burst of high explosives¹⁸. As shown in **Fig. 3a**, the shock tube consists of four main components, including: (1) a variable length driver section which generates the shockwave energy using compressed air, (2) a spool section which controls the firing of the shockwave using a differential pressure diaphragm, (3) and an expansion section which ends with (4) a 2 m × 2 m (80 in. × 80 in.) rigid end test frame. For non-planar elements such as columns, a load transfer device (LTD) is used to collect and transfer the pressure at the end-frame opening onto the specimens. Varying shockwave parameters (pressure, duration and impulse) can be obtained by adjusting the shock-tube driver length and driver pressure.

Test setup and procedure

Fig. 3d-e show a typical column prior to testing. Additional photos showing the load-transfer device and supports are shown in **Fig. 3b-c**. All columns were tested under combined axial and blast loads. For all tests the axial load, which was applied using a hydraulic jack, was 294 kN (66 kip), equivalent to 40% of the pure axial load capacity of the control SCC specimen. In this study the LTD consisted of a light gauge steel metal sheet connected to a series of steel beams which resulted in the transfer of the blast pressure as a uniformly distributed load along the tension face of the columns. The clear span of the columns between supports was 1980 mm (6.5 ft), with the columns connected to the shock-tube frame using partially-fixed supports. Complete displacement-time histories were recorded using two linear variable differential transducers (LVDT) placed at mid-height and at 1/3rd distance along the clear span as shown in **Fig. 3e**. Pressure sensors near the load transfer device were used to record complete reflected pressure time histories for each test. A high speed camera also recorded the response of the columns response at a frame rate of 500 frames per second.

Each column was tested with varying shock tube induced blast waves to test the specimens under yield and ultimate conditions. Blast 1 was meant to bring the control columns to yielding, while Blasts 2 and 3 were intended to damage and fail the control SCC and UHPC columns, respectively. It is also noted that the blast pressures were selected to allow for comparison with the previous tests conducted by the authors¹⁶. Sample shockwaves corresponding to Blasts 1 to 3 are shown in **Fig. 4a**. In this study, the driver length was kept constant at 2743 mm (9 ft) while the driver pressures varied. The following driver pressures were selected for Blasts 1-3, respectively: 35 psi (207 kPa), 80 psi (552 kPa), and 100 psi (689 kPa). The resulting average reflected pressure (P_r), positive phase duration (t_d), and reflected impulse (I_r) are summarized in **Table 4**. To correlate the tests to actual blast conditions, the scaled distance (Z) for each blast was estimated using the procedure outlined in the Annex A of the CSA S850-12 Blast Standard. For example, Blast 2 ($I_r = 812 \text{ kPa}\cdot\text{msec}$, $P_r = 88 \text{ kPa}$) corresponds to a scaled distance of $5.36 \text{ m/kg}^{1/3}$, which in turn corresponds to the detonation of 416 kg of TNT at 40 m.

EXPERIMENTAL RESULTS

Table 5 summarizes the experimental results for the columns in terms of measured shockwave parameters (reflected pressure, positive phase duration, and reflected impulse) and column response (maximum displacement and residual displacement) for Blasts 1-3 (see **Fig. 4b** for a typical response history). Blast 1 was intended to bring the longitudinal steel in the control column close to yielding, while Blast 2 and 3 were intended to damage and fail the columns. A summary of the test results for each column is provided in the following sub-sections.

Response of SCC-4#3

Column SCC-4-#3 was a control column built with conventional SCC concrete and 4 – No.3 normal-strength bars. Blast 1 (35 psi) was meant to bring the normal-strength longitudinal steel bars close to yielding. This shot resulted in some cracking at mid-span, with a maximum displacement ($d_{\max}$) of 33.7 mm (1.3 in.), and residual displacement (d_{res}) of 9.2 mm (0.36 in.). Failure of this column occurred at Blast 2 (80 psi), and resulted in crushing of concrete on the compression side, the formation of diagonal cracks at mid-span, significant spalling of cover concrete and buckling of the longitudinal reinforcement (see **Figs. 5a** and **7a**). Using the high-speed camera, the maximum displacement was determined to be 139 mm (5.5 in.) during this blast. High-speed video shows that this blast resulted in the formation of significant secondary blast fragments at failure.

Response of UHPC-4#3

Column UHPC-4#3 was the first of five UHPC specimens tested in this experimental program. This column was constructed with UHPC having 2% steel fibers and reinforced with 4 – No.3 normal-strength longitudinal bars ($\rho = 1.23\%$). The column experienced no observable damage and lower displacements to those of the control column after Blast 1 (35 psi) ($d_{\max} = 25.1$ mm (1 in.) and $d_{\text{res}} = 6.3$ mm (0.25 in.)). Unlike column SCC-4#3, this column did not fail under Blast 2 (80 psi) as the UHPC and fibers limited damage at midspan. The maximum and residual displacements after this blast were 97.2 mm (3.8 in.) and 67.6 mm (2.7 in.). Although this blast resulted in significant cracks and the formation of a plastic hinge at mid-span, damage remained well controlled, with no crushing or cover spalling (see **Fig. 5b**). This shot also resulted in the formation of a large tensile crack at midspan which reached 6.8 mm (0.27 in.) in width. The failure mechanism of this column was tension rebar rupture after Blast 3 (100 psi) (see **Figs. 6a** and **7b**). It was observed that the steel fibers pulled out at the single critical crack at failure. Converse to the control column, high-speed video data shows no discernible secondary fragments after Blast 3, despite the severe failure.

