BIAXIAL BEHAVIOR OF HIGH-PERFORMANCE FIBER- REINFORCED CEMENTITIOUS COMPOSITE PLATES

by

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ABSTRACT

A total of 127 plate specimens were fabricated and tested, of various mixture proportions, fiber types, and casting methods, in order to investigate the behavior of both plain concrete and high-performance fiber-reinforced cementitious composite (HPFRCC) specimens under multi-axial loading. The majority of the test specimens were initially fabricated as larger loaf specimens, to achieve proper fiber directionality in the out-of-plane direction, and then cut and trimmed, with steel brush platens used for loading to minimize friction between the testing machine and the plate specimens. The test results indicate that HPFRCC materials can exhibit enhanced biaxial compression performance, compared to plain concrete specimens, due to passive confinement provided by fibers in the out-of-plane direction. The multi-axial behavior of HPFRCC materials obtained from the tests was further used to construct biaxial failure curves, and several modeling parameters have then been derived for nonlinear finite element analysis of HPFRCC planar members subjected to biaxial stresses.

Keyword: HPFRCC, biaxial strength, uniaxial strength, fiber, finite element

INTRODUCTION

Typical concrete materials are characterized by quasi-brittle failure modes and low tensile strength, with limited ductility. In practice, reinforcing steel is added to provide a concrete structural member with the requisite tensile and confined compression capacities in order to achieve the desired strength and ductility levels. However, many structural concrete applications can require a large amount of reinforcing steel, resulting in construction of a design that is congested, costly, or even impractical (Gefken and Ramey 1989, Katzensteiner et al. 1994, Bayasi and Gebman 2002, Parra-Montesinos et al. 2005, Sirijaroonchai et al. 2010, Lee et al. 2014).

According to Parra-Montesinos et al. (2012), high-performance fiber-reinforced cementitious composites (HPFRCCs) can be defined as advanced composite materials that exhibit a strain-hardening tensile stress-strain behavior, with multiple cracking characteristics. Therefore, HPFRCCs could potentially alleviate the problems of low ductility and/or reinforcement congestion, through their inherent ability for bond and confinement (even with a reduced amount of transverse reinforcement), while still ensuring a ductile failure mechanism. This is because HPFRCCs have relatively large shear and tensile capacities with ductile hardening behavioral characteristics, and fibers at a stable crack interface can provide passive confinement, crack control capacity, and an additional energy dissipation mechanism (Krstulovic-Opara et al. 1994, Kang et al. 2012, Lee et al. 2014, Susetyo et al. 2013).

There have been a number of research efforts directed toward practical applications of HPFRCC materials in structural concrete members, such as coupling beams and beam-column connections, as well as in plastic hinge regions of beams, columns, and structural walls (Katzensteiner et al. 1994, Filiatrault et al. 1994 & 1995, ACI Committee 544 2002, Kim and Parra-Montesinos 2003, Canbolat et al. 2005, Parra-Montesinos 2005, Parra-Montesinos et al. 2005, Shannag and Alhassan 2005, Yun et al. 2007, Quang et al. 2016). For proper and accurate analysis of such structural elements utilizing HPFRCC materials, constitutive models of HPFRCC in compression and tension must be developed, in full consideration of multiaxial stress effects.

According to the authors' literature review, many experiments have been conducted on fiber-reinforced concrete materials since the 1960s (ACI Committee 544 2002), and the multiaxial behavior of HPFRCC materials has also been investigated by various researchers

(Traina and Mansour 1991, Chern et al. 1992, Murugappan et al. 1993, Torrenti and Djebri 1995, Swaddiwudhipong and Seow 2006, Ren et al. 2008, Sirijaroonchai et al. 2010, Noori et al. 2015). However, most of these existing studies focused on the tensile and triaxial compression behavior of fiber-reinforced concrete, while there is limited experimental data on the biaxial behavior of HPFRCC (Foltz et al. 2008, Sirijaroonchai et al. 2010).

In this study, an experimental program was carried out using a customized testing apparatus to investigate the biaxial behavior of both plain concrete and HPFRCC materials, with a total of 127 plate specimens fabricated and tested. Based on the test results, biaxial failure envelope curves were developed, and several modeling parameters were determined based on the concrete plasticity model provided in a commercial finite element analysis program.

EXPERIMENTAL PROGRAM

The experimental program investigated the behavior of both plain and HPFRCC specimens under multi-axial loading. The specimens had various mixture proportions, fiber types, and casting methods. A total of 127 plate specimens were fabricated and tested; a summary of the test specimens used in this experimental program is provided in Table 1. The multi-axial response of the loaded specimens was used to construct biaxial failure curves and to inform modeling parameters for subsequent use in related nonlinear finite element models.

Materials and mixture proportions

In this testing program, two concrete mixes have been explored. The first is a mortar mix (MM), in which two different types of fiber – Spectra (polyethylene) or Dramix (hooked steel) – and three fiber volume fractions (1.0%, 1.5%, or 2.0%) were considered. Some test results for that mix have already been reported elsewhere (Foltz et al. 2008, Sirijaroonchai et al. 2010).

The other mix was developed as part of a NEES research project entitled “Innovative Applications of Damage Tolerant Fiber-Reinforced Cementitious Materials for New Earthquake-Resistant Structural Systems and Retrofit of Existing Structures.” Liao et al. (2006) initially explored and established the basic mechanical properties of six different HPFRCC mixes as part of that project. The main objective in developing such mixes was to obtain a strain-hardening, self-consolidating concrete mix with 28-day compressive strength of between 5 and 9 ksi (34.5 and 62.1 MPa). Of the six mixes initially investigated in that

previous study (Liao et al. 2006), a specific one is further explored as a focus of this current work, called NEES Mix #6 (NM6), in part because it was the most economical choice that met the other material design objectives.

Table 2 presents the proportions of each mix, by weight of cement. The MM was used in six different ways: at three different volume fractions (1.0%, 1.5%, and 2%) of two different fiber types (Spectra and Dramix). The Spectra® fibers were an ultra-high molecular weight polyethylene, a trademark of Honeywell (Honeywell International Inc., 2015), while the hooked steel fibers were Dramix® RC-80/30-BP by Bekaert Inc. (2013), made of high-strength steel. NM6 only used the hooked steel fibers. Table 3 presents a summary of the fiber properties used in this study.

