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Recent Developments in High Strain Rate
Mechanics and Impact Behavior of Concrete

SP-347

Editors:
Eric Jacques and Mi G. Chorzepa



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Recent Developments in High Strain Rate Mechanics and Impact Behavior of Concrete

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PREFACE

Recent Developments in High Strain Rate Mechanics and Impact Behavior of Concrete

This Symposium Volume reports on the latest developments in the field of high strain rate mechanics and behavior of concrete subject to impact loads. This effort supports the mission of ACI Committee 370 “Blast and Impact Load Effects” to develop and disseminate information on the design of concrete structures subjected to impact, as well as blast and other short-duration dynamic loads. Concrete structures can potentially be exposed to accidental and malicious impact loads during their lifetimes, including those caused by ballistic projectiles, vehicular collision, impact of debris set in motion after an explosion, falling objects during construction and floating objects during tsunamis and storm surges. Assessing the performance of concrete structures to implement cost-effective and structurally-efficient protective measures against these extreme impacting loads necessitates a fundamental understanding of the high strain rate behavior of the constituent materials and of the characteristics of the local response modes activated during the event.

This volume presents fourteen papers which provide the reader with deep insight into the state-of-the-art experimental research and cutting-edge computational approaches for concrete materials and structures subject to impact loading. Invited contributions were received from international experts from Australia, Canada, China, Czech Republic, Germany, South Korea, Switzerland, and the United States. The technical papers cover a range of cementitious materials, including high strength and ultra-high strength materials, reactive powder concrete, fiber-reinforced concrete, and externally bonded cementitious layers and other coatings. The papers were to be presented during two technical sessions scheduled for the ACI Spring 2020 Convention in Rosemont, Illinois, but the worldwide COVID-19 pandemic disrupted those plans.

The editors thank the authors for their outstanding efforts to showcase their most current research work with the concrete community, and for their assistance, cooperation, and valuable contributions throughout the entire publication process. The editors also thank the members of ACI Committee 370, the reviewers, and the ACI staff for their generous support and encouragement throughout the preparation of this volume.

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On the Rate Sensitive Fracture Behavior of Strain-Hardening Cement-Based Composites (SHCC) Depending on Fiber Type and Matrix Composition

Iurie Curosu, Viktor Mechtcherine, Daniele Forni, Simone Hempel and Ezio Cadoni

Synopsis: Strain-hardening cement-based composites (SHCC) represent a special type of fiber reinforced concretes, whose post-elastic tensile behavior is characterized by the formation of multiple, fine cracks under increasing loading up to failure localization. The high inelastic deformability in the strain-hardening phase together with the high damage tolerance and energy dissipation capacity make SHCC promising for applications involving dynamic loading scenarios, such as earthquake, impact or blast.

However, the main constitutive phases of SHCC, i.e. matrix, fibers and interphase between them, are highly rate sensitive. Depending on the SHCC composition, the increase in loading rates can negatively alter the balanced micromechanical interactions, leading to a pronounced reduction in strain capacity. Thus, there is need for a detailed investigation of the strain rate sensitivity of SHCC at different levels of observation for enabling a targeted material design with respect to high loading rates.

The crack opening behavior is an essential material parameter for SHCC, since it defines to a large extent the tensile properties of the composite. In the paper at hand, the rate effects on the crack opening and fracture behavior of SHCC are analyzed based on quasi-static and impact tensile tests on notched specimens made of three different types of SHCC. Two SHCC consisted of a normal-strength cementitious matrix and were reinforced with polyvinyl-alcohol (PVA) and ultra-high molecular weight polyethylene (UHMWPE) fibers, respectively. The third type consisted of a high-strength cementitious matrix and UHMWPE fibers. The dynamic tests were performed in a split Hopkinson tension bar and enabled an accurate description of the crack opening behavior in terms of force-displacement relationships at displacement rates of up to 6 m/s (19.7 ft/s).

Keywords: SHCC, fiber reinforcement, strain-hardening, PVA, UHMWPE, impact, split Hopkinson tension bar, fracture energy

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INTRODUCTION

SHCC consist of fine-grained cementitious matrices and short, high-performance polymer fibers in volume fractions of up to 2% [1]. The micromechanics-based material design aims a balance between the cracking behavior of the cementitious matrix and crack bridging behavior of the fibers, in this way ensuring a pronounced tensile ductility through the successive formation of fine, densely spaced cracks under increasing tensile loading. The remarkably high strain capacity of SHCC in combination with their favorable fresh-state properties allows their efficient application not just as main material in structural elements but also as thin strengthening layers on existing structures [2-6], including those prone to severe dynamic actions [7-9].

However, under increasing loading rates, the disproportionate changes of the components’ properties alter the micromechanical balance as achieved for the case of quasi-static loading [10]. This might yield negative effects on the composite tensile behavior [11-14]. For the formulation of reliable material design concepts with respect to increased loading rates, the rate effects at the level of constituents, of the collective crack bridging and of the resulting composite behavior must be investigated and quantified.

Several studies were reported on the effect of increasing loading rates on the tensile behavior of SHCC, most of them being conducted in the range of low and moderate loading rates [9,11,13]. With regard to impact tensile loading, few reports were published [12,15,16], among which only Curosu et al. [16] and Heravi et al. [17] offered a detailed quantitative characterization of the impact tensile behavior of various SHCC in form stress-strain curves.

The crack opening behavior and fracture energy of notched SHCC specimens were only investigated in [12,15] in Hopkinson bar spall experiments. Using the evaluation methodology proposed by Schuler et al. [18], the specific fracture energy was derived from the impulse transfer between the separated fragments of notched specimens. These investigations provided valuable insights into the effects of high loading rates on the tensile behavior of SHCC

including the change of failure modes depending on loading rate and crack opening speed. However, the indirect quantitative evaluation left margin for interpretations.

Compared to the spall experiments in the Hopkinson bar, the split Hopkinson tension bars exhibit several important advantages: (1) they enable a direct material characterization in terms of force-crack opening curves, and (2) they imply a direct tensile loading wave. Among the various modifications of the split Hopkinson tension bar [19,20,21], only those configurations are suitable for SHCC, which can ensure a long tensile loading pulse as well as long incident and transmitter bars for capturing the entire specimen response including softening [16,17,22].

The current work is a continuation of the investigations reported in [16] using a split Hopkinson tension bar, in which three types of SHCC were investigated and in which the critical aspects related to the dynamic tensile testing of SHCC were highlighted. Since the crack opening behavior in SHCC is an essential material parameter, the same composites as in the previous study were tested on notched specimens under quasi-static and impact tensile loading. The impact testing setup enabled an accurate recording of the stress-crack opening displacement relationships and, in turn, a detailed quantitative assessment of the rate sensitive crack opening behavior of SHCC depending on fiber type and matrix composition.

MATERIALS UNDER INVESTIGATION AND SPECIMEN PREPARATION

The PVA fibers used in this investigation are high-tenacity Kuralon K-II REC15 fibers produced by Kuraray, Japan [23]. Their diameter is 40 μm (0.001575 in) and the cut length is 12 mm (0.47 in); see Table 1. According to Kuraray, they have a tensile strength of 1600 MPa (232060 psi) and a Young's modulus of 40 GPa (5801.5 ksi). Since the pronounced hydrophilic nature of PVA fiber results in a strong interfacial bond with cement-based matrices [24], these particular fibers are coated with an oiling agent during production, aiming a limitation of the bond strength and a favorable crack-bridging behavior in cement-based matrices [25,26].

The UHMWPE fibers (called shortly PE in the current paper) are produced by DSM, the Netherlands, under the product name Dyneema SK62 [27]. They have a tensile strength of 2500 MPa (362595 psi) and a Young's modulus of 80 GPa (11603 ksi). Their cut length is also 12 mm (0.47 in), but the diameter is just 20 μm (0.000787 in), i.e., half the diameter of the PVA fibers; see Table 1.

Same as in the previous studies [15,16], the SHCC mixtures were named according to the combinations of matrix and fiber, in which the normal-strength and high-strength matrices carry the indices M1 and M2, respectively. The normal-strength matrix M1 was specifically designed to be used with the PVA fibers under investigation. To ensure optimal fiber-matrix adhesion, the strength and density of the cementitious matrix were limited by a high content of fly ash and a moderate water-to-binder ratio; see Table 2. This is a common SHCC composition and it was previously studied at different scales and under different loading conditions, see e.g. [28,29].

According to Yang and Li [11,14], the frictional interaction of the PE fibers with the cementitious matrix is expected to be less loading rate sensitive in comparison to the chemical fiber-matrix bond in M1-PVA, which should also reduce the negative effect of the increasing loading rates on the tensile behavior of PE-SHCC when compared with PVA-SHCC. Thus, the combination of the normal-strength matrix with PE fibers was investigated to highlight the influence of fiber properties and, foremost, of the fiber-matrix bond on the tensile behavior of the composite under impact loading in comparison to M1-PVA. The normal-strength matrix M1 was combined with the PE fibers without adjusting it for the smaller fiber diameter, hydrophobic nature and weaker, purely frictional bond.

Because of their hydrophobic nature, but also due to their higher tensile strength in comparison to PVA fibers, the PE fibers are normally used in combination with high-strength cementitious matrices, such as M2 presented in this work and in the publications by other authors [9,30,31]. The high-strength matrix M2 was designed by adapting M1 for reaching high strength on one hand, and an enhanced fiber anchorage, on the other hand. The optimization with respect to the fresh-state properties was mainly focused on reaching a homogeneous mixture and a satisfactory workability. The high-strength matrix M2 has a high cement content and a low water-to-binder ratio of 0.18, while binder consists of cement and silica fume. In contrast to M1, fly ash was not added and the content of fine sand was significantly reduced. As a consequence, this cementitious matrix had not only a higher strength in comparison to M1 but also a higher packing density in fresh state, as well as a denser microstructure in hardened state.

For specimen production, 280 mm (11.02 in)-long SHCC beams were cast with side dimensions of 75 mm by 75 mm

(2.95 in by 2.95 in). The cylindrical specimens with a diameter of 20 mm (0.79 in), as imposed by the impact testing setup, were core-drilled longitudinally from the beams' middle regions [16]. Given the different specimen fixations in the quasi-static and dynamic testing setups, the total length of the samples intended for the quasi-static tests was 50 mm (1.97 in), while the specimens used for dynamic testing were 25 mm (0.98 in) long. The notches had a depth of 4 mm (0.16 in) and were cut with a 0.6 mm (0.024 in)-thin circular saw blade. They reduced the specimen cross-section by a factor of 2.8 from 314 mm² (0.00338 ft²) to 113.4 mm² (0.00122 ft²). This was done to ensure a considerably weaker middle section and to inhibit multiple cracking.

Due to logistical considerations, the core drilling and cutting of the specimens occurred on the 7th day after casting. The testing age was 14 days as in the previous test series and the fiber volume fraction was 2% in all tested composites.

SETUP FOR QUASI-STATIC EXPERIMENTS

To conduct a consistent experimental series with comparable results in both loading regimes, an adjustable fixing device for cylindrical specimens was built for the quasi-static tensile experiments; see Figure 1.

The specimens were glued at both ends in 12.5 mm (0.49 in)-thick steel rings. With a total specimen length of 50 mm (1.97 in), the remaining free length of 25 mm (0.98 in) was equal to that of the specimens tested in the split Hopkinson tension bar. The rings were bolted to special steel stamps and rigidly connected to the testing machine with steel rods, ensuring non-rotatable fixities. A bi-component glue consisting of the fluid component Degadur®1801 and powder component Degadur®7742 F (N), produced by Evonik Röhm GmbH, was used for this purpose. The circular frame with an adjustable height was fixed around the embedment rings and it supported two Linear Variable Differential Transducers (LVDTs) to measure the axial deformations.

The experiments were performed in a deformation controlled regime with a displacement rate of 0.05 mm/s ($16 \cdot 10^{-5}$ ft/s). At least five specimens were tested for each SHCC type.

IMPACT TENSION TESTS WITH THE SPLIT HOPKINSON TENSION BAR

In the split Hopkinson tension bar [22], the specimen is glued between two aluminum bars; see Figure 2. A bi-component epoxy resin was used to glue the specimens to the bars, which required at least 8 hours hardening time at room temperature. Accordingly, only one SHCC specimen could be tested per day. Three specimens were tested for each material combination. Because the specimens intended for a testing set were generally produced in the same batch, the testing ages of the specimens varied between 13 and 15 days.

The split Hopkinson tension bar consisted of a 3 m (118.11 in)-long input bar and a 6 m (236.22 in)-long output bar, both bars being made of aluminum and having a diameter of 20 mm (0.79 in). A 6 m (236.22 in)-long bar made of high-strength steel and having a diameter of 12 mm (0.47 in) was used as a pre-tensioned bar for generating the loading pulse; see Figure 2. The diameter of the pre-tensioned bar was chosen to obtain the same acoustical impedance as that of the input bar and avoid any spurious reflection at their connection.

During the experiment, the free end of the pre-tensioned bar is pulled with the help of a hydraulic jack, while the end connected to the input bar is fixed by a blocking device. By pulling the free end of the pre-tensioned bar, elastic energy is stored in it. At the rupture of the brittle intermediate piece in the blocking device, a sudden release of the stored elastic energy occurs, generating a tensile wave of trapezoidal shape of 2.4 ms duration with a rise-time of about 60 μ s [16]. In the present investigation, the target maximum displacement speed was 6 m/s (19.69 ft/s).

The generated pulse propagates along the input bar with the velocity C_0 of the elastic wave with its shape remaining constant. When the incident pulse ε_I reaches the specimen, part ε_R of it is reflected by the specimen due to the different impedances of the specimen material and aluminum, whereas another part ε_T passes through the specimen and propagates into the output bar. The relative amplitudes of the incident, reflected and transmitted pulses depend on the mechanical properties of the specimen. Strain gauges glued to the input and output bars of the device are used to measure as a function of time the elastic deformation created on both bars by said incident, reflected, and transmitted pulses.