Response of UHPC-4#3-HS

Column UHPC-4#3-HS was the first of two UHPC specimens reinforced with high-strength (HS) bars. With the exception of the HS bars, this column had identical properties to those of the previous specimen (UHPC, 2% fibers, 4 – No.3 bars, $\rho = 1.23\%$). This column experienced no observable damage and reduced displacements compared to column UHPC-4#3 after Blast 1 (35 psi): $d_{\max} = 19.1$ mm (0.75 in.) and $d_{\text{res}} = 3.6$ mm (0.14 in.). The maximum and residual displacements after Blast 2 (80 psi) were 57.8 mm (2.3 in.) and 22.7 mm (0.9 in.), which are reduced when compared to the previous specimen. Although tensile cracks were observed at mid-span and at the hoop locations, damage was relatively minor after this blast (see **Fig. 5c**). Despite the use of high-strength bars, the failure mechanism of this column was tension rebar rupture after Blast 3 (100 psi) as shown in **Figs. 6b** and **7c**. Significant damage was also observed at the column ends. High-speed video data shows no discernible secondary fragments despite the brittle failure. Once again fiber pullout occurred at the critical crack location at mid-span.

Response of UHPC-6#3-HS

This column was built with UHPC and 6-No.3 high-strength bars, resulting in an increased longitudinal steel ratio of $\rho = 1.84\%$ when compared to the previous specimen. Behavior of this column after Blasts 1 and 2 was similar to those of the previous two columns in terms of damage, however the column shows reduced mid-span displacements and support rotations (see **Table 5** and **Fig. 5d**). The increase in high-strength steel ratio resulted in a more controlled failure at Blast 3 (100 psi). The maximum and residual displacements were 145 mm (5.7 in.) and 58.4 mm (2.3 in.) after this blast. While blowout failure was prevented, close inspection of the tensile reinforcement revealed that one of the tensile bars ruptured during this test, and therefore the column can be considered to have failed (see **Figs. 6c** and **7d**). As with the previous two specimens no discernable fragments formed at failure, with fiber pullout only observed at the critical crack location.

Response of UHPC-4#3-SS

Column UHPC-4#3-SS was the first of two UHPC specimens reinforced with stainless steel (SS) bars for longitudinal reinforcement. Except for the SS bars, this column had identical properties to column UHPC-4#3, with a reinforcement ratio, $\rho = 1.23\%$. Response of this column after Blasts 1 and 2 was similar to its companion. The maximum / residual displacements after Blast 1 and Blast 2 were 20.5 mm (0.8 in.) / 3.9 mm (0.15 in.), and 73.4 mm (2.9 in.) / 30.3 mm (1.2 in.), which are reduced when compared to UHPC-4#3. The column also shows a reduction in the extent of cracking and damage at mid-span after Blast 2 (see **Fig. 5e**). Only one major crack formed at midspan with a width of 5.5 mm (0.2 in.), indicating the initiation of fiber pullout. Failure of this column occurred at Blast 3 (100 psi), with fracture of the tension stainless steel bars at mid-span. In addition to the failure at mid-span, opening of tension cracks was also observed at the column support locations (see **Fig. 6d**). Necking of the rebar was visible at failure, indicating that very large strains were sustained in the steel bars before rupture (see **Fig. 7e**).

Response of UHPC-6#3-SS

This was the last column tested in this research program and was built with UHPC and 6 –No.3 stainless steel bars, resulting in an increased steel ratio of $\rho = 1.84\%$ when compared to the previous specimen. Behavior of the column after Blast 1 is similar to the previously tested UHPC specimen. The maximum and residual displacements were 58.6 mm (1.9 in.) and 18.1 mm (0.3 in.) after Blast 2, which are reduced when compared to column UHPC-4#3-SS. Only fine cracking was observed after this blast (see **Fig. 5f**). Unlike all other specimens, the use of increased stainless steel ratio combined with ductile stainless steel bars was sufficient to prevent failure under Blast 3 (100 psi). The column sustained a maximum displacement of 102 mm (4 in.), with a residual

displacement of 71 mm (2.8 in.) during this blast. Although this blast resulted in important damage and the formation of a critical crack at mid-span, bar fracture was prevented due to the increased ductility of the stainless steel bars (see **Fig. 6e**). High speed video shows that the crack near mid-span opened significantly during this test. Since the maximum shock-tube driver pressure was reached, the column was subjected to another shock wave load corresponding to Blast 3. The second application of Blast 3 loading caused the critical crack at mid-span to open further to 20 mm (0.8 in.), however the tensile steel reinforcement did not rupture, resulting in a ductile failure (see **Fig. 7g**).

Effect of parameters on midspan displacements

In this section the results from Blasts 1, 2 and 3 in **Table 5** and **Fig. 8** are compared to investigate the effect of the various test parameters on maximum and residual displacements, blast capacity and failure mode.

1. Effect of UHPC — When compared to conventional concrete, UHPC has significantly higher compressive strength, as well as enhanced tensile resistance and toughness due to the provision of steel fibers. **Fig. 8a** compares the displacements sustained by specimens SCC-4#3 and UHPC-4#3 at Blast 2, while data at Blast 1 can be compared in **Table 5**. Both columns had identical properties except for concrete type. It can be seen that UHPC significantly improved blast performance by reducing maximum and residual displacements at mid-span. When compared to column SCC-4#3, maximum displacements for column UHPC-4#3 were reduced by 26% and 30% at Blasts 1 and 2, respectively. Similarly, residual displacements were reduced by 32% and 49%, at these same two blasts. While failure of the SCC column was associated with significant concrete crushing, cover spalling and bar buckling at Blast 2, damage remained well controlled in the UHPC companion, with failure delayed to Blast 3 (see **Fig. 5**). These results demonstrate the improved blast resistance capabilities of UHPC when compared to normal-strength concrete.