All test specimens were made using ASTM Type III Portland cement and class C fly ash. The coarse aggregate used in NM6 was a crushed limestone, with a maximum aggregate size of ½ in. (13 mm) and a specific gravity of about 2.7. The fine aggregate for all mixes was #16 flint silica sand, supplied by the U.S. Silica Company; the fine aggregate for MM had an ASTM C33 gradation of 30-70, while that for NM6 had an ASTM C33 gradation of 50-70. ADVA® Cast 530 was the polycarboxylate type superplasticizer used in each concrete mixture. For NM6, a fixed amount of superplasticizer was prescribed in advance, but for MM even more superplasticizer was added when the mix proved visually to be too dry. An additional viscosity modifying admixture (VMA – RHEOMAC® VMA 362) was used in NM6, to enhance viscosity and reduce fiber segregation in the presence of relatively high water-to-cementitious material ratios (Liao et al. 2006). The water-to-cementitious-material ratios for MM and NM6 were 0.35 and 0.43, respectively.

Details of plate test specimens

For the multi-axial testing regime described herein, 60 and 67 test specimens were fabricated by an individually-cast method and a loaf-cast cut-and-trim method, respectively, all with dimensions of 5.5 x 5.5 x 1.5 in. (140 x 140 x 38 mm). This specimen size is similar to that used in historical concrete biaxial strength experiments (Kupfer et al. 1969, Liu et al. 1972, Tasuji et al. 1978, Maekawa and Okamura 1983, Yin et al. 1989, Lan and Guo 1999, Hussein and Marzouk 2000, Lee et al. 2004).

In the testing program, two series (or generations) of multi-axial tests were conducted to investigate the influence of casting method on fiber orientation, and thus on biaxial behavior, of HPFRCC materials – i.e., individually-cast plate specimens and cut-and-trimmed loaf

specimens. In first generation testing, which has been reported in part elsewhere (Foltz et al. 2008, Sirijaroonchai et al. 2010), individually-cast plate specimens were fabricated of MM materials, and large 6.5 x 6.5 x 18 in. (165 x 165 x 457 mm) “loaves” of the MM were also cast, to ensure a random orientation of the fibers (especially in the out-of-plane direction). The loaves were then cut and trimmed to the aforementioned 5.5 x 5.5 x 1.5 in. (140 x 140 x 38 mm) specimen size using a diamond precision saw. Fig. 1(a) illustrates how the loaf specimens were cut and trimmed to thin-walled plate specimens, and Fig. 1(b) shows the likely effect of casting method on fiber dispersion. Upon visual inspection during and after saw-cutting, it was clear that the fibers were well-dispersed and randomly oriented in the loaf specimens.

The first generation of tests utilized the MM designs, with two different casting methods (Foltz et al. 2008, Sirijaroonchai et al. 2010). As explained previously in Foltz et al. (2008), individually-cast specimens reinforced with fibers, such as for the MM in first generation testing, tend to experience a significantly more explosive failure mode in the out-of-plane direction at their maximum stress, with limited residual strength and deformation capacity. In contrast, specimens cut and trimmed from HPFRCC MM loaves achieved a random orientation of fibers, which significantly enhances the residual strength and deformation capacity of these specimens. Further detailed explanations on this issue can also be found elsewhere (Yin et al. 1989). Therefore, after test results of the individually-cast plate specimens were compared to those of the cut-and-trimmed plates from loaf specimens, a decision was made in the second generation of testing to explore only casting in loaves for the NM6 mix design, as individually-cast specimens cannot guarantee well-dispersed fiber directionality in the out-of-plane direction.

With regard to the effect of fiber volume fraction (V_f), test results reported by Liao et al. (2006) and Traina and Mansour (1991), as well as those from the MM specimens presented in Fig. 2, clearly show a general trend that the effect of fiber volume fraction and fiber type on compressive strength of HPFRCC subjected to uniaxial and biaxial stress is marginal, unlike the case for tensile behavior of HPFRCC. On this basis, the second generation test program reported here mainly focuses on the NM6 specimens with and without 1.5% hooked steel fibers, in both an absolute sense and in comparison to similar MM specimens. Even more detailed experimental results from first generation testing can be found in the authors’ previous work (Foltz et al. 2008).

Mixing procedures

The mixing protocol outlined by Liao et al. (2006) was followed when batching materials for the specimens. Two specific mixing procedures for self-consolidating concrete were adopted: i) pouring the water pre-mixed with chemical admixtures in several steps, in order to provide a homogenous matrix, and ii) reducing the coarse-to-fine aggregate ratio, in order to provide a well-developed paste layer to fully coat the coarse aggregate.

First, the water was mixed with the superplasticizer and viscosity modifying admixture. Then, the cement, sand, and fly ash were mixed together using a concrete pan mixing machine for about 30 seconds. At this point, approximately half of the liquid solution was added to the mix until the dry components were fully mixed with the liquid. After mixing for about one minute, half of the remaining liquid was poured into the mix. After another minute of mixing, half of the remaining liquid was again poured into the concrete mixture. Then, after still another minute of mixing, the remaining liquid solution was poured into the mix. Next, all the coarse aggregate (when used) was added and mixed for about two minutes. Finally, fibers were slowly added incrementally to the mix. During addition of fibers, special care was made to ensure that they did not clump or ball up, especially for the Spectra fibers. After mixing for about three additional minutes, the HPFRCC was ready to be cast.

Before casting certain of the specimens, a slump flow test (EFNARC, 2002) was performed on the HPFRCC batches. Rather than measuring loss of height, as in the standard ASTM slump test, a slump flow test measures the average diameter of concrete spread in two perpendicular directions. In general, higher slump flow corresponds to an increased ability to fill formwork. The slump flow for NM6 was 22.2 in. (565 mm). Although this slump flow value is in the EFNARC (2002) SF1 slump flow class, which is the least flowable category of recommended SCC mixes, it actually indicates remarkable slump-flow for a fiber reinforced concrete mix. Also, when judged against other fiber reinforced concrete mixes found in the literature with similar fiber volume fractions, the measured slump-flow compares quite favorably (Liao et al. 2006).