The application of the elastic, uniaxial stress wave propagation theory to the Hopkinson bar system [32] allows calculation of the forces F_1 and F_2 and the displacements δ_1 and δ_2 acting on the two faces of the specimen in contact

with the input and output bars, respectively. The calculations are performed using Eqs. 1 to 4, which are based on the recorded strains ε_I , ε_R , ε_T of the elastic input and output bars [33]:

$$F_1 = E_0 A_0 (\varepsilon_I + \varepsilon_R) \quad (1)$$

$$F_2 = E_0 A_0 \varepsilon_T \quad (2)$$

$$\delta_1 = C_0 \int (\varepsilon_I - \varepsilon_R) dt \quad (3)$$

$$\delta_2 = C_0 \int \varepsilon_T dt, \quad (4)$$

where E_0 is the bar modulus of elasticity, A_0 the bar cross-section area, and C_0 the elastic wave velocity. The stress in the specimen can be determined with Eq. 5:

$$\sigma = \frac{F_1 + F_2}{2A_s} \quad (5)$$

where A_s is the specimen's effective cross-sectional area.

In the case of the short specimens with uniform cross-section, the travel time of the wave through the specimen is small in comparison to the duration of the test. Thus, the specimen can be considered as being in load equilibrium at its ends and in a uniform stress state created by the many wave reflections taking place at the ends of the specimen. The stresses at both ends are equal, i.e., the sum of the incident and reflected waves equals the transmitted wave ($\varepsilon_I + \varepsilon_R = \varepsilon_T$). The time histories of the stress and crack opening displacement (COD) in the specimen can be calculated using Eqs. 6 and 7:

$$\sigma(t) = E_0 \frac{A_0}{A_s} \varepsilon_T(t) \quad (6)$$

$$COD(t) = -2C_0 \int_0^t \varepsilon_R(t) dt \quad (7)$$

In verifying whether the specimen is in stress equilibrium, the stresses at its both ends must be compared. The stresses at the output end are calculated according to Eq. 6 while the stresses at the interface with the input bar are calculated considering the input and reflected waves using Eq. 8:

$$\sigma(t) = E_0 \frac{A_0}{A_s} (\varepsilon_I(t) + \varepsilon_R(t)) \quad (8)$$

Note that the notches in the specimens have a negative effect on wave propagation by causing additional reflections. The accurate assessment and quantification of these effects can be done with the help of numerical simulations, which is beyond the aim of the presented work. Nevertheless, the experimental results presented in the next sections will be discussed also considering the above-mentioned aspects.

RESULTS AND DISCUSSION

Quasi-static tension experiments

Comparing the current results to the previous investigations in [12] and [15], a clear effect of the specimen size and relative notch depth on the response of the specimens can be observed. Both the matrix cracking stresses and the peak crack bridging stresses are higher in the present investigation in comparison to the results obtained in the tests on larger cylindrical specimens with different ratio of notch depth to sample diameter: 4 mm/20 mm (0.16 in/0.79 in) vs. 18 mm/75 mm (0.71 in/2.95 in).

The average matrix cracking stress of the M1-PVA specimens is around 5.2 MPa (754 psi), while the average peak stress is 7.8 MPa (1131 psi), see Figures 3 and 4 and Table 3. The scattering of the strength parameters obtained on M1-PVA specimens is the lowest among the tested SHCC. This is a result of a proper homogeneity of the composite ensured by the optimum fresh-state properties and, consequently, uniform fiber distribution. The average fracture energy (area under the stress-displacement curves) of 5.4 kJ/m² (0.50 kJ/ft²) is the lowest among the composites under

investigation, which is a result of a complete fiber rupture after only a short pullout phase. The latter is caused by the slip-hardening pullout behavior of the PVA fibers [15].

For the M1-PE samples, the scatter of stress-displacement curves is considerably higher. This can be at least partly explained by the relatively poor workability of this material in fresh state, likely causing a non-uniform fiber distribution. The average matrix cracking stress of M1-PE is 5.0 MPa (725 psi) and the average peak crack bridging stress is 10.6 MPa (1537 psi), which is notably high compared to the strength parameters measured on unnotched specimens in the previous investigation [16]: 2.4 MPa (348 psi) for first crack stress and 3.6 MPa (522 psi) for tensile strength. It may be assumed that the notches coincided with regions of high fiber concentration in a considerable number of specimens. Another reason for such high values may be a particular favorable behavior of M1-PE under plain-strain conditions at the notches.

It should be noticed in respect to Figures 3 and 4 that further matrix cracking occurred in the vicinity of the notched sections due to the high crack bridging strength. This is also because SHCC are designed to form multiple cracking, often making the notches ineffective in impeding the formation of more than one crack.

The M1-PE specimens showed a complete fiber pullout, which can be also deduced from the descending branches of the stress-displacement curves in Figure 3. This resulted in the highest average fracture energy of 16.9 kJ/m² (1.57 kJ/ft²) among the tested SHCC, see also Table 3.

The M2-PE samples yielded an average first crack stress of 7.0 MPa (1015 psi) and peak crack bridging stress of 17.1 MPa (2480 psi). Figures 3 and 4 show that M2-PE, same as the M1-PE specimens, exhibited multiple cracking despite the notches. However, given the relatively strong fiber-matrix bond and the fiber length of 12 mm (0.47 in), the crack localization in the high-strength SHCC M2-PE was also accompanied by a pronounced fiber rupture, which limited the average fracture energy to 10.3 kJ/m² (0.96 kJ/ft²). Note that these results are opposite to those reported in [15] on larger cylindrical specimens, which accentuates the strong influence of the specimen size on the apparent material properties.

Impact tension experiments

In the split Hopkinson tension bar the specimens are subjected to two successive loading conditions: the initial stage is associated to an increasing displacement rate (acceleration), and the second stage yields a relatively constant displacement rate, see gray, dashed curves in Figure 5. In the case of notched specimens, wave reflections occur additionally at the notch flanks, resulting in pronounced oscillations in the input bar in the initial loading phase. Together with the wave effects generated by the cracking processes, this yields considerable differences between the signals in the initial loading phase measured by the strain gauges on the input and output bars. Wave analysis only yields stress equilibrium after matrix cracking, making it impossible to reliably assess the crack formation stress. Thus, only the crack bridging behavior of the investigated SHCC will be discussed in this work based on the transmitted waves, since the oscillations are considerably less pronounced in the transmitter bar, as shown in Figure 5.

M1-PVA yielded the lowest peak crack bridging stress. However, contrary to the quasi-statically loaded specimens, the specimens tested dynamically showed the highest average fracture energy as a result of a complete fiber pullout. The Dynamic Increase Factor (DIF) of the peak crack bridging stress is 1.4, while that of the fracture energy is 1.6, see Table 4.

The DIF of the crack bridging capacity of M1-PE is 1.4, considerably lower than as identified on unnotched specimens in the previous study. Such a small DIF is caused by the unusually high crack bridging capacity of the notched M1-PE specimens under quasi-static loading. The average fracture energy of M1-PE decreased by more than two times due to the strong increase in crack opening speed. Although the optical observations of the fracture surfaces of M1-PE show no obvious changes compared to the samples tested quasi-statically, see Figure 6, the microscopic images in Figure 7 give a clear insight into the rate dependent changes occurring at the interface. The pronounced oscillations associated to the M1-PE curves are a result of a pronounced multiple cracking occurring in the notch region during crack opening, as it will be shown in the next section.

As opposed to M1-PVA, the failure mode of the high-strength SHCC M2-PE shifted towards a considerably more pronounced fiber rupture with increasing displacement rate, which can be deduced from the corresponding curves in Figure 5. Compared to the other SHCC types, crack bridging of M2-PE vanished already at a crack opening of 0.15 mm (0.0059 in) and the dynamic peak crack bridging capacity of M2-PE is lower than under quasi-static conditions ($DIF = 0.8$). These results are confirmed by the fracture surfaces of M2-PE, which show just very short protruding fibers, see Figures 6 and 7. Furthermore, the average peak crack bridging stress of M2-PE is even lower than that of M1-PE, although in the tests on unnotched specimens in the previous study M2-PE yielded a higher tensile strength than M1-PE [16].

Same as under quasi-static loading, both types of SHCC made with PE fibers formed additional cracks in the notch regions, see Figure 8. This resulted in stress oscillations, as seen in Figure 5, and their wavelength is comparable to the displacement at which the crack bridging in the notched M2-PE specimens vanished. Thus, the reason behind the apparent dynamic decrease of the crack bridging capacity of M2-PE could be related to disadvantageous wave effects, i.e. relatively short softening phase superimposed with additional oscillations caused by matrix cracking and wave reflections at the notch flanks.

Morphological observations and discussion

Additionally to the mechanical testing, optical investigations were performed on fractured specimens to complement the test series with qualitative characterizations. Figure 6 presents the state of the fracture surfaces of representative SHCC specimens. The fracture surfaces confirm the findings from the previous studies in spall experiments, in which M1-PVA [12] showed pronounced fiber rupture under quasi-static loading and complete pullout under impact tension, while M2-PE [15] showed considerably more pronounced fiber rupture compared to the quasi-static case.

As opposed to the SHCC made with PVA fibers, the composites reinforced with PE fibers showed a trend towards fiber rupture with increasing crack opening speed. Based on the single-fiber pullout and tension tests at different displacement rates, Curosu et al. [15] showed that the difference in the strain-rate sensitivities of PVA and PE fibers with respect to their tensile strength and Young's modulus plays an essential role in achieving different fracture modes in SHCC depending on the crack opening speed.

It was shown that the PVA fibers yield a stronger dynamic enhancement of their tensile strength compared to that of their Young's modulus. On contrary, the PE fibers show a stronger enhancement of the Young's modulus compared to the tensile strength. Assuming a rate independent Poisson's ratio, this means that the dynamically loaded PVA fibers can bare higher tensile stresses which, in combination with a moderately increased Young's modulus, leads to stronger radial contractions compared to the quasi-static case, in this way limiting their anchorage strength.

These consideration are supported by the Environmental Scanning Electron Microscopy (ESEM) images in Figure 7, which present pulled out fibers on the fracture surfaces of the investigated SHCC. The ruptured PVA fiber belonging to the quasi-statically tested specimen shows a pronounced surface damage resulted from pilling and scratching during the short pullout phase, while closer to the fiber root, hydration products attached to the fiber can be observed, indicating a partly cohesive bond failure. The PVA fibers are known for their slip-hardening pullout character in cementitious matrices under quasi-static loading [24]. However, under high crack-opening rates the surface of the pulled out PVA fiber is relatively smooth, with no pronounced surface damage and with no attached hydration products.

At the same time, the dynamically loaded PE fibers with their less accentuated dynamic increase in tensile strength and the more pronounced increase in Young's modulus should yield limited transversal contractions during the crack bridging process, having a positive influence on the frictional interfacial stresses [34,35]. The resulting dynamic increase of the apparent bond strength was higher than that of the fiber tensile strength and led to increasingly more pronounced fiber rupture under increasing crack opening speeds. The ESEM images corresponding to M1-PE subjected to quasi-static loading show that the PE fibers exhibit no pronounced surface damage and only few attached

matrix fragments. After impact loading the fibers show a pronounced surface damage, being almost completely covered in matrix fragments and fly ash particles.

Finally, the fracture surfaces of M2-PE show only short protruding fibers, confirming the dominating fiber rupture mode. This can be traced back to the already strong anchorage under quasi-static conditions, which becomes excessively strong under impact tensile loading.

Although the state of the fibers in the localization cracks of various SHCC correspond to the findings from the previous investigations by the authors [12,15,16], the quantitative results presented in this paper are contradicting the findings on notched specimens in spall experiments [12,15]. In the spall tests at crack opening speeds of about 8 to 9 m/s (26 to 29 ft/s), M2-PE yielded the highest specific fracture energy, followed by M1-PE and finally by M1-PVA [15]. Furthermore, the values for specific fracture energy assessed in spall experiments on notched samples were extremely high: 38.4 kJ/m² (3.57 kJ/ft²) for M1-PE and 55.8 kJ/m² (5.18 kJ/ft²) for M2-PE. One reason for this could be the more pronounced multiple cracking developed around the notches in the larger specimens compared to those presented in the paper at hand.

Figure 8 presents polished longitudinal sections cut from the middle of representative tested specimens. The aim of these figures is to demonstrate the extent of multiple cracking which occurred in the vicinity of the failure localization cracks. Note that the specimens were initially embedded in epoxy resin for enabling the subsequent cutting and polishing without disturbing their inner structure. Since the M1 matrix has a significantly lower density than the high-strength matrix M2, the epoxy resin penetrated deeper inside the matrix, in this way visually enlarging the effective crack width. For demonstrating the actual crack pattern and the profile of the fracture surfaces, white contours were traced manually in the figures. The multiple cracking exhibited by the specimens as shown in Figure 8 is considerably less pronounced as in the previous investigations both under quasi-static and dynamic loading.

Furthermore, it should be noted that in the spall experiments, the fracture energy was derived from the impulse transfer between the fractured fragments (reduction of fragment velocities) using the evaluation methodology proposed by Schuler et al. [18]. According to this evaluation procedure, the fracture energy is the energy dissipated between two moments in time: (1) at crack initiation and (2) at complete crack opening. The velocities of the fragments at crack initiation were computed analytically, while the velocities of the fragments during their separation were measured with a high-speed optical extensometer [15]. The evaluation methodology proposed by Schuler et al. [18] relies on the assumption that the tested material exhibits elastic behavior up to failure localization. This condition is necessary for an accurate reconstruction of the reflected tensile wave inside the specimen based on the input compressive wave. Although applicable for concrete [36], the ductile and strain-hardening behavior of SHCC renders this assumption questionable and requires further investigations regarding the applicability of such an evaluation protocol in spall experiments on SHCC, for example with the help of numerical simulations. On contrary, the force-displacement curves presented in the current work imply a direct and explicit basis for material characterization.