2. Effect of high-strength reinforcement — The effect of high-strength reinforcement is highlighted by comparing the responses of columns UHPC-4#3 vs. UHPC-4#3-HS at Blasts 1 and 2 in **Table 5** and **Fig. 8a**. Both UHPC columns had identical properties except for the steel type (conventional vs. HS bars, respectively). When compared to UHPC-4#3, the maximum (d_{max}) and residual (d_{res}) displacements for the column with high-strength bars were decreased by 24% and 43% under Blast 1. Similarly, column UHPC-4#3-HS shows significant decreases of 41% and 66% in d_{max} and d_{res} at Blast 2. Furthermore, the use of high-strength bars resulted in reduced amounts of cracking and damage at midspan when compared to normal-strength bars (compare midspan damage in **Fig. 5**) at this shot. However, the use of HS bars did not prevent bar rupture at Blast 3 due to the relatively low reinforcement ratio used in these columns (**Fig. 6**).

3. Effect of stainless steel reinforcement — The effect of stainless steel reinforcement is examined by comparing columns UHPC-4#3 and UHPC-4#3-SS which were built with 4 – No.3 normal-strength vs. stainless steel bars, respectively. As shown in **Table 5** and **Fig. 8b** the use of stainless bars resulted in a better control of mid-span displacements at Blasts 1 and 2. Comparing to UHPC-4#3, column UHPC-4#3-SS shows reductions of 18% and 24% in maximum displacements at these blasts. Similarly, residual displacements are reduced by 38% and 55% at Blasts 1 and 2 for the column with higher-strength stainless reinforcement. The amount of cracking at mid-span at Blast 2 was also noticeably reduced for column UHPC-4#3-SS (see **Fig. 5c**). Due to the relatively low steel ratio, both columns failed at Blast 3 with rupture of the tension steel bars as shown in **Fig. 6**.

4. Effect of increased HS and SS steel ratio — Increasing the longitudinal steel ratio from $\rho = 1.23\%$ (4-No.3 bars) to $\rho = 1.84\%$ (6-No.3 bars) resulted in improvements in the blast performance of the UHPC columns. The effect of this parameter is investigated in two pairs of specimens: UHPC-4#3-HS vs. UHPC-6#3-HS, which had high-strength reinforcement, and UHPC-4#3-SS vs. UHPC-6#3-SS, which had stainless steel bars. Beginning with the high-strength (HS) set, it can be seen that column UHPC-6#3-HS shows reductions of 31% and 81% in maximum and residual displacement when compared to column UHPC-4#3-HS at Blast 1 (see **Table 5**), with corresponding reductions of 45% and 75% at Blast 2 (see **Fig. 8a**). Both columns fail at Blast 3 due to rupture of the tension bars, however the failure is significantly better controlled in column UHPC-6#3-HS which experiences lower displacement at failure (see **Fig. 6c**). Blowout failure is prevented with only one of the three tension steel bars ruptured in column UHPC-6#3-HS (see **Fig. 7d**).

The effect of stainless steel ratio is examined by comparing the responses of columns UHPC-6#3-SS and UHPC-4#3-SS. At Blast 1, maximum and residual displacements are reduced by 11% and 27% for the column with 6-No.3 stainless bars (see **Table 5**). Similarly, At Blast 2, increasing the steel ratio resulted in reductions of 40% and 73% in d_{max} and d_{res} displacements (see **Fig. 8b**). The benefit of increased steel ratio is further confirmed at Blast 3, where the increased reinforcement ratio prevents bar rupture and failure. An additional blast load corresponding to Blast 3 was required to produce failure in column UHPC-6#3-SS. Failure was associated with fiber pullout rather than tension steel rupture, resulting in a more controlled and ductile failure mode (see **Fig. 6**).

5. Effect of high performance steel type — The effect of high-performance steel type is highlighted by studying the responses of columns UHPC-4#3-HS vs. UHPC-4#3-SS, and columns UHPC-6#3-HS vs. UHPC-6#3-SS, which were designed with equal amounts of high-strength and stainless steel bars. Referring back to **Fig. 2b** it can be seen that both steel types show increased strength when compared to the NS bars, with the HS bars showing higher ultimate resistance but reduced ultimate strain capacity when compared to the SS bars. **Fig. 8c** compares the results in the first set (4-No.3) at Blast 2, where the use of higher-strength HS bars results in reductions of 21% & 25% in maximum and residual displacements. Both columns in this set failed at Blast 3 due to tension bar rupture given the low steel ratio.

The effect of steel type in the 6-No.3 set is studied in **Fig. 9d-e**. At Blasts 1 and 2, column UHPC-6#3-HS shows slight reductions of 22% and 8% in maximum displacement due to the increased strength of the ASTM A1035 bars (the column with HS bars also shows reduced residual displacements at these blasts – see **Table 5** and **Fig. 8d**). However, the use of more ductile stainless steel bars results in an improved response at Blast 3 (see **Fig. 8e**). In addition to the reduction in maximum displacement (102 vs. 145 mm (4.0 vs. 5.7 in.)), column UHPC-6#3-SS prevents bar rupture and failure at this blast when compared to column UHPC-6#3-HS, which suffered rupture of one of its tension bars (see **Fig. 6**). In general, the results point to the excellent compatibility of UHPC and ductile stainless steel reinforcement which can be used to prevent brittle failures due to tension bar rupture – further research is recommended.

ANALYTICAL INVESTIGATION

Single-degree-of-freedom (SDOF) analysis is commonly used in the blast analysis of reinforced concrete structures. In the current study the ability of dynamic inelastic SDOF analysis to predict the maximum displacement response of UHPC columns having varying steel types was investigated.

Material models

The first step in the analysis was to define the constitutive models for concrete, UHPC and the various steel types at high-strain rates. **Table 6**, **Table 7** and **Fig. 9** provides an overview of the models considered in this study.