Once the HPFRCC was deemed adequate, the material was cast into plastic molds and placed on a vibrating table to achieve sufficient compaction. After each concrete placement, specimens were kept in their molds and covered with plastic sheets for about 24 hrs. They

were then removed from the molds and placed into a water curing tank for at least another 28 days. All specimens were allowed to dry for at least 48 hours prior to testing.

As previously noted, specimens cast as loaves were cut with a diamond precision saw to 5.5 x 5.5 x 1.5 in. (140 x 140 x 38 mm), whereas the individually-cast specimens were already this size at casting. To ensure uniform biaxial stress and strain fields, the four sides of each specimen were lightly ground to achieve flat edges and right-angle corners. A problem reported during some previous testing, by Maekawa and Okamura (1983), was that local damage could occur at the interface between the specimen and the platen as a result of lower local strength due to bleeding in the concrete; trimming and/or grinding the edges of the specimens to avoid these areas proved to be effective in preventing damage localization at the edges in this study.

Testing apparatus

The plate specimens were tested under displacement-controlled loading, with the ratio of principal strains varied between tests in an effort to obtain a comprehensive understanding of the biaxial behavior of the material. As shown in Fig. 3, testing was conducted using an INSTRON biaxial servo-controlled hydraulic frame in the Newmark Structural Engineering Laboratory (NSEL) at the University of Illinois. A closed-loop system in displacement control was used to capture the post-peak response of the specimens, with all of the biaxial compressive loads applied simultaneously. Displacement control was provided by AC linear variable differential transformers (LVDTs) within each hydraulic actuator. Both axes of loading had one actuator slaved to a master actuator through digital line connections, and closed-loop control of the actuators was executed using INSTRON 8500 and INSTRON 8800 controllers.

Similar to what was done by previous researchers (Kupfer et al. 1969, Nelissen 1972), frictional confinement at edges of the test specimens due to loading was minimized by using brush-type loading platens. The brush platens were pin-connected to testing fixtures, including simple guide-ways to ensure planar loading, which were then in turn mounted to the load cell of each actuator. For uniaxial compression and equal biaxial compression loading, the strain rate was 0.01 in./min (0.25 mm/min), based in part on ASTM C39 (2006). For intermediate targeted compression stress ratios, σ_1/σ_2 , the strain rate was simply reduced in the horizontal direction to achieve the desired ratio.

Figs. 3(a) and 3(b) show both a picture of the typical test setup and a detail of measuring devices. Strain and displacement measurements were obtained using a non-contact Krypton K600 Coordinate Measuring Machine (CMM). The Krypton CMM can simultaneously obtain the three-dimensional location of numerous small light-emitting diodes (LEDs), to an accuracy of ± 0.0008 in. (0.02 mm). Eight LEDs were placed around the perimeter of an overall 3 in. x 3 in. (75 mm x 75 mm) grid, with a 1.5 in. (38 mm) target spacing, centered on the specimen. To obtain additional out-of-plane data, two 0.25 in. (6 mm) stroke LVDTs were positioned on special frames and placed such that they were touching the center of each face (front and back) of the test specimen. Analog output signals from the measuring devices were connected to an NSEL data acquisition system, with four-axis control of the tests and collected data synchronized using a PC. The Krypton measuring system had its own data acquisition software, so the two sets of data were further synchronized during post-processing. Once a specimen was secured in the testing frame, it was preloaded to about 225 lbs (1 kN) in the direction(s) of loading, in order to remove any excess flexibility in the system and ensure proper platen contact with the specimen before test initiation.

EXPERIMENTAL RESULTS

Uniaxial test results

To eventually better understand the response of plate specimens under more complex biaxial stress states, uniaxial stress-strain behavior and failure modes are first investigated. Fig. 4 shows a comparison of the average stress-strain response of test specimens from NM6 HPFRCC vs. plain concrete results; the plain concrete specimens have the same mix proportions as NM6, only without any fibers. These stress-strain curves were obtained from averaging applied stress across a group of specimens for a given strain level from the Krypton LEDs. When processing the data, the flexibility of the test setup was assessed by comparing the relationship between the stiffness found using data from the loading platens and that obtained from the Krypton targets. Local strain accumulations could adversely influence data collected at individual LEDs on the surface of a specimen, so once the test setup flexibility was corrected for in the experimental data, reliable post-peak strain behavior for the fiber-reinforced specimens was then also able to be obtained from the actuator data. As shown in Fig. 4, plain concrete specimens failed abruptly without any significant post-

peak response; however, the HPFRCC exhibited a gradual descending branch and residual compressive capacity of approximately 50% of the maximum applied load out to 3.0% strain.

Fig. 5 shows typical failure modes of loaf NM6 plate specimens, with and without fibers, subjected to uniaxial compression. For plain concrete, when the material is loaded in one direction without any active or passive confinement, the largest macrocracks develop in the direction of loading. Thus, for the uniaxial compression case, crack formation was characterized by a series of vertical tensile splitting cracks and out-of-plane deformations, as shown in a side view of a plain concrete specimen, which finally resulted in a dramatic loss of capacity for such specimens. However, with addition of fibers for loaf specimens, growth of macrocracks is retarded, and the material ultimately fails by interconnection of numerous small microcracks along a faulting zone inclined at an angle to the direction of applied load. This type of failure has been characterized as a faulting failure (Yin et al. 1989). The fibers are able to arrest the propagation of macrocracks, and can provide passive confinement in the out-of-plane direction, so the HPFRCC specimens still exhibited significant ductility beyond peak loading.

Fig. 5 also shows that plate specimens with or without fibers had nearly identical stiffness. It is expected that the addition of fibers will not affect the stiffness in the linear range, but at strains when plain concrete begins to undergo damage, the HPFRCC specimens are restrained by the fibers. It can further be seen that the average uniaxial compression strengths of similar types of plain and HPFRCC specimens differed only slightly, which is consistent with the literature (Yin et al. 1989).

Table 1 presents a summary of average uniaxial compressive strengths (σ_{co}) for all the specimen types tested in this study. When comparing individually-cast plate specimens to those cut from large loaves, it can be seen that the uniaxial compressive strength of individually-cast plates was significantly higher than that of cut-and-trimmed plates from loaves. The individually-cast specimens also experienced an abrupt failure, with very limited residual strength. This again indicates that orientation of fibers for the individually-cast specimens was mostly in the plane of loading, due to the thin-walled geometry of the casting molds, and thus the experimental results presented here will focus on specimens from the second generation testing program, which were cut and trimmed from HPFRCC loaves.