CONCLUSIONS AND OUTLOOK

Quasi-static and impact tensile experiments were performed on notched cylindrical specimens with a diameter of 20 mm (0.79 in). With the help of the split Hopkinson tension bar, displacement rates of 6 m/s (19.7 ft/s) were achieved. The main findings from the experimental investigation are summarized below.

When subjected to high strain rates, the anchorage of the PVA fibers in the normal-strength cementitious matrix suffers a peculiar alteration, leading to a strain-softening behavior. Reason for this is assumed to be the disproportionate dynamic increases of fibers' tensile strength (high increase) and Young's modulus (moderate increase), which influence the radial contractions of the tensioned fibers and, consequently, their pullout behavior.

Compared to the PVA fibers, the PE fibers show an opposite rate effect, leading to a more pronounced fiber rupture under high displacement rates and having a negative influence of the dynamic fracture energy of the corresponding SHCC. However, as opposed to PVA fibers, the PE fibers ensure a predictable evolution of bond strength under

increasing strain rates. This enables a straightforward material design of impact resistant SHCC by adjusting the matrix composition, i.e. limiting the matrix strength and density.

The stress-deformation relationships measured in impact experiments on notched specimens showed that the dynamic fracture energy of the high-strength SHCC M2-PE is the lowest among the investigated SHCC due to the pronounced fiber rupture and less pronounced multiple cracking, while M1-PVA yielded the highest fracture energy as a result of a complete fiber pullout.

The results on notched specimens in the split Hopkinson tension bar contradict the findings from spall experiments in the previous investigations and indicate that the evaluation protocol based on momentum transfer should be critically evaluated with regard to its applicability to SHCC. At the same time, the effect of notches on the wave propagation and on the accuracy of the derived material properties in the split Hopkinson tension bar should be assessed with the help of numerical simulations.

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Table 1— Properties of the polymer fibers under investigation [23,27]

Producer	Kuraray	DSM
Brand	Kuralon®	Dyneema®
Material (fiber type)	PVA	UHMWPE
Diameter [μm] (in)	40 (0.001575)	20 (0.000787)
Length [mm] (in)	12 (0.4724)	12 (0.4724)
Aspect ratio [-]	300	600
Density [g/cm ³] (lb/ft ³)	1.26 (78.65)	0.97 (60.55)
Tensile strength [MPa] (psi)	1600 (232060)	2500 (362585)
Tensile modulus of elasticity [GPa] (ksi)	40 (5802)	80 (11603)
Elongation at break [%]	7	3.5

Table 2— Mixture compositions of the SHCC under investigation

Components / content / SHCC type	M1-PVA	M1-PE	M2-PE
CEM I 42.5R-HS [kg/m ³] (lb/ft ³)	505 (31.53)	505 (31.53)	-
CEM I 52.5 R-SR3/NA [kg/m ³] (lb/ft ³)	-	-	1460 (91.14)
fly ash [kg/m ³] (lb/ft ³)	621 (38.76)	621 (38.76)	-
silica fume [kg/m ³] (lb/ft ³)	-	-	292 (18.22)
quartz sand 0.06 - 0.2 mm [kg/m ³] (lb/ft ³)	536 (33.46)	534 (33.33)	145 (9.05)
viscosity modifying agent [kg/m ³] (lb/ft ³)	4.8 (0.30)	4.8 (0.30)	-
water [kg/m ³] (lb/ft ³)	338 (21.10)	334 (20.85)	315 (19.66)
HRWRA (superplasticizer) [kg/m ³] (lb/ft ³)	10 (0.62)	25 (1.56)	35 (2.18)
PVA fibers [kg/m ³] (lb/ft ³)	26 (1.62)	-	-
UHMWPE fibers [kg/m ³] (lb/ft ³)	-	20 (1.24)	20 (1.24)

Table 3— Average values of the mechanical properties of the investigated SHCC obtained from quasi-static tension experiments; standard deviations are given in parentheses next to the metric values

SHCC type and specimen geometry	M1-PVA	M1-PE	M2-PE
First crack stress [MPa] (psi)	5.2 (0.7) (754)	5.0 (1.2) (725)	7.0 (2.1) (1015)
Peak-crack bridging stress [MPa] (psi)	7.8 (1.1) (1131)	10.6 (2.9) (1537)	17.1 (3.0) (2480)
Fracture energy [kJ/m ²] (kJ/ft ²)	5.4 (0.4) (0.50)	16.9 (6.1) (1.57)	10.3 (2.8) (0.96)

Table 4—Dynamic mechanical properties of the investigated SHCC and the corresponding DIFs; standard deviations are given in parentheses next to the metric values

SHCC type and specimen geometry	M1-PVA	M1-PE	M2-PE
Peak crack bridging stress [MPa] (psi)	10.2 (1.8) (1479)	14.6 (2.5) (2118)	13.5 (1.8) (1958)
Fracture energy [kJ/m ²] (kJ/ft ²)	8.7 (2.2) (0.81)	6.2 (2.6) (0.58)	1.3 (0.2) (0.12)
DIF of peak crack bridging stress	1.4	1.4	0.8
DIF of fracture energy	1.6	0.37	0.1



Fig. 1— Quasi-static testing setup with a notched SHCC specimen

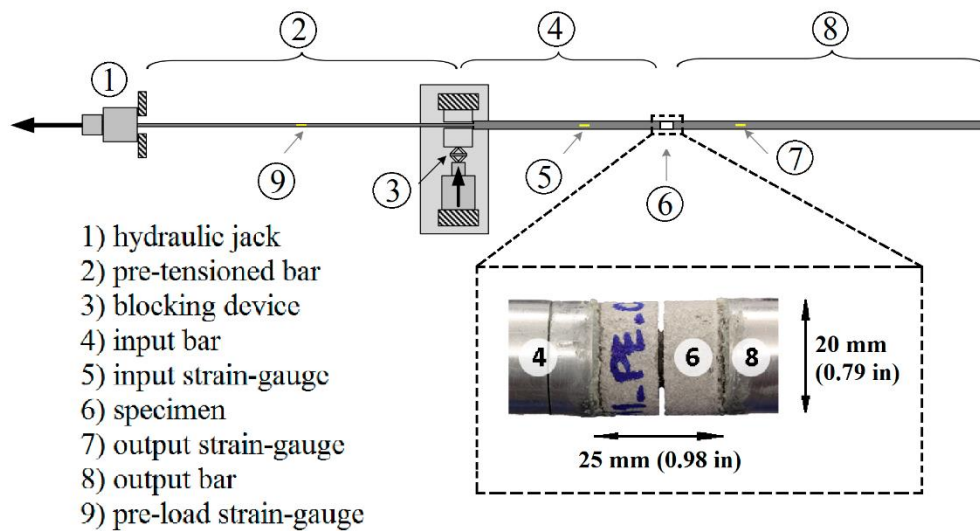


Fig. 2— Split Hopkinson tension bar testing configuration with a notched SHCC specimen

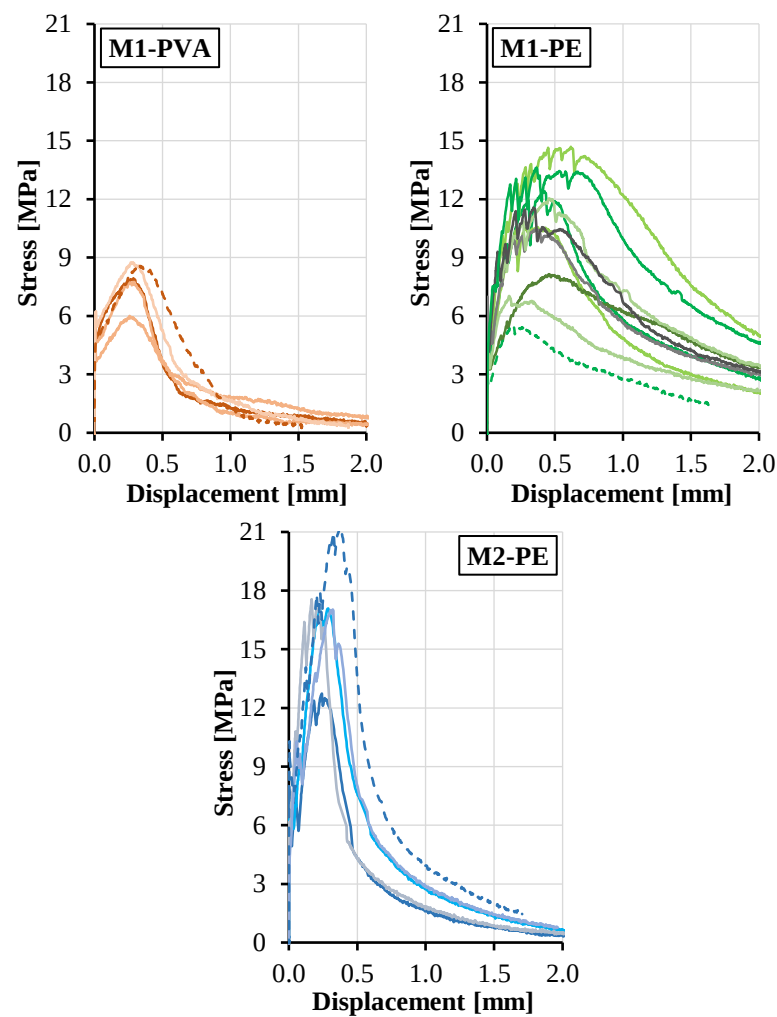


Fig. 3— Quasi-static stress-displacement curves of notched SHCC specimens

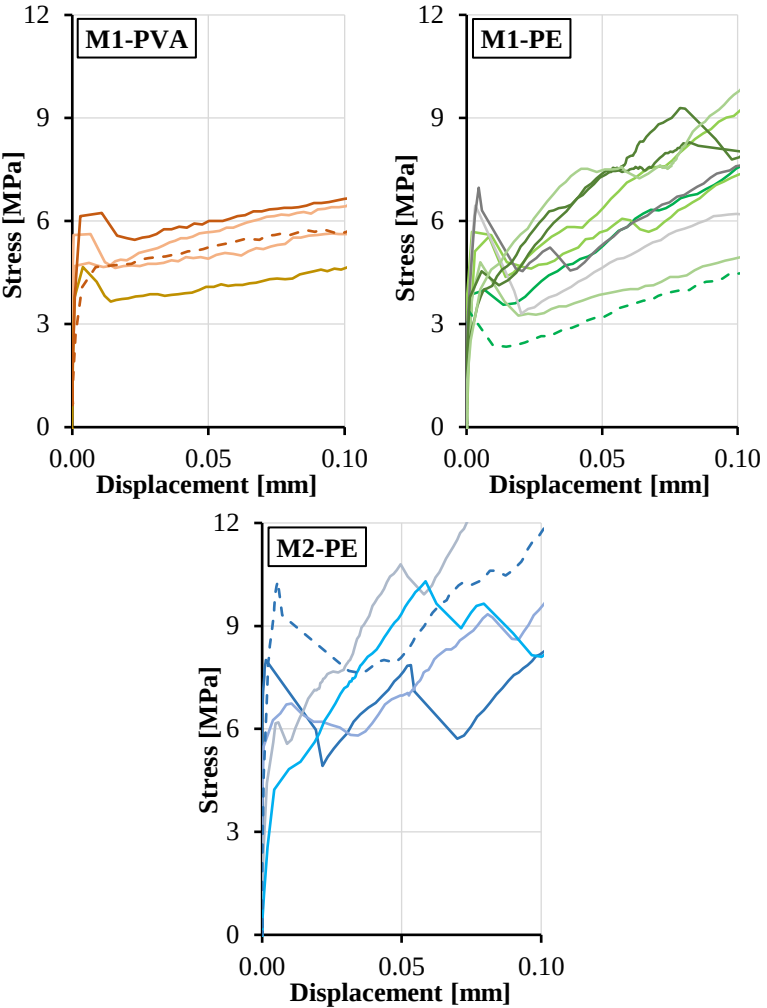


Fig. 4— Initial portions of the quasi-static stress-displacement curves of notched SHCC specimens

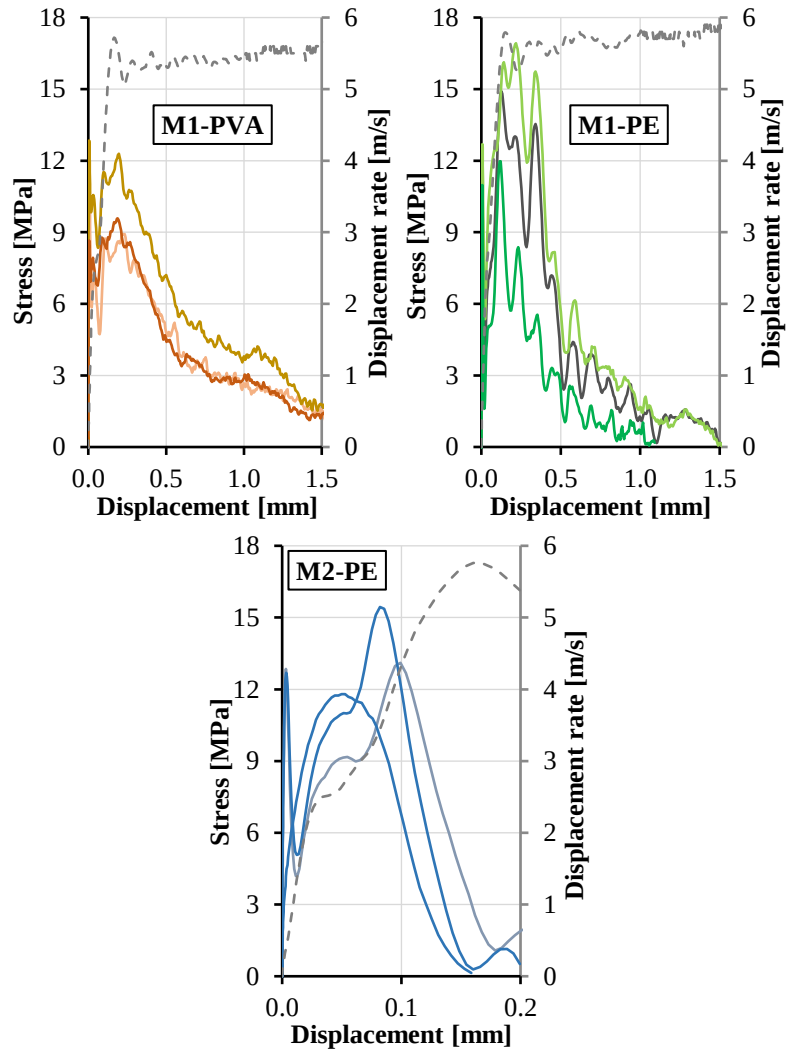


Fig. 5— Stress-displacement and displacement rate-displacement curves (dashed) obtained in high-speed experiments on notched SHCC samples; note different displacement axes' limits