For the SCC column, the model proposed by Légeron and Paultre¹⁹ was used for concrete in compression. This model accounts for important parameters affecting confined concrete response including the configuration, spacing and yield strength of the transverse reinforcement. Confinement is modelled using a "confinement index" ($I_E = f_{le}/f'_{co}$), where the effective confining pressure (f_{le}) is calculated using the nominal confining pressure provided by the steel ties (f_l), and an "effective confinement coefficient" (K_e) which accounts for the arching action which occurs between the levels of the transverse steel in rectangular columns:

$$f_{le} = K_e f_l \quad (1)$$

$$K_e = \frac{\left(1 - \frac{\sum w_i^2}{6c_x c_y}\right) \left(1 - \frac{s'}{2c_x}\right) \left(1 - \frac{s'}{2c_y}\right)}{1 - \rho_c} \quad (2)$$

In the above equation, s' , c_x , c_y , $\sum w_i^2$ and ρ_c represent the clear spacing between ties, the width of the concrete core in the x and y directions, the sum of the squares of the clear spacing between adjacent longitudinal bars and the ratio of longitudinal steel in the core region, respectively. As shown in **Table 6**, the stress-strain curve is defined using the point of peak confined strain and stress (ϵ'_{cc} , f'_{cc}) and the strain at 50% drop in peak confined stress (ϵ_{cc50} , $0.5f'_{cc}$), with an exponential descending branch.

For the UHPC columns, the model proposed by Aoude²⁰ was used for confined UHPC in compression. This model modifies the Légeron and Paultre¹⁹ model for the additional confinement provided by steel fibers and has been shown to predict the compression response of UHPC columns with good accuracy²¹. According to this model the additional fiber confining pressure (f_{lf}) is obtained by multiplying the number of fibers per unit area (N_f) by the average fiber pullout strength ($F_{pullout}$):

$$f_{lf} = N_f \times F_{pullout} = \alpha v_f \frac{l_f}{d_f} \tau_{bond} \quad (3)$$

where α = fiber orientation factor (taken as 3/8), v_f , l_f , d_f = volume fraction, length and diameter of the fibers, respectively; and τ_{bond} = matrix bond strength, assumed to be equal to two times the tensile strength of concrete ($\tau_{bond} = 0.6 \times f'_{co}^{2/3}$). **Table 6** summarizes the equations for peak stress/strain, and the ascending/descending branches of the stress-strain curve. Consistent with the Légeron and Paultre¹⁹ model, the stress-strain curve is defined using the point of peak confined strain and stress (ϵ'_{cc} , f'_{cc}) and the strain at 50% drop in peak confined stress (ϵ_{cc50} , $0.5f'_{cc}$). The model assumes that fibers and transverse reinforcement play a hybrid role in improving the post-peak behavior of the core concrete, with fibers contributing more significantly to ductility in nominally

confined columns, and to a lesser extent in well confined columns. To account for this effect, strain ϵ_{cc50} is modified as follows in the model (where ϵ_{co50} = strain at 50% f'_{co}):

$$\epsilon_{cc50} = \epsilon_{co50} \left[1 + 60 \left(\frac{f_{fe}}{f'_{co}} + \left(\frac{f_{ff}}{f'_{co}} \right) (1 - K_e) \right) \right] \quad (4)$$

The simplified stress-strain curve presented in **Fig. 10a** was used for unconfined UHPC in compression²¹. The model consists of a linear ascending branch having a slope equivalent to the modulus of elasticity of UHPC (taken as $E_c = 50$ GPa) and a maximum stress equivalent to the peak unconfined stress of UHPC (f'_{co}). Hosinieh et al.²¹ showed that this model closely approximates the stress-strain behavior obtained from UHPC cylinder tests.

Nielsen²² has shown that provision of fibers in UHPC allows the material to show important post-cracking resistance and strain-hardening behavior in tension. The model shown in **Fig. 9b** was used for the tension portion of the UHPC stress-strain curve²³. In this model, the pre-cracking portion of the curve is linear having a slope equivalent to the modulus of elasticity of UHPC and a maximum stress equivalent to the cracking stress of UHPC (taken as $f_{ct} = 0.3f'_{co}{}^{2/3}$). The post cracking branch of the curve is a bilinear relationship, with the first portion expressed as linear from the point of cracking to a stress $f_2^* = \frac{1}{2}v_f\tau_{bond}\frac{l_f}{d_f}$ and strain $\epsilon_2^* = \tau_{bond}\frac{l_f}{d_f}\frac{1}{E_{fp}}$, where $E_{fp} = 200$ GPa and v_f , l_f , d_f and τ_{bond} are as defined previously. The second portion of the post-cracking branch is linearly descending from the stress f_2^* to tensile stress failure at a strain of 0.02 mm/mm²³.

The responses of the normal-strength and high-strength reinforcements were modeled using the relationships proposed by Jacques et al.²⁴ and in ACI ITG-6R²⁵, respectively. As shown in **Fig. 9c**, the Jacques et al.²⁴ model consists of two linear segments for elastic-plastic response and a parabolic function for strain-hardening. The three-part function proposed by the ACI ITG guide and shown in **Fig. 9d** provides a lower bound of the actual stress-strain response of Grade 690 MPa ASTM A1035 reinforcement²⁵. No model was available for the type of stainless steel used in this study, therefore the response of the stainless steel bars was modeled using data from the coupon tests.

With the material properties defined, dynamic increase factors (DIF) of 1.24 and 1.30 (1.1 after ultimate) were applied to the normal-strength concrete and steel models, respectively, to account for the effect of high-strain loading. These factors were determined using the CEB²⁶ and Saatcioglu et al.²⁷ models and assuming a strain-rate of 1 s^{-1} in the shock-tube experiments. Dynamic increase factors of 1.17 and 1.14 were applied to the UHPC in compression and tension, based on the models proposed by Zhang and Mindess²⁸ and Malvar and Ross²⁹, respectively. A description of the DIF models can be found in **Table 7**. A dynamic increase factor of 1.0 was conservatively applied to the HS and SS reinforcement based on data which suggests reduced DIF for higher grade steel reinforcement³⁰.