Biaxial test results

Biaxial stress-strain behavior of concrete and HPFRCC is largely dependent upon the ratio of applied axial loads (Kupfer et al. 1969, Nelissen 1972, Tasuji et al. 1978, Hussein and Marzouk 2000). As previously described, the loading ratio (σ_1 / σ_2) was varied in this study by altering the horizontal strain rate in the biaxial testing machine. Table 4 presents a summary of biaxial compressive strengths (σ_{bo}) obtained from the experiments, while Fig. 6 shows the average compressive response in the vertical direction of NM6 specimens subjected to biaxial stresses of various loading ratios. It can be seen that similar peak stresses were obtained for loading ratios of 0.5, 0.7, and 1.0, with about 10% less for 0.3. Also, significant residual strengths were maintained up to very large compressive strain levels; in fact, more than half the peak strength was generally observed up to 3.0% strain, and about 35% of the peak strength even remained at 6.0% strain.

Fig. 7 displays the average compressive response of biaxially-loaded NM6 specimens for the horizontal direction (in which active confining stress is provided). When comparing this with Fig. 6, it can be seen that the ascending branch stiffness is essentially the same in both loading directions, regardless of loading ratio, further indicating a thorough and random dispersion of fibers in the loaf specimens. In addition, as reported by Sirijaroonchai et al. (2010), when comparing results of HPFRCC uniaxial, biaxial, and triaxial compression tests, the modulus of elasticity is essentially independent of multi-axial loading – the modulus of elasticity of NM6 specimens (with and without fibers) was about 2300 ksi (15.9 GPa). Another observation is that biaxially-loaded specimens subjected to low loading ratios experienced earlier softening with respect to the applied strain in the horizontal direction (such as seen for a 0.3 loading ratio). This result can be attributed to a significant accumulation of damage in the vertical direction during such tests – loading in each direction was begun simultaneously, so the specimen had already undergone considerable deformation (and damage) in the vertical direction before obtaining a substantial strain in the horizontal direction. Also, the strain at maximum applied stress shifted to about 0.4% for specimens subjected to a biaxial loading ratio of 0.5 or greater, whereas the strain at maximum stress was approximately 0.3% for uniaxial test specimens.

As shown in Fig. 8, van Mier (1986) demonstrated that the magnitude of ultimate compressive strength for concrete materials increases rapidly with an increasing magnitude of confining pressure in the active confinement direction, as long as some modest confining pressure remains in the other perpendicular direction (i.e., the out-of-plane direction in Fig.

8). According to van Mier, if there is however no confining stress available in a specific direction for a triaxial stress condition, then the strength increase due to multiaxial stress is only marginal. The described scenario is similar to the biaxial tests conducted in this study, because biaxial stresses are introduced in the vertical and horizontal directions while a relatively small passive confining stress also exists in the out-of-plane direction (with HPFRCC). This means that HPFRCC plate specimens subjected to biaxial compression can show enhanced compressive performance through active confining stress in the horizontal direction plus passive confining stress provided by fibers in the out-of-plane direction (when compared to plain concrete). Yin et al. (1989) also demonstrated that the effect of adding fibers into concrete is equivalent to providing some small confining (compression) in the unloaded direction(s). Under uniaxial loading, there is only modest confining pressure in each perpendicular direction, without any active confinement, and so the beneficial strength effect is minimal, but the failure mechanism is changed from splitting to faulting. Under biaxial loading, confining stress in the out-of-plane direction is small, but that in the horizontal direction is relatively large. Thus, for this case, as explained by van Mier (1986), the effect of confinement is significant on both strength and failure mode. These observations can help to explain how the compression performance of HPFRCC test specimens was significantly enhanced compared to those without fibers.

According to Fantilli et al. (2011), the stress-inelastic displacement relationship is a critical characteristic affecting the effectiveness of confinement (the so-called equivalent confinement) and post-peak ductility provided by fibers. To more clearly evaluate the effect of fibers, the stress-inelastic displacement relationship of NM6 specimens in the post-peak regions are presented in Fig. 9, where σ_{bo} is the biaxial compressive strength (i.e., the peak stress), σ_p is the vertical compressive stress in the post-peak regime, and w is the inelastic displacement calculated based on the block-sliding model proposed by Fantilli et al. (2011). As expected per Fantilli et al. (2011), NM6 specimens showed large inelastic displacements regardless of the active confinement stress levels, and these stress-inelastic displacement relations indicate that HPFRCC materials can provide high equivalent passive confinements.

The typical failure mechanism of plain concrete specimens was by tensile splitting. On the other hand, failure mechanisms experienced in biaxially-loaded HPFRCC specimens cut and trimmed from loaves were considerably different. Similar to as described in previous research (Yin et al. 1989), these specimens experienced a faulting or shear failure due to

formation of multiple fault planes in the specimen. All specimens exhibited either single shear or multiple shear failure modes, which is similar to previous results found in the literature (Ren et al. 2008). Fig. 10 shows examples of both the single shear and multiple shear failure modes for HPFRCC specimens, as well as the tensile splitting failure commonly observed in plain concrete specimens. The single shear failure mode can be identified by one diagonal crack inclined at about 30° to the unloaded out-of-plane surface, resulting in two triangle-shaped prisms. The multiple shear failure mode is similar to single shear, except the specimen fails along several inclined diagonal cracks, resulting in it being divided into a few triangular pyramids.

It was observed from post-test inspection that the individually-cast specimens had more fibers oriented in the plane of loading directions than in the 1.5 in. (38 mm) out-of-plane direction. As a result, individually-cast specimens failed in a brittle manner characterized by concrete exploding in the out-of-plane direction of the specimen. This demonstrates that the fibers were not oriented properly to provide passive confinement, as has also been pointed out previously (Foltz et al. 2008).