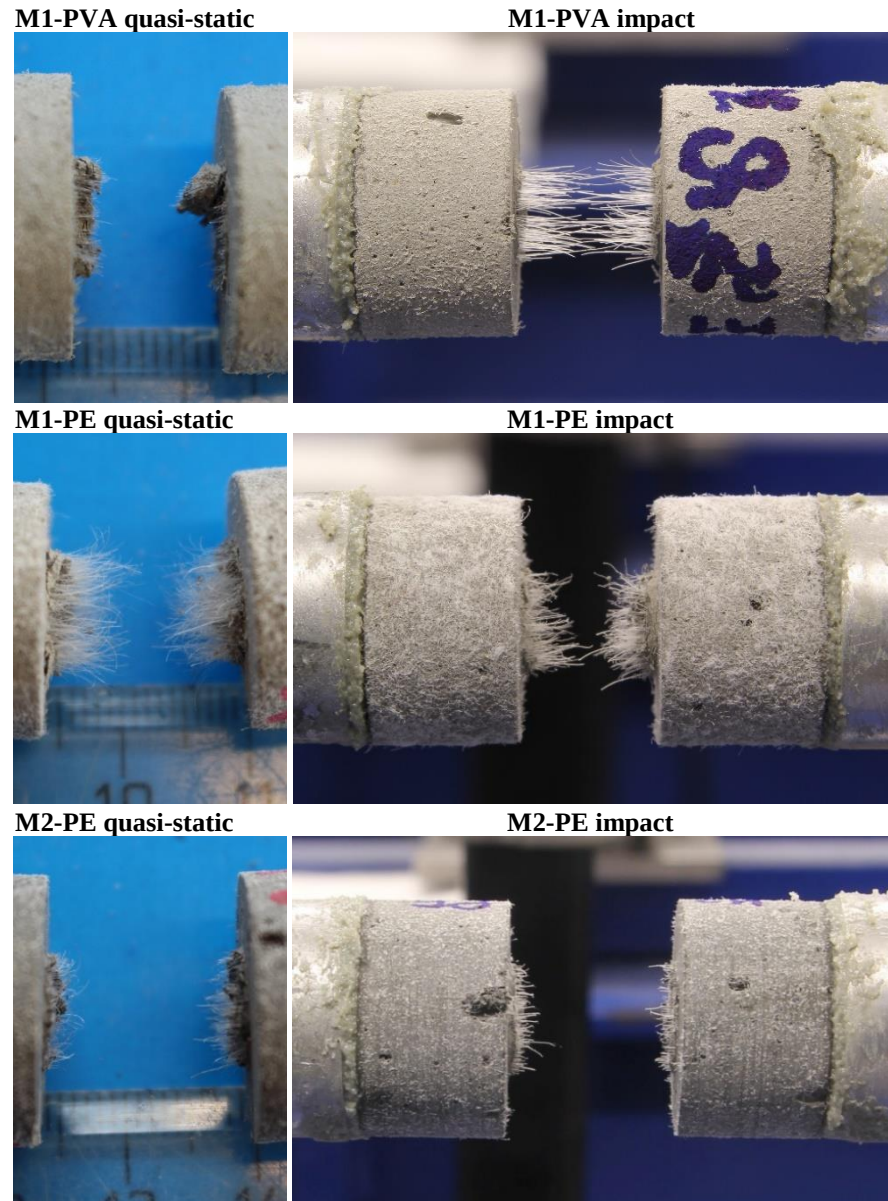


Fig. 6— State of the fracture surfaces of the tested SHCC depending on the displacement rate and material composition

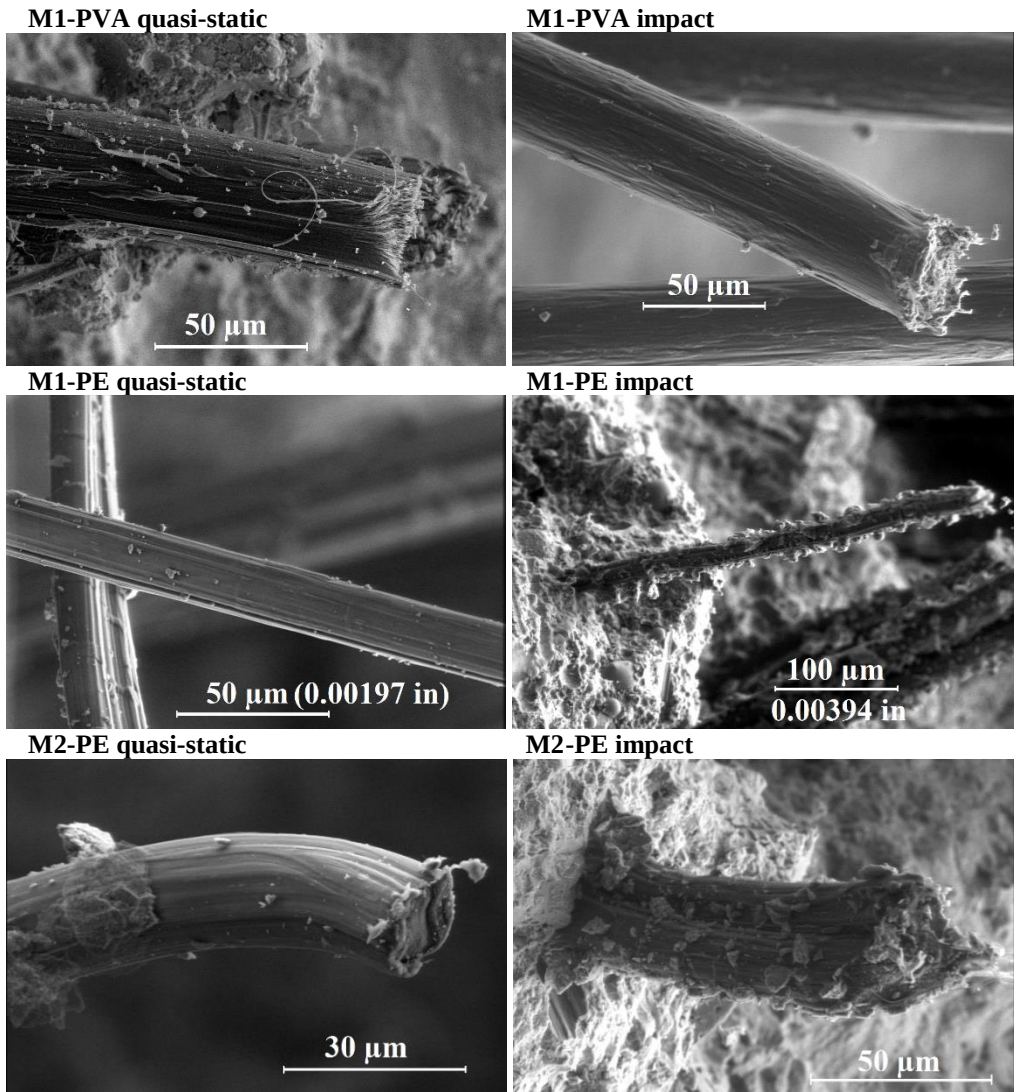


Fig. 7— ESEM images of fibers captured on the fracture surfaces of the tested SHCC depending on displacement rate and material composition

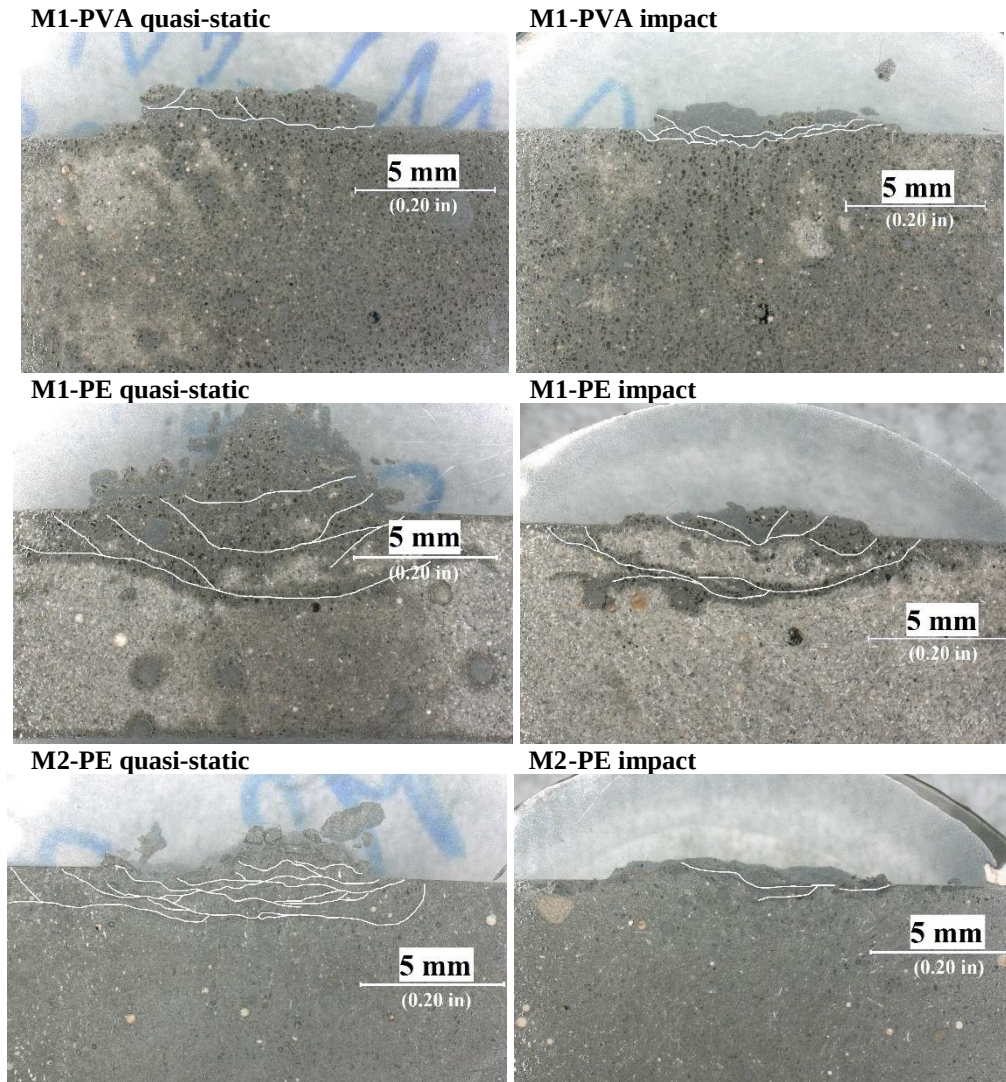


Fig. 8— Polished longitudinal slices from representative notched M1-PVA, M1-PE and M2-PE samples tested in quasi-static and impact regimes (cracks traced with white contours for highlighting their pattern)

High Strain Rate Properties of CFRP Sheets Surface Bonded to Concrete

Jonathan Harman, Emmanuella O. Atunbi, Alan Lloyd

Synopsis: Many common building materials, such as concrete and steel, are understood to experience a change in apparent material properties under high strain rates. This effect is often incorporated into impact and blast design by using dynamic increase factors (DIFs) that modify properties of the material such as strength and stiffness when subjected to high strain rates. There is currently limited guidance on dynamic properties of fiber reinforced polymer (FRP) sheets bonded to concrete. Since FRP is a common retrofit material for blast and impact load vulnerable structures, it is important to have a full understanding on the behaviour of the FRP material and of the composite action between the FRP sheet and the substrate it is bonded to. Important parameters for blast and impact resistant design of reinforced concrete structures retrofitted with surface bonded FRP include dynamic measures of debonding strain, development length, and bond stress. This paper presents the results of an experimental program measuring the dynamic properties of carbon fiber reinforced polymer (CFRP) sheets bonded to concrete under impact induced high strain rates.

A series of rectangular concrete prisms were cast and fitted with surface bonded CFRP sheets to facilitate pull-out shear tests that directly measure the FRP to concrete bond. The bonded length of the CFRP sheet was variable with three different lengths explored. A series of static tests have been conducted to measure the strain fields on the FRP sheets under load up to failure. These strain fields, which were measured with digital image correlation techniques, were used to determine development length, bond stress, and ultimate strain of the FRP sheet prior to debonding. A companion set of prisms have also been cast and will be tested under impact loading to explore the same properties at high strain rates of around 1 s⁻¹. Initial test results indicate a potential increase in both ultimate strain and bond stress, and a decrease in development length under high strain rates. The results of the larger study will be compiled and, when compared with the static companion set, be used to propose DIFs for FRP sheets bonded to concrete for use in design in high strain rate applications.

However, the main constitutive phases of SHCC, i.e. matrix, fibers and interphase between them, are highly rate sensitive. Depending on the SHCC composition, the increase in loading rates can negatively alter the balanced micromechanical interactions, leading to a pronounced reduction in strain capacity. Thus, there is need for a detailed investigation of the strain rate sensitivity of SHCC at different levels of observation for enabling a targeted material design with respect to high loading rates.