SDOF analysis

After defining the constitutive material models, the dynamic response of the columns was predicted using software *RCBlast* developed by Jacques³¹. Analysis using *RCBlast* begins with defining the cross-sectional properties of the columns and constitutive models for the concrete and steel reinforcement as described above. Using this data, the program then computes the moment-curvature relationship of the column section using sectional analysis. With the moment-curvature relationship and a user-defined plastic hinge length, the resistance functions are then computed using an elastic beam with non-linear rotational hinge analogy as defined by Jacques et al.²⁴ and previously described by Aoude et al.¹⁶ (**Fig. 10**). The plastic hinge length at midspan and at the supports was taken to be equal to 135 mm (5.3 in.), which corresponds to the distance between the extreme compression fiber and the tensile steel reinforcement. During testing the columns experience a reduction in axial load during maximum response¹⁶. To account for variable axial load, composite resistance functions were determined using 6 axial load steps from 0 to 294 kN (66 kips) using an available function in *RCBlast*. The load-deformation characteristics generated by the program and an idealized triangular blast load were then used to conduct the SDOF analysis by solving the equation of motion shown below:

$$K_{LM}\ddot{u}(t) + R(u(t)) = AP_r(t) \quad (5)$$

where m is the mass of the system, $\ddot{u}(t)$ and $u(t)$ are the acceleration and displacement at column mid-height, K_{LM} is the load-mass factor used to transform the structure into an equivalent SDOF system, and $R(u(t))$ is the column resistance as a function of displacement. In the analysis, m was taken to be 315 kg, equivalent to the total mass of the column and load-transfer device. The right side of the equation represents the loading function, where $P_r(t)$ is the time-variant blast pressure and A is the loaded area (shock-tube end-frame opening: 4.12 m^2 [44 ft^2]).

Results

Fig. 11 and **Table 8** compare the predicted results to the experimental maximum displacements (d_{exp} and d_{anls}) for all columns at Blasts 1 and 2 (results for column CRC-6#3-SS at Blast 3 are also included since these shots did not cause bar rupture in this specimen). Considering all columns, the average d_{anls}/d_{exp} ratios is found to be 1.03 with an average absolute error of 13%, and a coefficient of variation of 16%. Considering the control SCC and UHPC columns, the results are well predicted with predicted-to experimental displacement ratios of 1.03 and 0.97 at Blast 2 (errors of $\sim 3\%$). Relatively larger errors are observed at Blast 1 due to the low displacements recorded at this shot. Considering the high-strength and stainless steel UHPC columns it can be observed that the d_{anls}/d_{exp} ratios are generally greater than 1.0, which indicates that the analysis over-predicted the displacements in most cases. This can be explained by the fact that DIF for the HS and SS reinforcement were conservatively taken as 1.0 for these specimens. Further research is recommended to develop dynamic increase factors which can be used in the analysis of columns reinforced with HS and SS bars.

SUMMARY, FINDINGS AND CONCLUSIONS

This paper presented the experimental results from six conventional concrete and UHPC columns tested under simulated blast loading. The experiments examined the effect of UHPC, steel type and steel reinforcement ratio on blast response. Based on the results, the following conclusions can be drawn from this study:

1. The results confirm that the use of UHPC improves the blast response of reinforced concrete columns. In addition to reducing maximum and residual displacements at equivalent blasts, the use of UHPC increased blast resistance and resulted in enhanced damage control prior to failure;
2. Substitution of normal-strength bars with high-strength (HS) or stainless steel (SS) bars in the UHPC columns resulted in further enhancements in performance, with better control of maximum and residual displacements at equivalent blasts;
3. The failure mode for all UHPC columns with 4 –No.3 bars ($\rho = 1.24\%$) was tension rebar rupture, regardless of steel type. This mode of failure may be linked to the high compressive strength and bond capacity of UHPC and the relatively low steel ratio in these columns. In general, it is recommended that higher steel ratios be used when designing UHPC columns for blast resistance;
4. Increasing the HS or SS ratio (6 – No.3 bars, $\rho = 1.84\%$) further improved the blast performance of the UHPC columns, and prevented brittle failure. The use of increased amounts of highly ductile stainless steel bars was particularly effective, and transformed the failure mode from bar rupture to fiber pullout in the UHPC. Further research examining the benefits of combining UHPC and high-performance reinforcement is recommended;
5. SDOF analysis can be used to predict the blast response of UHPC columns designed with varying steel types with reasonable accuracy, however further research is needed to better simulate the high strain-rate response of high-strength and stainless steel reinforcement.

The columns in this study had relatively small square cross-sections and were tested under idealized conditions using a shock-tube. Additional research on larger-scale columns with varied cross-sections (e.g. circular) and tested under live explosive loading and varied scaled distances is recommended. Given the increased interest in using UHPC and high-strength reinforcement in bridges additional research on columns and bridge piers/bents reflective of bridge construction is also recommended. Previous research indicates that the use of high-strength steel as transverse reinforcement improves confinement in columns. Further research on the benefit of implementing high-strength transverse reinforcement in columns subjected to blast loads warrants further study. Finally, different high-strength steel bars have distinct stress-strain responses and therefore additional research on columns designed with varying high-strength steel grades is recommended.