Failure envelope based on test results

Normalized ultimate strength combinations (σ_1 / σ_{co} and σ_2 / σ_{co}) are depicted as points on biaxial failure envelopes in Figs. 11 and 12, where stresses are reported as a fraction of the average unconfined uniaxial compressive strength of a plate specimen (σ_{co} , as given in Table 4) depending on the particular concrete mixture and casting method. These average envelope curves were obtained by first normalizing each specific concrete batch by its average uniaxial value, and then experimental data points having similar failure stress ratios for a particular specimen, mix, and fiber type were averaged to create the points for the curve. For example, the average failure curve for MM loaf specimens with Spectra fibers was made by first normalizing the 1.0, 1.5, and 2.0 percent fiber volume fraction results by their respective average uniaxial values, and then data points with similar failure stress ratios were averaged together. Each averaged point used to generate the failure envelope represents the results of 3 to 7 tests, depending on the quantity of specimens available for a particular mix. Each plot has been normalized by its corresponding average uniaxial compressive strength, so individually-cast specimens were not actually weaker than loaf specimens, but rather they simply did not benefit as much from the addition of a second principal confining stress.

Fig. 11 shows all the biaxial failure envelopes obtained from this study, including first generation test results already reported previously elsewhere (Foltz et al. 2008). It illustrates the effect of a more random fiber orientation by plotting the average result for each concrete mixture and casting method (i.e., individually-cast vs. cut-and-trimmed). Fig. 11 also displays the biaxial failure envelope of MM specimens without fibers. It can be seen that plain MM plate specimens showed an even greater biaxial strength increase than individually-cast HPFRCC specimens, which reconfirms the importance of fiber directionality (especially in the out-of-plane direction). Post-experiment inspections of individually-cast HPFRCC specimens showed that most fibers were indeed aligned in the plane of the specimen due to the relatively modest specimen thickness.

Fig. 12 shows a comparison between the biaxial failure envelope obtained from the reported experiments and those presented in the literature for tests on plain concrete (Kupfer et al. 1969, Nelissen 1972, Tasuji et al. 1978, Hussein and Marzouk 2000). It can be seen that the benefit of a biaxial stress state on strength enhancement of concrete is markedly increased with the addition of fibers in HPFRCC specimens, when comparing the average biaxial results from loaf NM6 specimens to historical plain concrete results.

Table 4 provides a summary of the average uniaxial and equal-biaxial compressive strengths for each specimen type. It can again be seen that individual specimens had much greater uniaxial strength, but did not experience any significant increase in strength under biaxial compression (with the exception of plain concrete, which had some increase). Meanwhile, all the loaf specimens (MM with hooked steel and Spectra fibers, and NM6 with hooked steel fibers) experienced a much greater benefit under biaxial loading due to passive confinement provided by randomly oriented fibers; in fact, a strength increase of more than 40 percent when under equal biaxial loading (vs. the respective uniaxial strength) was observed.

ANALYTICAL VALIDATIONS

The analytical phase of this research focuses on the derivation of material modeling parameters for nonlinear finite element analysis of HPFRCC planar members. In this study, the ATENA software (developed by Cervenka Consulting) has been adopted, which provides many advantages for conducting simple, yet accurate, nonlinear finite element modeling of structural concrete members. The primary intent is to validate material parameters within

the ATENA platform by modeling the plate specimen tests, which will enable the developed HPFRCC material model to then be extended to more complex structural components, such as columns, beam-column connections, coupling beams, or walls.

Material modeling background

The material tests were conducted on planar thin-walled plates, so the ATENA 2D software was used for validation of the HPFRCC modeling parameters. HPFRCC material was modeled using a fracture-plastic constitutive model, with an assumption of fully-composite material (Ju et al. 2013, Lee et al. 2014). Specifically, the CC3DNonLinCementitious2 model was implemented for HPFRCC. As described in detail by Cervenka et al. (2010), the fracture-plastic model combines constitutive models for tensile fracturing and plastic compressive behavior. The fracture model considers a classical orthotropic smeared crack formulation and crack band model. It uses the Rankine failure criterion with exponential softening. The plasticity model is based on the Menetrey-William failure surface (Menetrey and William 1995, Grassl et al. 2002) with a return mapping algorithm for the integration of constitutive equations. Strain decomposition, as described by De Borst (1986), is used to combine the fracture and plasticity models together, where the strain is computed as the sum of the elastic, plastic, and fracture components.

The Rankine criterion is implemented for concrete cracking, with the crack opening determined as a function of characteristic length. The characteristic length concept for crack band size originated in work by Bazant and Oh (1983), with the modification and approach suggested by Cervenka et al. (1995) used in this analysis. The plasticity model for concrete crushing is based on work by van Mier (1986), where the ascending branch of the compressive law is based on strains and the descending branch is based on displacements, to introduce mesh objectivity. Further detailed information about the theory and functionality of the ATENA software can be found in Cervenka et al. (2010).

Material properties for finite element analysis

The plate tests are the primary source of the material properties that are input into the finite element models. User-defined stress-strain curves were employed for both the uniaxial compressive and uniaxial tensile properties of the HPFRCC material. When defining material properties, the first parameters to apply were the compressive strength, tensile strength, Poisson's ratio, and initial modulus of elasticity. Tensile properties of the NM6

HPFRCC material were simply obtained from direct tension dog-bone tests; those experimental results have already been reported in Liao et al. (2006).

Figs. 13(a) and 13(b) show the uniaxial stress-strain behavior in compression and tension, respectively, of the HPFRCC material used in finite element modeling. For the plain concrete material, all of the properties were obtained from the plate tests, with default values used for parameters that were not directly tested. Several additional properties are required for finite element modeling, including the characteristic size (L_{ch}) and localized strain value (ε_{loc}^f). The compressive result is the average response obtained on 5.5 in. (140 mm) wide specimens, so this dimension was considered as the characteristic size in compression. Similarly, the width of the dog-bone specimens was 2 in. (50 mm), and thus this dimension was considered as the corresponding tensile characteristic size. The localized strain values were also obtained from the experiments. For both the compressive and tensile tests, strain began to localize at the peak strength, and then softening ensued through slow pullout of the fibers under continued deformation. The localized strain values were 0.0031 and 0.0046 in compression and tension, respectively.