The crack opening behavior is an essential material parameter for SHCC, since it defines to a large extent the tensile properties of the composite. In the paper at hand, the rate effects on the crack opening and fracture behavior of SHCC are analyzed based on quasi-static and impact tensile tests on notched specimens made of three different types of SHCC. Two SHCC consisted of a normal-strength cementitious matrix and were reinforced with polyvinyl-alcohol (PVA) and ultra-high molecular weight polyethylene (UHMWPE) fibers, respectively. The third type consisted of a high-strength cementitious matrix and UHMWPE fibers. The dynamic tests were performed in a split Hopkinson tension bar and enabled an accurate description of the crack opening behavior in terms of force-displacement relationships at displacement rates of up to 6 m/s (19.7 ft/s).

Keywords: FRP sheet, CFRP, surface bonded; high strain rate

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INTRODUCTION

Fibre reinforced polymer (FRP) sheets are commonly used to reinforce and retrofit ageing concrete structures [1]. These FRP sheets are made of a variety of materials, including aramid, glass, and carbon [2], the focus of this study. There are a variety of reasons why a concrete structure may need to be reinforced with FRP.

A major contributor to the need for FRP reinforcement is the corrosion of reinforcing steel. Over time the corrosion of reinforcing steel results in a smaller cross section of steel and the steel to concrete bond deteriorates. This leads to a lower ultimate capacity and potential sudden failure. This corrosion is mainly caused by poor design of the concrete structure, including inadequate cover, as well as poor quality of the concrete itself. Additionally, de-icing salts and exposure to other chlorides speed up the corrosion process [3]. As roads, and bridges, are seeing higher traffic flow than ever before, these structures are deteriorating at an even higher rate and require continual maintenance [1].

Carbon Fibre Reinforced Polymer (CFRP) sheets can increase the stiffness and capacity of reinforced concrete (RC) structures, though often at the cost of brittle failures [4]. The most common method of failure for the various types of FRP is delamination where the FRP prematurely peels off due to a failure in the bond between the FRP and the substrate material [5].

CFRP has many benefits over other reinforcing materials, as well as many disadvantages. The main advantages CFRP offers are its high strength to weight ratio, non-susceptibility to creep, and its elastic properties up until failure. The brittleness of the material is one of its disadvantages [6].

Surface mounted strain gauges have often been used in experimental testing to measure response of FRP. Strain gauges, however, may be unreliable and can be inaccurate [7]. These inaccuracies are partly due to only measuring very localized strains, and when needing to measure across an entire field, the strains are an average of various points along the length of the field. The surface of CFRP is also not ideal for the use of strain gauges as it is very non-uniform. Other, more uniform materials may provide a better surface to attach strain gauges to, as the adhesive does not bond well when there are ridges and inconsistencies [7].

Digital Image Correlation (DIC) offers an alternative to strain gauges and can capture an entire field of view, as opposed to a single point that the traditional strain gauges offer. Measuring the strains over an area is a much more accurate method to obtain valuable data than averaging between a number of known points. It is not only more accurate than contact gauges, it is also an easier process. Where strain gauges require a complete, level bond to be accurate, DIC requires no externally bonded materials to the surface and uses a speckle pattern painted onto the material can be tracked using optical data measurement without the need of contact with the specimen [7].

The dynamic increase factor (DIF) is defined as the ratio of the material strength at a given strain rate, by the static strength of the same material [8], the tensile strength will be the focus of this paper. When rates of approximately 1s^{-1} are achieved, the DIF curve, for concrete in tension, drastically increases the slope as the strain rates increase [8]. Currently, there is much more research data available focusing on the compressive response of concrete than the tensile response [9]. The aim of this research paper is to calculate the DIF when strain rates of 1s^{-1} are achieved by using the strain data obtained by DIC as well as the load data. The failure modes viewed in this experiment will also be compared to that of the static experiments conducted by Atunbi [10].

LITERATURE REVIEW

The use of CFRP sheets has become a common method to retrofit old, degrading structures in the past few decades. The need for these externally bonded CFRP sheets has increased due to demand on structure members, such as bridge piers, dramatically increasing from heavier traffic flow, as well as from continual degradation of the concrete members [11].

Material and Design Properties

CFRP has been used to strengthen the flexural capacity as well as the shear capacity of beams, as well as column strengthening by means of completely enveloping the column. CFRP has been found to be the ideal solution to increasing the structural capacity of these aging, failing members due to their strength, ease of application, and low maintenance [2].

Externally bonded CFRP sheets tend to have a brittle failure mechanism. This is when the ductility has been reduced, resulting in a sudden failure. The brittle CFRP failures tend to be caused by debonding at the ends of the reinforcing. In contrast, when looking at internally mounted FRP bars, the FRP bars provide a more ductile failure and fail due to crushing of the concrete, which tends to be more desirable for design. Both methods, however, provide an increase in overall strength using equivalent amounts of reinforcing material [12].

Previous studies performed

Concrete prisms, tested in a double lap shear pull-out test configuration, with surface bonded CFRP strips on opposite sides were statically loaded by Atunbi [10]. In this experiment, the researcher found that the failure mechanism was due to the concrete in tension, as opposed to the CFRP or adhesive failing. This failure was caused by a wedge crack in the concrete, described as a tension crack at the critical end of the CFRP sheet. Three CFRP bonded lengths were tested by Atunbi [10]: 160mm, 240mm, and 350mm. It was found that when increasing the bonded length, there was an insignificant increase in the bond stress of the members. This conclusion fits well with other research conducted in the field [11] [13]. Atunbi's experiment also confirmed that use of DIC to measure strain was accurate when compared to the more traditional strain gauges used.

Comparison studies between static and dynamic loading on concrete members have been conducted for many decades. Under dynamic loading conditions, concrete is known to have a higher ultimate strength under compressive loads, and an even larger increase in strength in tensile loading [14]. Currently, the Canadian blast code CSA S850-12 [15] uses a Dynamic Increase Factor (DIF) of 1.1 under compression, while the CEB-FIB Model Code for Concrete Structures 2010 [16] and the UFC 3-340-02 Structures to Resist the Effects of Accidental Explosions [17] both use a DIF of 1.12 for strain rates of 1s^{-1} . For flexural design, these values are increased to 1.2 for the Canadian blast code, whereas the European and the US standards both use 1.19. The Canadian blast code [15] also has a DIF of 1.0 for FRP under flexural loading conditions.

When examining the experiments ran by Atunbi [10], as described previously, the failures were due to the concrete failing in tension. This explains why there is no DIF, for any FRP material, currently in the design codes considered for this research [15] [16] [17].

Huo et al. [13] found that, when performing a three-point bending test on their concrete specimens reinforced with CFRP sheets, the effective bond length of the CFRP sheets were shorter when the load was applied under impact conditions, as opposed to static. The effective bond length, as described by Huo et al. [13], is the distance from the peak strain to where the measured strain is lower than 5% of the peak strain. This, however, as mentioned in their paper, is contradicted in other research. A direct shear test on two steel members connected using CFRP sheets was carried out by Al-Zubaidy et al. [11], where it was found that increasing the loading rates did not have the same, substantial, effect on the effective bond length as what was found by Huo et al. [13].

EXPERIMENTAL PROGRAM

The experimental program conducted consisted of a total of twenty specimens. These twenty specimens were split amongst three different bonded lengths. Only three from each bonded length, however, will be considered in this paper. These specimens were tested in a double tab shear pull test on the CFRP to concrete bond. This section of the paper will detail the specimens used, experimental setup, and the method for capturing all data obtained.

Specimen preparation

The specimens used in this experiment were prepared in conjunction with the static loading experiments conducted by Atunbi [10] based on the experimental procedure detailed in Annex N of the CSA S806-12 standard [18]. The researcher cast the concrete prisms, with dimensions of 500 mm in length and a cross-section of 150 mm by 150 mm, using ready-mix concrete. Each prism was cast with a 20M steel reinforcing bar embedded into the centre. These reinforcing bars extended out the ends of the concrete and were used to hold the specimen, and load cells in place during tests [10]. The naming convention for the specimens used in the experiments discussed in this paper uses two distinguishing identifiers. The naming convention used the following LX-# format, where the “X” denotes the bonded length being either 160 mm, 240 mm, or 350 mm. The “#” refers to the specimen number in the bonded length group. The CFRP used in this experiment was the same product used by Atunbi [10], MasterBrace FIB 300-50CFS [19]. This product is comprised of unidirectional carbon fibres with a thickness of 0.165 mm. The adhesive that bound the CFRP to the concrete face was also manufactured by Master Brace, the adhesive was a combination of a primer, putty and a saturant [10].

To evaluate the compressive strength of the concrete used, standard cylinders of dimensions 100 mm in diameter, and 200 mm in height were tested. The compressive strength was found to be 32.3 MPa at the time of static testing (279 days) [10]. The remaining cylinders were then tested at the time of testing for the experiments in this paper (27 months), where the compressive strength was found to be 29.4 MPa. The MasterBrace FIB 300-50CFS CFRP product [19] has a tensile strength, elastic modulus, and ultimate strain of 642 MPa, 37.8 GPa, and 1.9 percent respectively. These values were measured from composite fibre and polymer matrix coupons that were representative of the composite CFRP strips bonded to the concrete specimens.

Each specimen consisted of two concrete prisms, which were set with a 3 mm gap between the two prisms filled with plaster with a plastic sheet against each prism to reduce any bonds between the prisms. As per Annex N of the CSA S806-12 standard [18] the specimens were cleaned of any dirt or debris to allow the best possible bond between the CFRP and concrete as possible. After the surface cleaning, primer and putty were applied following the manufacturers’ specifications, which creates a smoother surface for the bonding. Once the concrete surface was properly prepared, the CFRP sheets were cut into their respective lengths, and the resin was infused into the CFRP before being applied. Finally, another layer of resin was applied after the CFRP sheets were set [18].

Test set-up

The test apparatus, as shown in Figure 1, supported the concrete specimens between the two posts shown and rotate about the hinge on the left post.

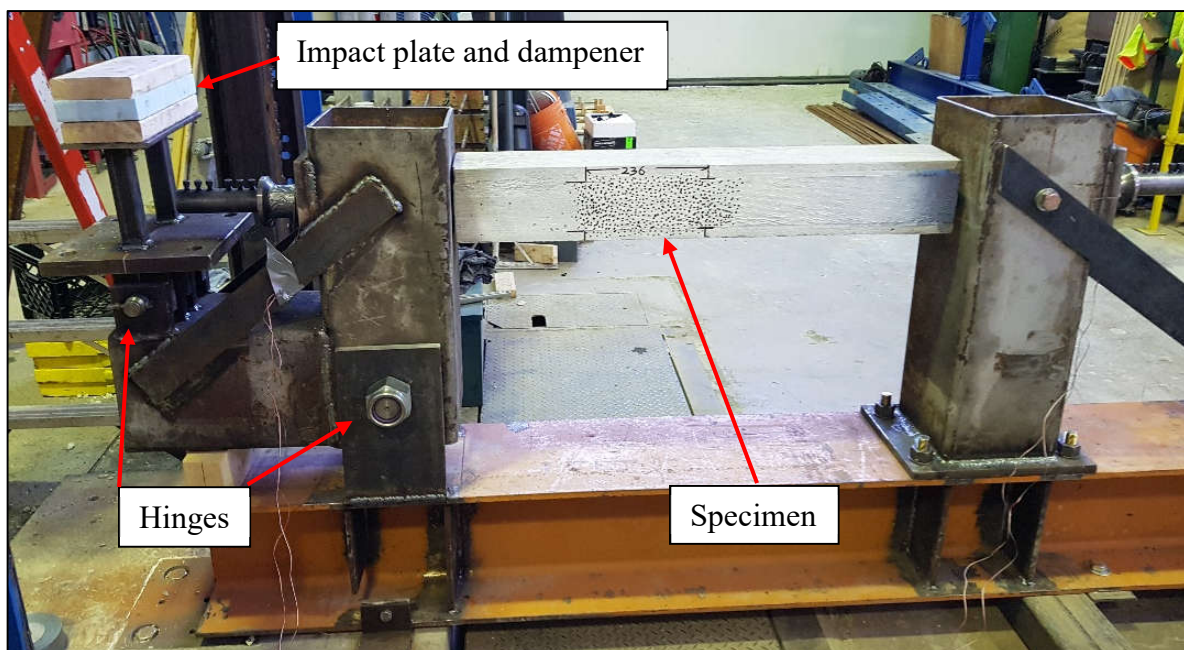


Figure 1 — Test apparatus.

As can be seen in Figure 1, the concrete specimen was placed between the two posts, supported by the rebar. The reinforcing steel extending from the prisms fits through holes in both posts and secured on the outside with rebar couplers. Once the concrete member was placed in the apparatus, the load cells were placed on either end of the specimen, with the rebar fitting through the load cells, and the high-speed camera put into position. This position was determined upon setup as lighting and safety for the equipment were both the main concerns of the camera position. A large mass drop tower was used to load one end of the prisms. The mass was dropped onto the impact plate and the hinge was employed to transfer the vertical impact force through the horizontal specimen. A wood and expanded polystyrene layup was used as an impact damper on the impact plate. This damper allowed the force transfer to slow to a reasonable level to ensure FRP strain rates were approximately equal to the goal of 1s^{-1} .

Before testing could occur, the concrete prisms were prepared for optical measurements with DIC. This was done by choosing one of the prism's faces with the CFRP and applying a thin coat of white spray paint. Once dry, large black dots were applied to create a high contrast speckle pattern. This paint and speckle pattern allowed the DIC software to detect and track deformations, which were used to calculate the strain fields. Figure 2 and Figure 3 show a typical test specimen before and after painting.