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Table 1 – Details of column specimens

Column ID	Concrete mix	Fiber Content %	Avg. Concrete Strength MPa (ksi)	Long. Reinforcement			Transverse Reinforcement
				Configuration	Steel Type	Steel Grade MPa (ksi)	spacing mm (in.)
SCC- 4#3	SCC	0.0	48 (7)	4 – No. 3	Normal strength	400 (60)	75 (2.5)
UHPC-4#3	UHPC	2.0	--	4 – No. 3			
UHPC-4#3-HS			150(21.8)	4 – No. 3	High-strength (ASTM A1035)	690 (100)	75 (2.5)
UHPC-6#3-HS			150(21.8)	6 – No. 3			
UHPC-4#3-SS			132(19.1)	6 – No. 3	Stainless Steel (XM-28)	520 (75)	75 (2.5)
UHPC-6#3-SS			151(21.9)	6 – No. 3			

Table 2 – SCC and UHPC mixture manufacturer properties

SCC*	HSF Cement (kg/m ³)	Water-cement Ratio	Coarse Aggregate (kg/m ³)	Fine Aggregate (kg/m ³)	Ratio Fine/Total Aggregate
	500	0.42	765	915	0.55
UHPC*	Densit Binder (kg/m ³)	Water-binder Ratio	Quartz Sand (kg/m ³)		
			0–0.25 mm	0.25–1 mm	1–4 mm
	1146	0.18	236	404	809

Note: 1 kg/m³ = 0.0624 lb/ft³

*Details of manufacturer's additives are proprietary

Table 3 – Reinforcing steel properties

Rebar Type	Rebar Size	Yield		Strain Hardening	Ultimate		Rupture	
		Stress f_y MPa (ksi)	Strain ϵ_y	Strain ϵ_{sh}	Stress f_u MPa (ksi)	Strain ϵ_u	Stress f_r MPa (ksi)	Strain ϵ_r
Normal strength	No. 3	424 (61.5)	0.0024	0.0155	616 (89.3)	0.1467	522 (75.7)	0.1630
High-strength (ASTM A1035)	No. 3	950 (137.8)*	0.0070	--	1200 (174)	0.0480	1196 (173.5)	0.0570
Stainless Steel (XM-28)	No. 3	630 (91.4)*	0.0071	--	937 (135.9)	0.3571	766 (111.1)	0.3995

* yield point determined 0.2% offset method

Table 4 – Shocktube driver parameters and Average shockwave properties

Blast ID	Driver Pressure P_d kPa (Psi)	Driver Length L_d mm (ft)	Avg. Reflected Pressure P_r kPa (Psi)	Avg. Reflected Impulse I_r kPa*msec (Psi *msec)	Avg. Positive Phase Duration t_d msec	Scaled distance Z (m/kg ^{1/3})	Equivalent Charge weight (kg) ^a
Blast 1	241 (35)	2743 (9)	47.4 (7)	455 (65)	22.0	7.65	143
Blast 2	552 (80)	2743 (9)	87.9 (13)	812 (117)	23.3	5.36	416
Blast 3	690 (100)	2743 (9)	98.9 (14)	920 (131)	22.0	5.05	497

^a Assumed stand-off distance = 40 meters.

Note: 1 kg = 2.2 lb, 1 m = 3.3 ft.

Table 5 – Experimental results (shockwave properties and midspan displacements)

		Shockwave Properties ¹			Midspan Displacements ²	
		P _r	I _r	t _d	d _{max}	d _{res}
Column ID	Blast ID	(kPa)	(kPa*msec)	(msec)	(mm)	(mm)
SCC- 4#3	Blast 1	43.7	430	22.4	33.7	9.2
	Blast 2	82.7	787	23.6	139.2*	131.8*
UHPC-4#3	Blast 1	49.4	466	21.0	25.1	6.3
	Blast 2	86.7	789	22.9	97.2	67.6
	Blast 3	95.8	887	20.3	335*	335*
UHPC-4#3-HS	Blast 1	49	459	22.3	19.1	3.6
	Blast 2	88.9	834	25.7	57.8	22.7
	Blast 3	102	952	23.2	300*	297 *
UHPC-6#3-HS	Blast 1	49.2	448	23.5	17.4	1.2
	Blast 2	94.9	823	21.4	53.8	16.9
	Blast 3	104.6	1004	21.5	145*	58*
UHPC-4#3-SS	Blast 1	45.5	472	20.5	19.7	3.2
	Blast 2	82.2	810	24.7	57.0	30.0
	Blast 3	93.1	891	23.8	NO DATA **	210†
UHPC-6#3-SS	Blast 1	47.7	457	22.4	22.4	4.6
	Blast 2	92.2	829	21.5	58.6	18.1
	Blast 3	104	969	21.4	102.2	71.2
	Blast 3 (2)	93.7	812	23.0	92.4	61.3

Note: 1 kPa = 0.145 Psi, 1 mm = 0.0394 in.

¹ P_r = reflected pressure; I_r = reflected impulse, t_d = positive phase duration;

² d_{max} = maximum midspan displacement; d_{res} = residual midspan displacement

*LVDT removed; displacements determined using high speed camera

** No data available due to equipment malfunction

† Residual displacement estimated manually using measuring tape

Table 6 – Description of concrete and UHPC confinement models

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

Table 7 – Dynamic increase factor models

Concrete in compression	Concrete in tension	Normal-strength Steel in tension
CEB: $DIF_c = \begin{cases} \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^{1.026\alpha} & \text{for } \dot{\epsilon} \leq 30s^{-1} \\ \gamma \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^{1/3} & \text{for } \dot{\epsilon} > 30s^{-1} \end{cases}$ Where: $\log \gamma = 6.156\alpha - 2$; $\alpha = \frac{1}{(5+9f'_c/10)}$; $\dot{\epsilon}_s = 30 \cdot 10^{-6}s^{-1}$	Malvar and Ross: $DIF_t = \begin{cases} \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^{\delta}, & \dot{\epsilon} \leq 1s^{-1} \\ \beta^* \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^{1/3}, & \dot{\epsilon} > 1s^{-1} \end{cases}$ Where: $\log \beta^* = 6\delta - 2$; $\delta = \frac{1}{(1+8f'_c/10)}$; $\dot{\epsilon}_s = 10^{-6}s^{-1}$	Saatcioglu et al.: At yield $\rightarrow DIF_y = 0.034 \ln(\dot{\epsilon}) + 1.30 \geq 1.0$ At ultimate $\rightarrow DIF_u = 0.0101 \ln(\dot{\epsilon}) + 1.10 \geq 1.0$

Note: $\dot{\epsilon}_s$ – static strain rate; $\dot{\epsilon}$ – dynamic strain rate; α, γ, β^* and δ = various curve fitting parameters; DIF_c, DIF_t, DIF_y & DIF_u : Dynamic increase factors for concrete in compression, concrete in tension and steel at yield & ultimate.