The strain value used to determine the strength is calculated per the following equations (as described further below) for tensile strain, considering the characteristic length:

$$\tilde{\varepsilon}_1^f = \varepsilon_1^f \quad \text{If } \varepsilon_1^f < \varepsilon_{loc}^f \quad (1)$$

$$\tilde{\varepsilon}_1^f = \varepsilon_{loc}^f + \left(\varepsilon_1^f - \varepsilon_{loc}^f \right) \frac{L_t}{L_{ch}} \quad \text{If } \varepsilon_1^f \geq \varepsilon_{loc}^f \quad (2)$$

where L_{ch} is the crack band size, which is calculated as the size of the element projected onto the crack direction, and L_t is the mesh size used in the finite element analysis model. Strain values for the compressive case and for the shear strength retention function are based on similar strain calculations; however, appropriate values for the characteristic size and localized strain value should be used in each case. It is important to note that ε_1^f is the tensile strain calculated from the strain tensor at the integration points of the finite element, whereas the strain $\tilde{\varepsilon}_1^f$ is used to determine the current tensile strength from the provided stress-strain diagrams (i.e., Fig. 13). Use of the characteristic length represents a scaling that takes into account the difference between the experimental size and the size of the finite element. This approach ensures that the same amount of energy is dissipated when using large and small finite elements.

For a biaxial failure criterion, the plasticity model proposed by Menetrey and William (1995) is used. This failure criterion is characterized in two-dimensions by three parameters: f'_c (uniaxial compressive strength), f_t (uniaxial tensile strength), and e (an eccentricity parameter that influences the size and shape of the failure surface in deviatoric stress space). The default value of the eccentricity parameter (e) is 0.52, corresponding to the failure surface of plain concrete based on test results from Kupfer et al. (1969). Fig. 14 shows a comparison of the failure envelope obtained by Kupfer et al. (1969) to a biaxial envelope curve estimated from the ATENA models with the eccentricity parameter (e) set equal to 0.52 as a default value provided by ATENA. It can be confirmed that the analysis results with the eccentricity parameter (e) of 0.52 match quite well with Kupfer's biaxial strength curve (for plain concrete), but it would result in significant error if the default eccentricity parameter value of 0.52 is adopted for HPFRCC materials. This is because the test results presented in this study showed an expanded biaxial failure surface for HPFRCC, beyond that of plain concrete, and thus the eccentricity parameter (e) should be modified to better reflect those experimental results. The Menetrey-William failure surface with an e value of 0.554 gave a reasonable match with the NM6 HPFRCC failure surface, as shown in Fig. 14. This failure surface is slightly conservative at around 0.2 of compressive stress ratio, while it is slightly unconservative at around 0.6 of compressive stress ratio, where the compressive stress ratio is defined as the ratio of horizontal (X-direction) displacement rate to vertical (Y-direction) displacement rate during the biaxial plate experiments.

Compression softening of the material due to tensile strains in the orthogonal direction was also considered, as has been done in the modified compression field theory (Vecchio and Collins 1986). In addition to the previously described material parameters, multiple cracking during the hardening phase and localized cracking during softening of the HPFRCC can also be modeled. It was assumed that a set of parallel planar multiple cracks will form perpendicular to the maximum principal stress when the applied stress exceeds the first cracking strength, as described by Kabele (2007). As loading increases, additional cracks may form within the finite element, but the direction of the initial set of cracks is fixed to be perpendicular to the principal stress direction. Cracks are then allowed to slide if the direction of principal stress changes; however, crack opening and sliding is resisted by fiber bridging at crack interfaces in HPFRCC. The crack sliding phenomenon is implemented using a variable shear retention factor (β), with β defined as the ratio of the material's

post-cracking shear stiffness (G^c) to its elastic shear stiffness (G). The shear retention factor (β) is affected by fiber volume fraction, fiber modulus of elasticity, and fiber aspect ratio (Lee et al. 2014).

Within the finite element, a secondary set of cracks may form in a direction perpendicular to the primary set of cracks if the maximum normal stress in the secondary crack-normal direction also exceeds the cracking stress. No interaction is assumed between the two sets of cracks. Once the normal cracking strain in a set of multiple cracks exceeds the cracking strain capacity, a localized crack will form and material softening will occur. Overall strain of the representative volume of material is then obtained as the sum of the strain in the material between the cracks, the cracking strains due to multiple cracking, and the cracking strains due to localized cracks.

Of the material parameters used for HPFRCC in the finite element modeling, all properties were obtained experimentally except for the shear retention behavior. There are limited test results to define the shear retention behavior of HPFRCC material as a function of crack opening, and so the model by Kabele (2007) was adopted in this study; this shear retention behavior may only have a marginal effect on the compression responses of HPFRCC (Lee et al. 2014).

Mesh and boundary conditions

A brief mesh sensitivity study was conducted on the finite element models under uniaxial compression. The NM6 concrete mix had aggregate as large as 3/8 in. (9.5 mm); therefore, a minimum reasonable mesh size was considered to be 0.4 in. (10 mm). Several analyses were conducted using increasingly larger mesh sizes, and it was found that a mesh size of 0.6875 in. (17.5 mm) provided adequate convergence. In the finite element models, displacements were imposed on the specimens through elastic steel elements, with the forces then transferred to the HPFRCC through interface elements. An interface material model was also used, to simulate contact between the concrete and the steel brush platens. The main purpose of the brush platens was to reduce lateral confinement of the specimen (to obtain the true stress-strain response), and thus it was important to appropriately model the interface elements. The elastic normal and shear stiffness of these elements were selected as small enough values to allow for the transmission of force while not restraining the edges of the specimen, and the cohesion was set to zero. For uniaxial tests, displacements were

applied at the top brush platen, while the bottom platen was fixed against vertical displacement along its entire length (with the left-most node of the bottom platen restrained against lateral displacement). For biaxial compression simulations, displacements were applied at the top, left, and right brush platens, while the bottom platen was restrained against vertical and horizontal displacements.