Four test specimens for each of the three bond lengths (160 mm, 240 mm, and 350 mm) were tested as static loading control specimens [10]. A further eight 160 mm bonded length specimens, and six of both 240 mm and 350 mm bonded length specimens were tested under impact loads. This paper presents results from three impact tests from each group.



Figure 2 — Typical test specimen (before painting).



Figure 3 — Typical test specimen (after painting)

Data acquisition

During testing, there were two data acquisition systems in place. The first system measured the axial force in the reinforcement. Two load cells were used for testing, these are designed as cylindrical with hollowed centres, which the rebar could pass through, along with a rebar coupler pressed on the outside to hold in place. These load cells calculate the force using two quarter bridge strain gauges on each load cell, placed on opposite sides of the test specimen. To calculate the total force, the two strain gauges in each load cell were averaged and a calibrated relationship between measured strain and applied load was applied. For the load cells to accurately measure the peak of the impact, a sample rate of 50000 hz was used. Calibrating the load cells was completed using a 250 kN Instron universal testing machine.

Other data was acquired from a high-speed camera. To achieve a balance between high framerate, and high resolution, a 16000 fps recording rate was used. Specimens were cleared of any dirt and debris before a light coating of white spray paint was applied. This provided a uniform background that contrasted against the black dot speckle pattern applied using a black permanent marker after the painting. The camera was placed at 4.5 m from the specimen face. At this distance, it was both safe from any debris that may have damaged the camera, as well as being able to capture the entire area of interest while keeping an adequate resolution for DIC. Data was recorded for a total of four seconds, with 50 % of the data occurring before the recording was triggered and the remainder after. This was to ensure that if the researcher either triggered the camera slightly early, or late, the impact would be captured within a reasonable range. Once the camera was placed in position, level and perpendicular to the test specimen, small marks were placed on the specimen, close to the edge of the field of vision of the camera. The distance between these marks was measured, recorded, and written on one of the test specimen faces. This distance was used to calibrate the DIC measurements during the analysis of the experiments.

The software used for the DIC calculations was VIC 2D [20]. This software takes each frame and tracks the changes in location of the previously mentioned black dots on the test specimen. For DIC to accurately calculate the strains, an initial reference image was required. The reference image used in each of the experiments was one of the frames just before the impact.

EXPERIMENTAL RESULTS

Following each experiment, the failure of the specimen was observed and documented. As expected, debonding began at the joint between the two concrete prisms, referred to as the critical point, which extended outwards. With the statically loaded specimens, tested by Atunbi [10], the failure was due to the concrete failing in tension. With these, the CFRP peeled back, removing a layer of concrete with it, indicating that the adhesive fully bonded the CFRP to the concrete face. The dynamically loaded concrete prisms also failed by means of tension in the concrete, however, it was not as consistent. Where the CFRP still had a full layer of concrete attached to it after debonding, some specimens only had small parts of concrete still attached, whereas others had a higher percentage. This means that these

dynamically loaded specimens tended to at times fail due to the capacity of the bonding agent, whereas others had the concrete failing in tension. Far more specimens failed due to the concrete failing in tension than the adhesive bond failing; however, they were never as consistent or uniform of a failure as those tested under static conditions. Figure 4 shows the variation in failures, from a high percentage of adhesion failing to a very low percentage of adhesion failing, as well as a photo of a typical failure from the static loading scenario where it fails due to the concrete.



a) Dynamic test with mainly concrete failing.



b) Dynamic test with mainly adhesive failing.



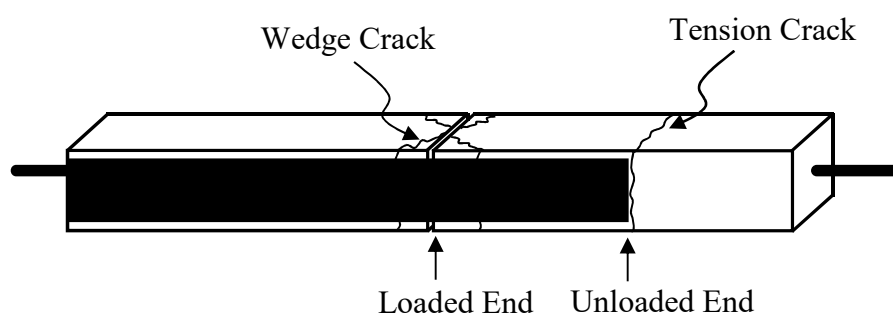
c) Typical static test with concrete failing.

Figure 4 — Differing failure modes between dynamic tests and a typical static test.

As mentioned above, the CFRP began debonding at the critical point. This occurs due to a wedge crack, also referred to as a corner crack, forming at the critical point. These wedge cracks are believed to be formed from tension cracks occurring near the prism end and breaking off. An example of a prism with typical wedge cracks, along with a depiction of a typical wedge crack formed from the static testing, can be seen in Figure 5.



a) Wedge cracks formed after dynamic testing.



b) Depiction of wedge cracks from typical static testing.

Figure 5 — Typical wedge cracks.

Further tension cracks appeared on the loaded end of the specimens. This contrasts with what was found in the static tests Atunbi [10] conducted. The researcher in these tests found that the tension crack was found on the unloaded side. Additionally, Atunbi [10] found that these tension cracks were not present on the specimens with a bonded CFRP length of 350 mm. In the experiments conducted for this experiment, tension cracks were always present on the loaded end of all specimens, regardless of bonded CFRP length. The two shorter bonded lengths, 160 mm and 240 mm, however, always cracked at or slightly after the end of the bonded CFRP. The average distance passed the CFRP cut-off point was 26 mm and 16 mm, respectively. This was not the case for the specimens with a bonded length of 350 mm, where the tension crack occurred 220 mm from the critical point, on average.

Load cell data

The ultimate loads for each of the nine specimens, distributed among the various bonded lengths examined, is summarized in Table 1. This table shows that the average ultimate force for the 160 mm, 240 mm, and the 350 mm bonded length specimens are as follows: 72.7 kN, 65.1 kN, and 76.8 kN, respectively. This shows that there was a drop in the average ultimate force between the 160 mm and the 240 mm bonded specimens.

The load, as discussed in the experimental program chapter of this paper, was recorded using two independent load cells, each equipped with two strain gauges placed at 180 degrees of each other. The values obtained from the two strain gauges, and the two load cells were averaged to give the final load over the period of the impact. Table 2 shows the peak load for each individual specimen that is being considered in this paper along with the average peak load for all three of the bonded lengths.

The average loads found during the dynamic tests appear to be much more varied than those found during the static testing. With the static testing, the average peak loads found have a maximum variability of 4.4 kN, which is between the L160 and L240 bonded length groups. This variability is increased drastically when looking at the dynamic testing group, where the difference is increased to 11.7 kN. There are many factors that could lead to this drastic increase in variation, as there is a much higher level of control of the strain rate during static testing as opposed to dynamically. Figure 6, Figure 7, and Figure 8 show the force versus time graphs for L160-2, L240-3, and L350-1, respectively. Each graph shows a rapid increase in force during the elastic region, with a very short to no plastic region, before debonding.

Figure 8, L350-1, shows the longest plastic region before failure, whereas L240-6 shows a very sudden failure. Each graph has the individual calculated load from both independent load cells along with the averaged force between the two with four points marked out, corresponding with the points that will be examined further on in this paper.

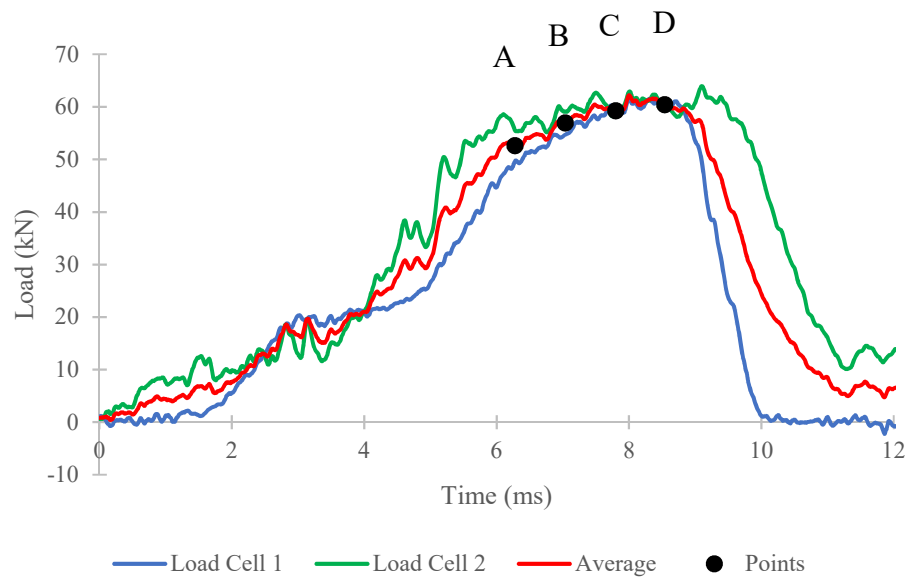
Table 1—Ultimate loads of specimens.

L160			L240			L350		
P (kN)			P (kN)			P (kN)		
Specimen	Load	Avg.	Specimen	Load	Avg.	Specimen	Load	Avg.
L160-1	76.0		L240-1	57.2		L350-1	59.6	
L160-2	62.7	72.7	L240-2	67.0	65.1	L350-2	84.4	76.8
L160-3	79.2		L240-3	70.9		L350-3	86.4	

Table 2—Maximum recorded force compared to maximum force from static tests.

Specimen Dynamic	Max. Force (kN) Dynamic	Avg. Force (kN) Dynamic	Avg. Force (kN) Static*	DIF
L160-1	76.0	72.7	39.0	1.86
L160-2	63.0			
L160-3	79.2			
L240-1	57.2	65.1	43.4	1.50
L240-2	67.0			
L240-3	70.9			
L350-1	59.6	76.8	40.0	1.92
L350-2	84.4			
L350-3	86.4			

*Static loads taken from data obtained by Atunbi [10].


Figure 6 — L160-2 force vs. time graph.

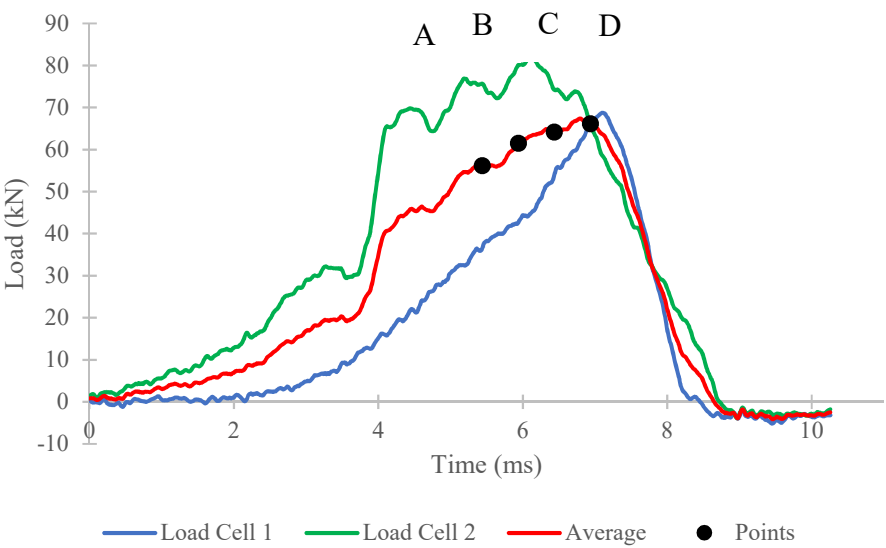


Figure 7 — L240-3 force vs. time graph.

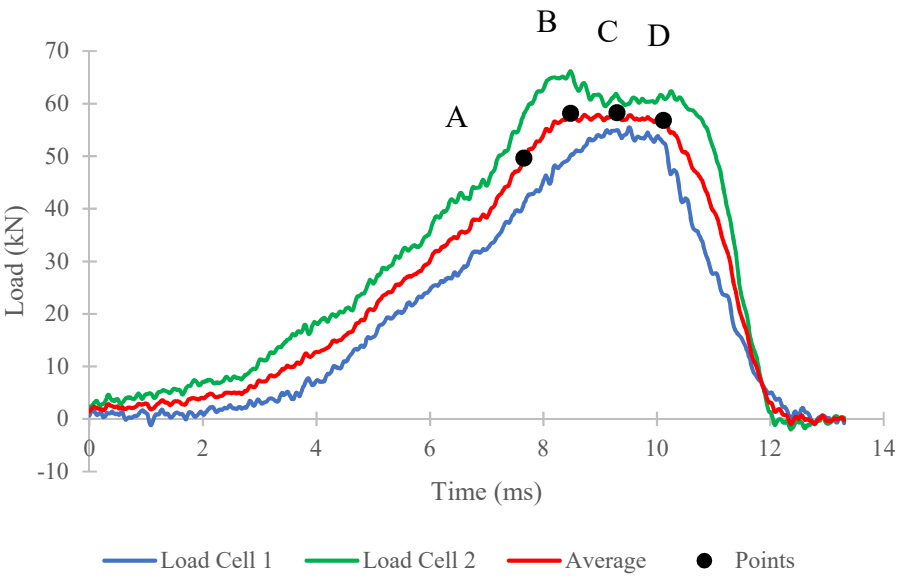


Figure 8 — L350-1 force vs. time graph.

Strain data

Each of the specimens were also analyzed using the DIC software. This software takes each individual frame from the video file and tracks the movement of the markers placed on the specimen to detect the strains across the field of vision. Once the debonding occurred at the critical point, the strain readings reached a plateau and stayed relatively constant at that point. The debonding, as described, began at the critical point and propagated outwards, this is visible in the strain graphs shown in Figure 6, Figure 7, and Figure 8. Here, the strains reached their peak values at the critical point, and then a plateau in the strain value extends further out until the CFRP has completely debonded from the concrete prism. To accompany the graphs showing the strain across the length of the beam, and the strain contour graphs are Figure 9 — L160-2 strain time history., Figure 10, and Figure 11. These figures show when each of the snapshots in time were taken, with respect to the peak strain by placing a mark at each location.