Table 8 – Blast analysis results

Column ID	Shot #	Blast ID ^a	Idealized Shockwave Properties ^{1b}			Midspan Displacements ²			
			P _r (kPa)	I _r (kPa·msec)	t _{di} (msec)	d _{max} (mm)	d _{anls} (mm)	d _{anls} /d _{max}	Absolute % Error
SCC- 4#3	1	Blast 1	43.7	430	19.7	33.7	28.4	0.84	15.8%
	2	Blast 2	82.7	787	19.0	139.2	143.8	1.03	3.3%
UHPC-4#3	1	Blast 1	49.4	466	18.9	25.1	22.0	0.88	12.4%
	2	Blast 2	86.7	789	18.2	97.2	94.2	0.97	3.1%
UHPC-4#3-HS	1	Blast 1	49	459	18.7	19.1	20.6	1.08	7.9%
	2	Blast 2	88.9	834	18.8	57.8	71.2	1.23	23.1%
UHPC-6#3-HS	1	Blast 1	49.2	448	18.2	17.4	18.3	1.05	4.9%
	2	Blast 2	94.9	823	17.3	53.8	51.6	0.96	4.0%
UHPC-4#3-SS	1	Blast 1	45.5	472	20.7	19.7	17.2	0.84	16.0%
	2	Blast 2	82.2	810	19.7	57.0	84.2	1.15	14.7%
UHPC-6#3-SS	1	Blast 1	47.7	457	19.2	22.4	23.0	1.02	2.5%
	2	Blast 2	92.2	829	18.0	58.6	83.2	1.42	42.0%
	3	Blast 3	104	969	18.6	102.2	116.4	1.14	13.9%
	4	Blast 3 (2)	93.7	812	17.3	92.4	80.6	0.87	12.8%
Statistical analysis					d _{anls} /d _{max}		Overall	Blast 1	Blast 2
					Mean		1.03	0.95	1.13
					Standard deviation		0.16	0.11	0.18
					Coefficient of variation		15.8%	11.6%	15.8%
					Max.		1.42	1.08	1.42
					Min.		0.84	0.84	0.96

^a Shots which resulted in bar rupture were not included in the analysis.

^b Idealized triangular pressure-time history with same reflected pressure and impulse observed in experiments.

¹ P_r = Reflected pressure; I_r = Reflected impulse; t_{di} = Idealized positive phase duration = 2×I_r / P_r;

² d_{max} = experimental mid-span displacement; d_{anls} = analytical SDOF mid-span displacement.

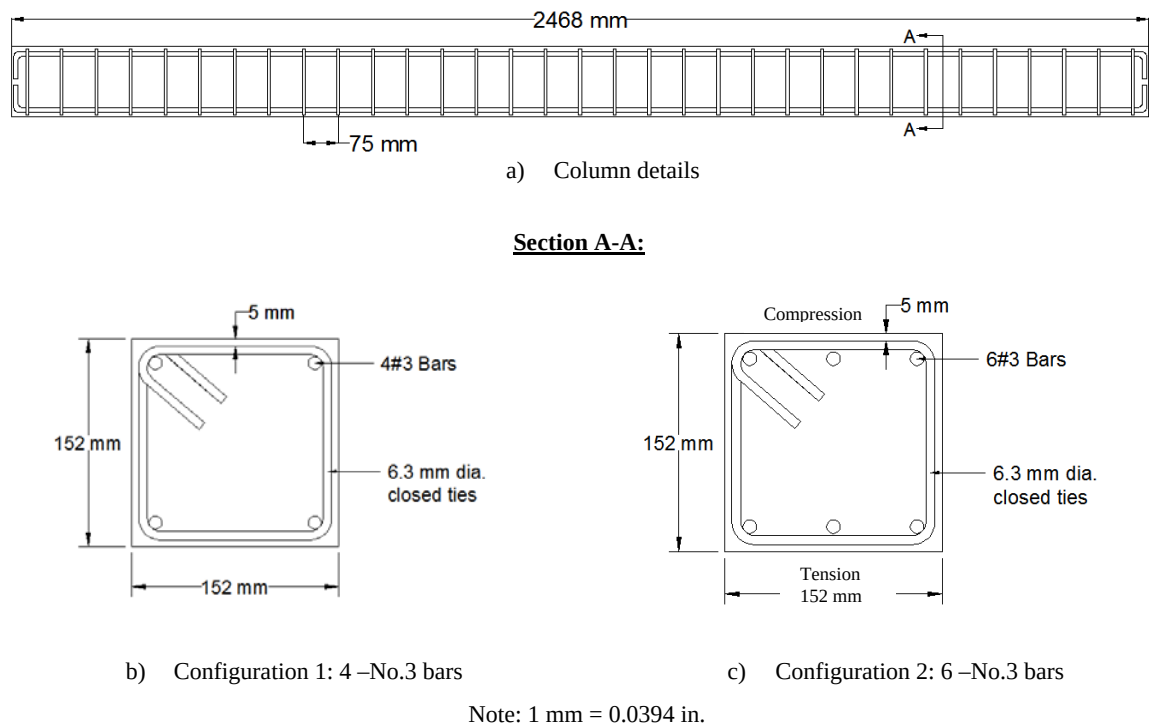


Fig. 1 – Column dimensions and reinforcement details.