Finite element analysis validations

With material parameters completely defined for the HPFRCC, an analytical modeling program was undertaken to validate the material model by reproducing some results from the previously described suite of tests on plate specimens. The plate specimens were modeled with plane quadrilateral elements having four Gauss integration points. The first step in material test validation was to model plain concrete plate specimens subjected to uniaxial loading. This step could confirm that the assumed boundary conditions were appropriate, and that the response of concrete without a significant post-peak response could be captured. For loading, displacement increments of 0.001 in. (0.0254 mm) were applied to the brush platen at each load step, and a modified Newton-Raphson approach was used as the solution method. Results from plain concrete modeling showed that the stresses were indeed uniformly distributed in the specimen, without a concentration at the interface between the platen and the specimen. It was additionally observed that appropriate properties had been input for the interface elements to ensure that the specimen was free to deform without interfacial restraint; with brush platens used in the experiments, it was important to model this aspect of the tests correctly in order to achieve the proper stresses and strains in the material. The strains and stresses were recorded at an integration point near the center of the specimen, and Fig. 15 shows a comparison of the experimental and modeling results for a plain concrete plate specimen subjected to uniaxial compression. It can be seen that the analysis ended abruptly for this plain concrete specimen, which is reminiscent of the experimental failure mode.

The next step in the analytical program was to model the uniaxial response of HPFRCC specimens. The same model was used as shown previously for the plain concrete specimens; however, the appropriate HPFRCC material properties were applied. Fig. 16 shows a comparison of the experimental and analysis results for a NM6 HPFRCC plate specimen subjected to uniaxial compression, which indicates that adequate modeling of the

uniaxial compressive behavior has been achieved. In addition, it was important that the finite element analysis results could capture the ductility inherent in the HPFRCC material.

The first biaxial modeling effort was to simulate the effect of equal biaxial compression, as was done during some of the experiments. Loads were applied through prescribed displacement increments on the top, left, and right platens. The platens were connected to the specimen through interface elements, and the properties of the interface elements allowed for expansion or contraction of the specimen along the platen – it was found that the model did indeed adequately represent the real boundary conditions. The biaxial response of the specimen was recorded through four monitoring points located at the Gauss integration point closest to the center of the specimen. Two of the monitoring points recorded strains (X and Y), while the other two recorded the stresses (X and Y) in the specimen. As shown in Fig. 17(a), the equal biaxial modeling results do show some difference versus the test results, but the finite element analysis results closely capture the overall behavior of the HPFRCC material under biaxial compression – namely, the strength, stiffness, ductility, and large residual strength.

The unequal biaxial models used the same prototypical modeling approach as the equal biaxial models; however, the prescribed horizontal displacements were varied to achieve the desired compression ratio (of horizontal strain rate to vertical strain rate). For example, the 70% biaxial result had a vertical strain rate of 0.001 in. (0.025 mm) per load step and a horizontal strain rate of 0.0007 in. (0.018 mm) per load step. To complete the full biaxial failure envelope, the model was run at many different loading ratios, but only certain cases were selected to be shown because they correspond to the specific average experimental values that were used to create the experimental biaxial failure envelope. Figs. 17(b), 17(c), and 17(d) display comparisons of the vertical (Y) and horizontal (X) stress-strain responses obtained from the finite element analyses and experiments. For the 70% stress ratio shown in Fig. 17(b), the analytical peak stress and stiffness (both horizontally and vertically) aligned well with the experimental results. In the vertical direction, even the post-peak behavior was quite similar to the test results, but the horizontal modeling result did not experience the same softening before the maximum horizontal stress was achieved. This is largely a shortcoming of the modeling approach, which was seen for each of the loading ratios. Despite the lack of pre-peak softening in the horizontal direction, the essence of biaxial response was reasonably captured.

For the 50% and 30% stress ratios, Figs. 17(c) and 17(d) exhibit the same trends, and again the main properties of HPFRCC such as ductility and residual strength were well represented. For the 30% stress ratio, however, the modeling results somewhat less closely resembled the experimental results. It was found that the 30% strain rate did not necessarily result in an approximate 30% force, a trend that continued as displacement rate was reduced, so to complete the biaxial envelope at lower compression stress ratios, prescribed forces were applied. While using load-control in the model does not provide any post-peak behavior, it does allow the user to identify the maximum loading in order to represent a point along the failure envelope.

For a full comparison of experimental results to modeling results, additional simulations to those described were conducted at various biaxial loading ratios, and tension tests were also simulated. A review of the literature showed that Demeke and Tegos (1994) conducted tests on fiber reinforced concrete under compression-tension conditions, and their results showed that the compression-tension regions of the biaxial failure envelope were essentially linear from the uniaxial compressive strength to the uniaxial tensile strength; however, as expected, the uniaxial tensile strength was considerably larger than for plain concrete response. The finite element analysis conducted in this study showed a similar trend in the compression-tension region, as can be seen in Fig. 18 that displays a comparison between the experimental failure envelope and the failure envelope obtained from finite element modeling. It shows that under biaxial compression, the results align well from compression stress ratios of 50% on up to equal compression; however, analysis results underestimate the test results at lower loading ratios. For the estimated biaxial strength of the tensile regimes, while specific experimental data was not available to validate the model of plate specimens under compression-tension and tension-tension, it can be confirmed that the estimated compression-tension and tension-tension regimes of plate specimens were reasonable in that they are linearly continuous from the compression-compression regime, as reported in Demeke and Tegos (1994). Overall, the model has shown that it is able to capture the strength, deformation, stiffness, and ductility of HPFRCC under both uniaxial and biaxial loading. For easy application of a biaxial failure envelope for HPFRCC materials in compression, a simple biaxial failure envelope curve of HPFRCC was derived based on a model presented by Darwin and Pecknold (1974), as follows:

$$\frac{\sigma_2}{\sigma_{co}} = -0.6 \left(\frac{\sigma_1}{\sigma_{co}} \right)^2 + 1.1 \frac{\sigma_1}{\sigma_{co}} + 1.0 \quad \text{for } \sigma_2 \geq \sigma_1 \geq 0 \quad (3)$$

where σ_{co} is the average unconfined uniaxial compressive strength of HPFRCC; this equation is symmetric about $\sigma_2 = \sigma_1$. As shown in Fig. 18, Eq. (3) provides a good approximation to the testing and finite element analysis results.