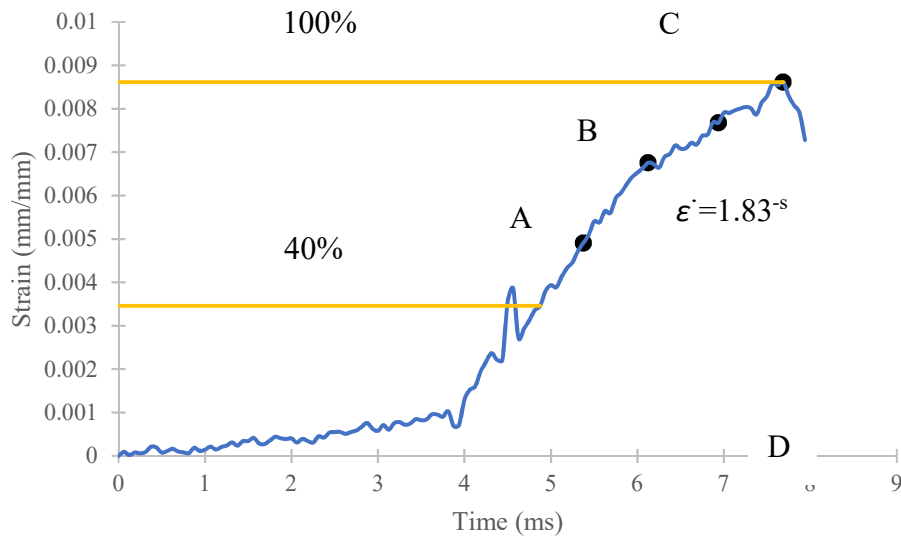


Figure 9 — L160-2 strain time history.

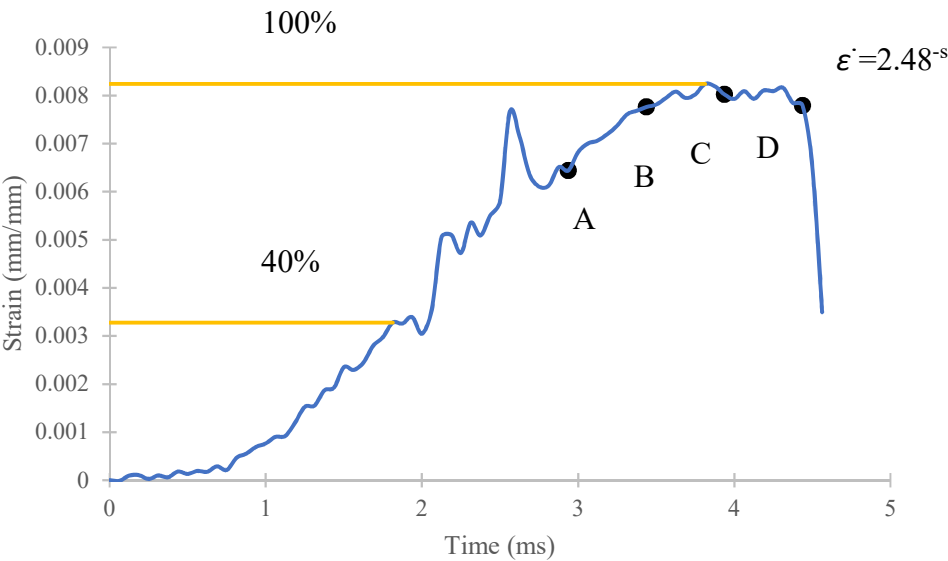


Figure 10 — L240-3 strain time history.

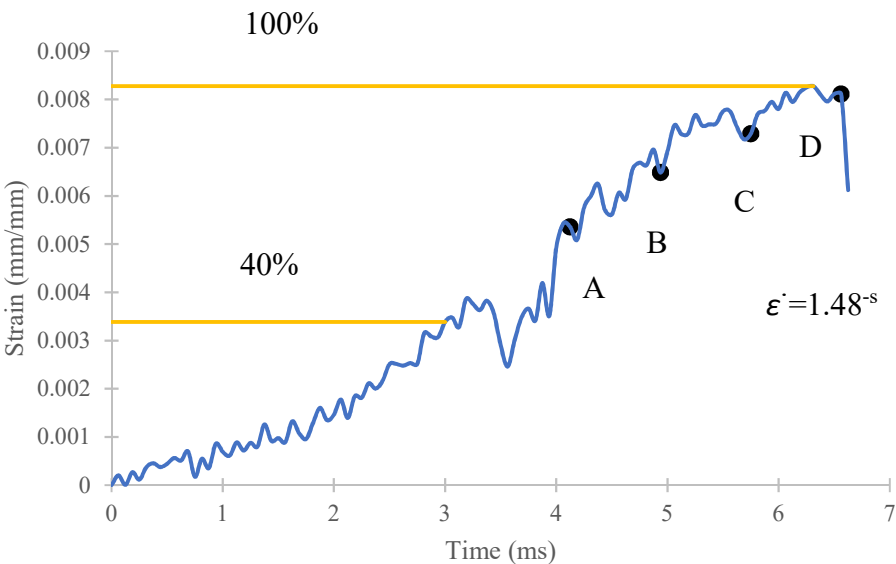


Figure 11 — L350-1 strain time history.

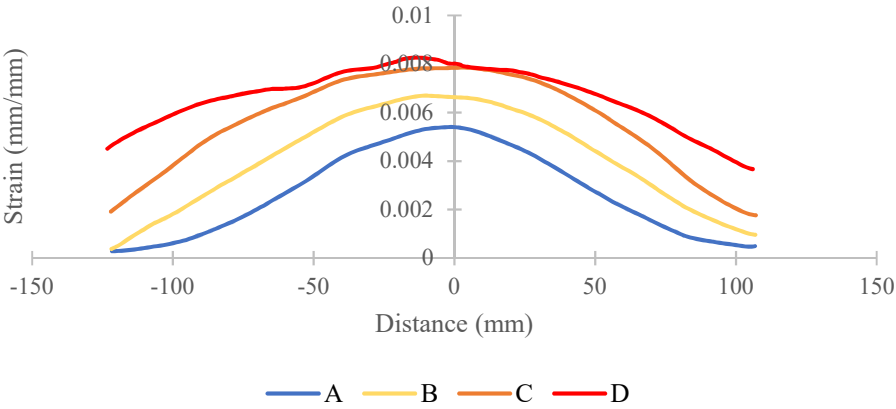


Figure 12 — L160-2 strain field at four distinct times.

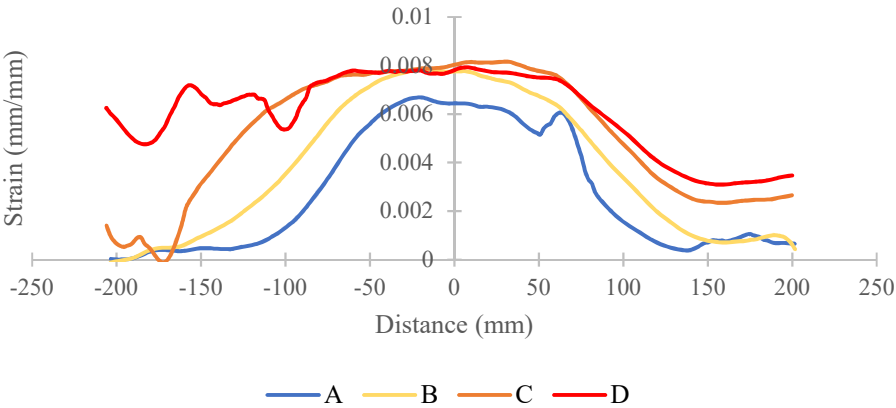


Figure 13 — L240-3 strain field at four distinct times.

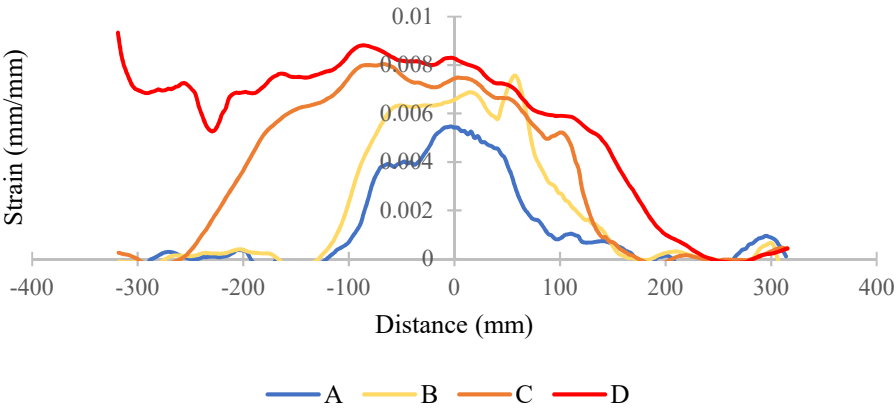


Figure 14 — L350-1 strain field at four distinct times.

From Figure 12, Figure 13, and Figure 14, the maximum strains can be seen for specimens L160-2, L240-3, and L350-1. These graphs were created in the software VIC 2D and used a single inspection line across the area of interest of the relevant specimen. When compared to the statically loaded specimens tested by Atunbi [10], these values are much higher. This was expected as materials loaded dynamically tend to have a higher ultimate capacity than those loaded statically. The plateaus beginning at the critical point and extending outwards tend to form very rapidly near the end of the impact. The strain versus time graphs show that the strains continue to grow throughout the impact, and slowly become more uniform until it reaches the peak strain at which time the remaining CFRP reaches that peak in a short period of time.

As previously discussed, materials loaded under dynamic conditions may have a higher ultimate capacity than those loaded statically, known as DIF. Table 3 shows the average maximum strains for each specimen group for the tests performed for this paper, against the statically loaded specimens by Atunbi [10]. The average maximum strains for the dynamically loaded group in the L160, L240, and L350 bonded length groups are 0.82 %, 0.86 %, and 1.14 %, respectively. These maximum strain values are higher than those reported from the static groups, this was expected, however, the magnitude is greater than the expectation. The DIF values calculated for the L160, L240, and L350 are 1.82, 1.51, and 2.00, respectively. As mentioned, these DIF values are much higher than what was expected, as reported by the relevant codes [15, 17, 16], particularly the L350 bonded length group. This, however, is supported by the data collected from the load cells. Table 3 shows both the DIF calculated from the strain data, as well as the DIF calculated from the load data, for each of the three specimen groups. The DIF values calculated are very similar. These two measurement systems were fully independent of each other and produced similar DIF values.

Table 3—Average maximum strain comparison between dynamic and static tests.

Specimen Group	Dynamic Maximum Strain (%)	Static Maximum Strain (%)	DIF (Strain Data)	DIF (Load Cells)
L160	0.82	0.45	1.82	1.86
L240	0.86	0.57	1.51	1.50
L350	1.14	0.57	2.00	1.92

CONCLUSIONS AND RECOMMENDATIONS

This paper explored the results of the experiments investigating the bond between CFRP and concrete when placed in direct tension under dynamic loading conditions. Two load cells were used in this experiment, which were placed on either end of the specimen and averaged to obtain accurate data over the course of the experiments. DIC was used to measure the strains across the specimens. DIC is non-invasive and can measure strains across an entire field of view, which is a considerable improvement over the commonly used strain gauges. Together, using these two methods of obtaining data, the following conclusions and recommendations can be drawn from:

1. The use of DIC has shown to be a great improvement over the use of strain gauges. There were difficulties when using the software that is used to track the speckle pattern over the number of frames; however, further research and use of this may discover more effective methods of mitigating these difficulties.
2. The failure method observed in this experiment was not solely due to the concrete failing in tension as expected. The failure was, in part, due to the adhesive failing, resulting in very little concrete still attached to the CFRP post-debonding. As seen in the experiment run by Atunbi [10], the concrete was still attached to the back of the CFRP, showing that the concrete failed in tension. It is believed that, as the DIF of the concrete in tension was higher than expected, the ultimate capacity of the concrete increased beyond the strength of the adhesive bond in some of the specimens. Further experimentation with different adhesives is recommended, as this adhesive may not be forming the complete bond as was expected and assumed.
3. Much higher DIF values were found than expected when examining both the load data and strain data. These DIFs ranged from 1.50 to 2.00, which differs greatly from the 1.19 to 1.20 for concrete in flexure, and the 1.0 for FRP in flexure, as reported by the relevant codes [15, 16, 17]. Further dynamic tensile tests should be conducted with a larger sample size to find a more precise DIF for CFRP externally bonded to concrete.
4. The strain plateaus observed by Atunbi [10] were observed in this experiment as well. These plateaus formed, starting at the critical point and propagating outwards, as the concrete to CFRP bond was broken, and continued along the length of the specimen during the propagation of debonding.
5. There was a small difference in ultimate bond strength between the three bonded lengths, 160 mm, 240 mm, and 350 mm. These values were 72.7 kN, 65.1 kN and 76.8 kN, respectively. The 240 mm bonded length specimens had an average ultimate capacity lower than either the 160 mm or 350 mm bonded groups. This is reflected in the calculated DIF values from both the load cells and strain data of 1.5, which was lower than the other specimen groups. Further tests with a larger sample size are recommended as this was an unexpected result and may have been due to an outlier specimen.
6. Tension cracks were found on the loaded end of the specimens for all experiments. These tension cracks were found, typically, at or near the end of the bonded length for the 160 mm, and 240 mm specimens. As for the 350 mm specimens, the tension crack was found at an average of 220 mm along the length of the CFRP, measured from the critical point. This differs from the statically loaded specimens tested by Atunbi [10], where the tension cracks always formed on the unloaded side, though no cracks were found on the 350 mm bonded length group. The difference in tension crack location is believed to be due to the inertial effects present in dynamic testing.