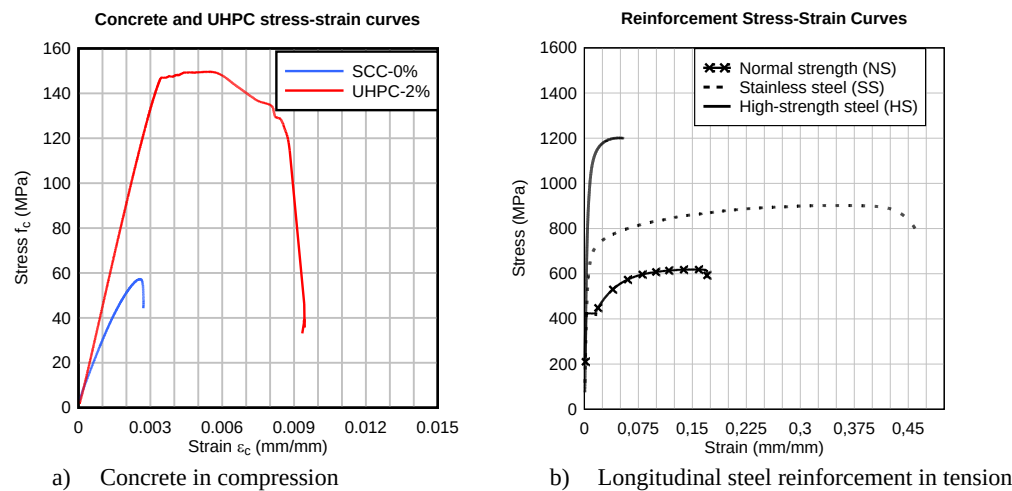


Fig. 2 – Typical stress-strain curves for concrete and steel reinforcement.

Note: 1 MPa = 145 Psi

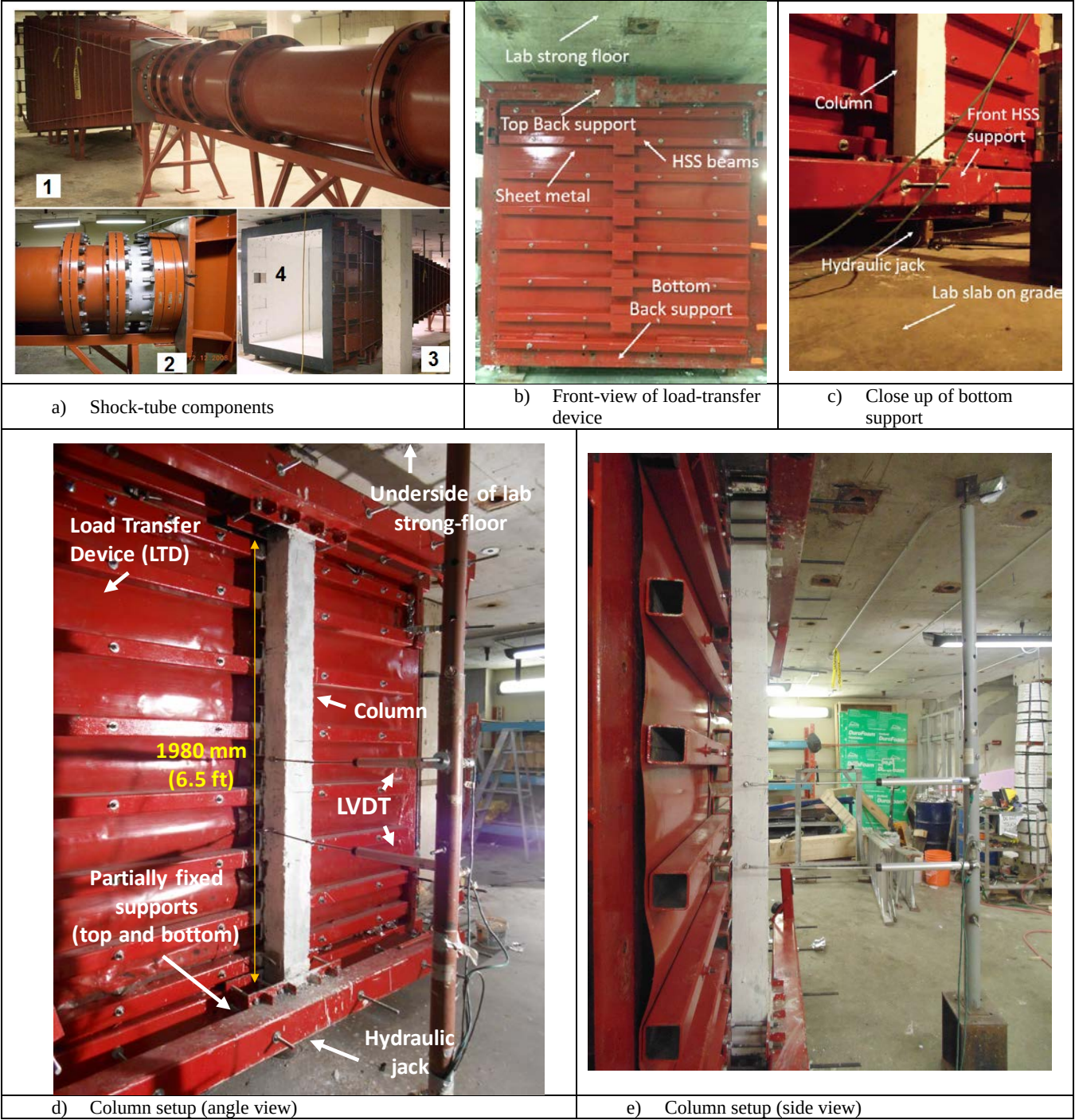


Fig. 3 – Shocktube and column test setup.