CONCLUDING REMARKS

This study has investigated the biaxial behavior of high-performance fiber-reinforced cementitious composites subjected to various compressive stress ratios. The experimental program was performed on HPFRCC plate specimens, and analytical validations were also conducted. On this basis, the following conclusions can be drawn:

1. Specimen fabrication method was considered as a key test variable in the first phase of the experimental program. It was confirmed from these test results that individually-cast specimens reinforced with fibers show limited residual strength and explosive failure characteristics in the out-of-plane direction due to poor fiber dispersion.
2. Specimens fabricated from HPFRCC loaves using a cut-and-trim method show good fiber directionality, and their biaxial performance can be significantly enhanced compared to plain concrete (in terms of residual strength and deformation capacity).
3. A biaxial compressive failure envelope was derived based on the test results, which indicates that some modification of an existing biaxial strength model is required to properly estimate the multi-axial strength of HPFRCC materials.
4. Failure envelopes and constitutive relationships of HPFRCC materials were obtained from the experiments. These properties were then implemented into nonlinear finite element models; material performance of the HPFRCC under multi-axial loads can be adequately estimated by finite element analysis in a relatively simple manner.
5. The developed material modeling approach can be extended for analyses of actual structural members utilizing HPFRCC materials, such as beam-column connections, shear walls, and columns.

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Table 1 – Test matrix and uniaxial compressive strengths

Mix	Specimen type	Fiber type	Average uniaxial compressive strength, ksi (MPa)	Number of specimens
Mortar mix (MM)	Individual	Steel fiber	10.2 (70.1)	30
		Spectra fiber	8.7 (60.2)	21
		Plain	8.6 (59.4)	9
	Loaf	Steel fiber	6.6 (45.8)	17
		Spectra fiber	5.8 (39.9)	18
NEES Mix #6 (NM6)	Loaf	Steel fiber	4.9 (33.5)	20
		Plain	5.4 (37.3)	12

Table 2 – Mixture proportions

Matrix type		MM	NM6
Cement type III (Early ages)		1	1
Aggregate	Silica sand (Flint)	1	2.2
	Coarse aggregate	-	1.2
Fly ash class C		0.15	0.875
Chemical admixtures	Superplasticizer	*	0.005
	VMA	-	0.038
Water		0.4	0.8
Fiber	Type of fiber	Steel and Spectra	Steel
	Fiber volume content (%)	1.0, 1.5 & 2.0	1.5
28-day compressive strength, ksi (MPa)		80 (55.2)	5.5 (37.9)

* Superplasticizer added as needed when MM was too dry

Table 3 – Fiber properties

Fiber type	Diameter, inch (mm)	Length, inch (mm)	Density, g/cc	Tensile strength, ksi (MPa)	Elastic modulus, ksi (MPa)	Aspect ratio
Steel fiber	0.015 (0.38)	1.18 (30)	7.9	304 (2100)	29000 (200)	79
Spectra	0.0015 (0.038)	1.50 (38)	0.97	374 (2585)	16960 (117)	1000

Table 4 – Summary of average uniaxial and equal biaxial strengths of test specimens

Mix	Specimen type	Fiber type	Fiber volume fraction (%)	Average uniaxial strength, ksi (MPa)	Average equal biaxial strength, ksi (MPa)
Mortar mix (MM)	Individual	Steel fiber	1.0	10.2 (70.5)	10.3 (71.0)
			1.5	10.1 (69.4)	9.3 (64.4)
			2.0	10.2 (70.6)	10.8 (74.2)
		Spectra fiber	1.0	9.1 (62.8)	9.2 (63.2)
			1.5	9.0 (61.8)	8.7 (60.1)
			2.0	8.1 (56.0)	8.5 (58.9)
		Plain	-	8.6 (59.4)	10.6 (73.1)
	Loaf	Steel fiber	1.0	5.2 (36.1)	10.2 (70.2)
			1.5	7.2 (49.9)	10.1 (70.0)
			2.0	7.5 (51.6)	9.5 (65.3)
		Spectra fiber	1.0	6.3 (43.8)	9.4 (64.6)
			1.5	5.4 (37.2)	9.3 (64.1)
			2.0	5.6 (38.9)	9.0 (62.0)
NEES mix #6 (NM6)	Loaf	Steel fiber	1.5	4.9 (33.5)	6.7 (46.2)
		Plain	-	5.4 (37.3)	4.4 (30.7)

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Figures

(a) Dimensional details of loaf specimen

(b) Effect of casting method on fiber directionality

Fig 1 – Trimming and cutting of loaf specimens

(a) Spectra polyethylene fiber

(b) Hooked steel fibers

Fig 2 – Equal biaxial compression test results of MM plate specimens (per Foltz et al. 2008)

(a) Test set-up

(b) Details of specimen and measuring devices

Fig. 3 – Biaxial experimental test set-up

Fig. 4 – Average uniaxial compressive response of test specimens

(a) HPFRCC specimen (NM6)

(b) Plain specimen (NM6)

Fig. 5 – Typical failure modes of plate specimens with and without fibers

Fig. 6 – Average NM6 biaxial compressive responses in vertical direction of test specimens

Fig. 7 – Average NM6 biaxial compressive responses in horizontal direction of test specimens

Fig. 8 – Active and passive confinement

Fig. 9 – Stress-inelastic displacement relationship of NM6 HPFRCC plate specimens

(a) Single shear failure mode

(b) Multiple shear failure mode

(c) Splitting failure mode

Fig. 10 – Typical failure modes of plate specimens subjected to biaxial compression

Fig. 11 – Normalized biaxial compressive failure envelopes obtained from tests

Fig. 12 – Comparison of normalized biaxial compressive failure envelopes

(a) Compression behavior

(a) Tensile behavior

Fig. 13 – Uniaxial stress-strain behavior in compression and tension of HPFRCC material used in finite element analysis modeling

Fig. 14 – Experimental results of NM6 HPFRCC plate specimen and approximate failure envelope curve used in finite element analyses

Fig. 15 – Comparison of uniaxial compression test results of a plain concrete plate specimen with finite element analysis results

Fig. 16 – Comparison of uniaxial compression test results of a NM6 HPFRCC concrete plate specimen with finite element analysis results

Fig. 17 – Comparison of biaxial compression test results of NM6 HPFRCC concrete plate specimens with finite element analysis results

Fig. 18 – Comparison of biaxial failure envelopes of HPFRCC concrete plate specimens with finite element analysis results and proposed simple model