Further studies where the specimens are loaded dynamically is recommended. This study has shown that the use of DIC is a useful, reliable method of measuring the strain fields under dynamic conditions, so it is suggested to continue the use of this method. Future studies should involve more specimens per bonded length group, along with multi-layered CFRP specimens. As the failure method of the specimens was expected to be solely due to the concrete failing in tension, and this was not always the case, an investigation into the reason for the adhesive failing would be essential. This investigation should include different adhesives, as well as attempting different strain rates to find at what point the adhesive begins to fail over the concrete.

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Using Steel Fibres to Increase the Projectile Impact Resistance of Cementitious Composites

Radoslav Sovják, Sebastjan Kravanja, Jan Zatloukal

Synopsis: Steel fibres in cementitious composites play a crucial role in making structures less susceptible to the damage caused by projectile impacts. A synergistic effect is achieved when steel fibres and an otherwise brittle cementitious matrix are blended together to produce a high-performance fibre-reinforced cementitious composite with enhanced ductility and strength. These composites also display strain hardening in tension, which leads to enhanced energy absorption and dissipation capacity. In this study, in-service 7.62×39 mm [0.28×1.54 in.] cartridges were used as projectiles. The muzzle velocity and weight of the projectiles were 710 m/s [2329 ft/s] and 8.04 grams [0.284 oz], respectively. Projectiles were shot with a stationary semi-automatic rifle into specimens made of high-performance fibre-reinforced cementitious composites with various fibre volume contents. Fibres used in this study were straight with a smooth surface. The aspect ratio of the fibre was 108:1 and corresponding dimensions were 14×0.13 mm [0.55×0.005 in.]. The tensile strength of the fibres was 2,800 MPa [406 ksi] and the modulus of elasticity was 210 GPa [30,458 ksi]. Owing to their exceptional mechanical properties, the fibres played a key role in controlling the response of the specimens when impacted by projectiles. The highest fibre volume content used in this study was 2% by volume; the cube compressive strength of the resulting mixture was 144 MPa [20.9 ksi]. Specimens were examined for the possible presence of spalling, scabbing, cracking, or full perforation. Depth of penetration, crater area, and crater volume were also tested. Results showed that steel fibres, due to the aforementioned synergistic effect with a cementitious matrix, notably protected specimens from erosion and significantly reduced cratering damage.

Keywords: cementitious composites, crater area, crater volume, depth of penetration, projectile impact, shear crack analysis, steel fibres

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INTRODUCTION

Concrete has traditionally been used as an engineering solution for protective structures due to its ability to withstand impact and point loads. Localized impact of projectiles is, therefore, one of the primary high-speed types of loading that must be considered when designing such structures. In a broad sense, the projectile impact can be understood as the impact of fragments or projectiles generated by the failure of high-speed rotating machines, explosions, or—last but not least—direct armed attacks. Resistance to penetration by projectiles is of importance in strategically significant buildings, nuclear power plants [1–3], and other such facilities (chemical, heavy industry) where relatively small, high-velocity objects could cause disproportionate damage. Several authors have suggested that structures should be required to withstand impact loads caused by plant-internal accidents [4]. A typical case of internal phenomenon would be the fracture of a rotary machine, which could result in turbine missiles—projectiles which travel at high velocities [5]. Such projectiles can result in severe damage to facilities and endanger the safety of personnel [6]. Such high security requirements extend beyond the framework of conventional design methodologies.

Due to its structural ability to resist impact and point loads, concrete has commonly been used as an engineering solution for use in fortifications. The improvement of concrete composition has resulted in high-performance fibre-reinforced cementitious composites (HPFRCCs). The development of various high-performance fibre-reinforced cementitious composites has shown that HPFRCCs can be potentially employed in infrastructure protection for protective structures [7]. The main motivation is usually the improvement of fresh and hardened concrete properties as well as enhancement of the durability characteristics of the material and the lifetime of the concrete structures [8]. These composites also perform strain hardening behaviour under tensile loading [9, 10]. Owing to HPFRCC's high shear resistance, strength, and ductility, it is well suited for withstanding extreme load events such as shocks and impacts. Micro fibres incorporated into cementitious composites enhance ductility and strength [11]. Short micro fibres with a diameter less than 0.2 mm [0.008 in.] may significantly reduce the damage resulting from a projectile impact. The response of a composite material to high-speed impact loading is different when compared to quasi-static loading. Most of the impact energy is absorbed and dissipated by the brittle fracture of material rather than by the deformation of the loaded member. Using fibres is essential when dealing with impact loading, because composites tend to be more brittle than under quasi-static loading conditions; increased ductility becomes a primary concern.

The enhanced material characteristics of HPFRCC result mainly from disperse fibre reinforcement. Fibres play an essential role in providing a sufficient degree of protection within fibre-reinforced cementitious composites. Therefore, percentage of fibre volume content was the main test variable for this study. A common type of in-service bullet was used to examine impact loading behaviours. Both deformable and non-deformable versions of 7.62×39 mm [0.28 × 1.54 in.] cartridges were used; impact energy was 1903 J [1404 ft lbf]. Owing to their widespread use, interaction of these bullets with emerging cementitious composites has been investigated in the framework of this study. Several studies employed an air gun to examine the phenomena of hard projectile impact [12–15]. However, to our best knowledge, studies with real in-service projectiles fired from a semi-automatic rifle under real conditions have seldom been conducted. Small-calibre projectile impact tests using truncated-ogive-nosed bullets have not often been

performed [16]. This study fills this research gap by describing the response of cementitious composite specimens to real projectile impacts in order to assess the degree of resultant damage, notably depth of penetration, crater diameter, and crater volume based on various fibre volume contents in composite samples. Both semi-infinite targets and thin plates were prepared using HPFRCC and their response to both deformable and non-deformable projectiles was examined.

TECHNICAL BACKGROUND

Cementitious composites

Ever since the emergence and development of high-strength concrete (HSC), it has been used in countless areas of construction, mainly due to its distinctive mechanical properties. However, HSC is an extremely fragile material, which is why it cannot be used to fabricate all structures. For seismically resistant elements, for example, high strength must necessarily be associated with high ductility. This has led to the development of engineered cementitious composites (ECC) [17]. ECC is a special subtype of HPFRCC with enhanced ductility and strain hardening with the formation of multiple microcracks. ECC may have a tensile strain capacity ranging from 3% to 7% (for conventional concrete, tensile strain capacity is 0.01%). Tensile stress capacity (strength) for ECC typically ranges from 4 MPa [0.580 ksi] to 6 MPa [0.870 ksi]. Compressive strength typically ranges from 30 MPa [4.35 ksi] to 80 MPa [11.6 ksi] with an ultimate compressive strain ranging from 0.4% to 0.65% (for conventional concrete, compressive capacity is 0.35%). Li et al. [18] noted that ECC can withstand large deformations, remaining ductile even under high-stress conditions. ECC can take many forms, including light-weight [19], self-compacting [20, 21], shotcrete [22], and extruded ECC [23]. ECC can be therefore defined as a cementitious composite with above-average ductility provided by one type of short micro-fibre reinforcement [Fig. 1].

HPFRCCs contain a large volume of steel fibres, a low water-to-binder ratio, high micro-silica content, and no coarse aggregates larger than 4 mm [0.16 in.]. HPFRCCs have outstanding material properties defined by high mechanical parameters (compressive strength, tensile strength, shear strength) coupled with improved tensile behaviour, so-called strain hardening, which rarely occurs within conventional normal strength concrete (NSC) containing coarse aggregates or in conventional fibre-reinforced and steel-fibre-reinforced concretes (FRC, SFRC) with macro fibres (diameters larger than 0.2 mm [0.0079 in.]). FRC exhibits better tensile behaviour than traditional concrete, but after cracking, it is no longer able to carry additional tensile stress; strain softening occurs immediately. HPFRCCs exhibit strain hardening, which means they are able to resist increased tensile force after crack formation over a significant tensile deformation range [24]. The use of ductile materials is advantageous since they are able to carry a load even after ultimate loading has been reached, allowing for the absorption and dissipation of more mechanical energy [25]. This property has been successfully used for impact absorption and dissipation of materials [26].

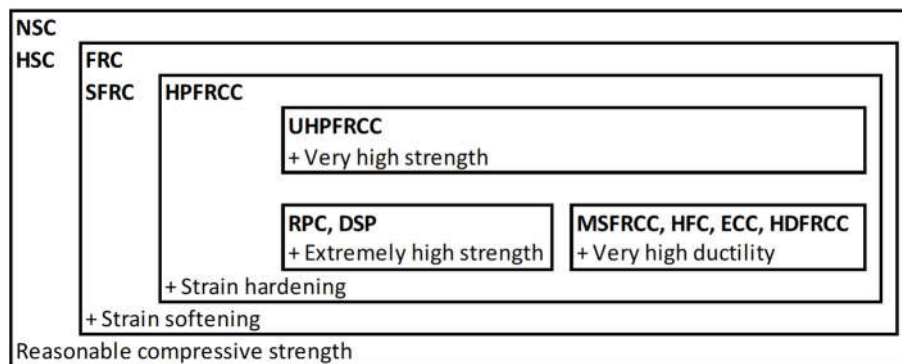


Figure 1: Differentiation of concretes and cementitious composites

HPFRCC with uniaxial compressive strength greater than 150 MPa [21.8 ksi] can be termed as Ultra-High-Performance Fibre-Reinforced Cementitious Composite (UHPFRCC). UHPFRCCs are typically characterized by compressive strengths greater than 150 MPa [21.8 ksi] and up to approximately 250 MPa [36.3 ksi] [27, 28]. Riedel et al. [29] implied that UHPFRCC is potentially suitable for improving protective structural elements. Other researchers

have also noted that UHPFRCC provides a feasible solution for protective structures due to its enhanced mechanical properties and impact resistance [30]. One of the most recent steps in the development of UHPFRCC has been the creation of reactive powder concrete (RPC) [31, 32]. RPC has a compressive strength of up to 800 MPa [116 ksi] achieved by the use of several super fine grain materials (maximum grain size of 300 μm [0.01 in.]), optimization of the grain size of the individual components, solidification of the material under pressure, heat treatment, and the addition of small steel fibres to improve toughness. One early pioneer (1980s) of ultra-high strength concrete was H. H. Bache [33]. His research focused on the size and type of aggregates in concrete and on reducing the water-to-cement ratio. The material Bache developed was referred to as densified systems with small particles (DSP). Even today, DSP is used for industrial flooring and other areas prone to cracking.

High ductility (exceeding 3% under tension) can be achieved by incorporating synthetic fibre into a cementitious paste or mortar, creating a so-called high-ductile fibre-reinforced cementitious composite (HDFRCC) [34, 35]. UHPFRCC has a very high compressive strength which exceeds 150 MPa [21.8 ksi] and ductility below 1% under tension. HDFRCC, with its high ductility, has a lower compressive strength than UHPFRCC [36]. Another interesting example may be a composite with different types of fibres, whether they are fibres with different materials or fibres with different geometries such as macro and micro fibres. The combination of different materials of fibrous reinforcements within one composite has led to the development of and research into hybrid fibre concrete (HFC) [37]. Various types of reinforcement for one composite can be achieved by means of fibres having different geometries such as a combination of micro and macro fibres [38]. Since fibres of different sizes are combined, such materials are referred to as multiscale fibre-reinforced cementitious composites (MSFRCC). Recent research suggests that a combination of the right volume of low modulus polypropylene fibres and high strength steel fibres may result in improved tensile strength, which in turn can improve resistance to penetration; an increase in the deformation characteristics is crucial for a high rate of shock absorption capacity [39, 40].

High-velocity projectile impact

The impact of a high-speed projectile with a concrete or cementitious composite is associated with local damage around the point of the impact [41, 42]. The degree of damage depends on a variety of factors related to the projectile, including mass, geometry, impact velocity, trajectory, and material properties [43]. Local damage may consist of spalling from the proximal face of the target and scabbing from the distal surface of the target along with the penetration of the bullet. The scabbing of the material from the back face is caused by the rebound of the shock wave from the back face of the protective system. For thick elements, the stress wave will reach the back of a concrete slab before the projectile. Because of concrete's low tensile strength and the tensile stresses arising from complex reflections of the stress wave, many concrete fragments are produced on the rear face of a concrete slab before a projectile perforates it [44]. The resistance of concrete structures to penetration can be improved in several ways, including engaging a steel casing, inserting an elastic absorbent pad between two concrete layers, or using FRC or HPFRCC. It is well known that conventional FRC provides good impact resistance [45]. However, many authors suggest that HPFRCCs have a much greater capacity to withstand impact and shock loads [Fig. 2] [46, 47].

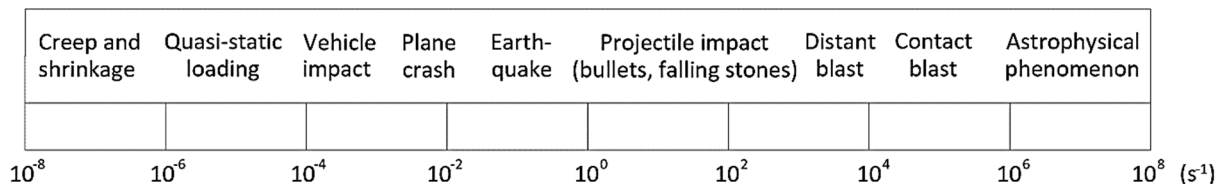


Figure 2: Various strain-rates loading conditions

Perforation of concrete by projectiles is a very complex problem because it involves compression with confinement pressure, tensile cracking, and shear stress developments. The complexity is further increased by the fact that various phenomena are occurring under various strain rates. HPFRCCs are strain sensitive, which means that there are variations in uniaxial compression or tension strength under different strain rate loading conditions. In addition, the rate sensitivity of HPFRCC varies according to fibre type, fibre volume content, and matrix type [48]. When a projectile first hits a specimen, a compression wave is generated at the proximal face of the specimen and travels to the distal