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PERFORMANCE OF ENGINEERED CEMENTITIOUS COMPOSITES UTILIZING LOCALLY AVAILABLE MATERIALS IN THE STATE OF LOUISIANA

Hassan Noorvand [1, 3], Gabriel Arce [1, 4], Marwa M. Hassan [1, 5], and Tyson Rupnow [2, 6]
[1] Department of Construction Management, Louisiana State University, USA
[2] Louisiana Transportation and Research Center, USA
[3] hnoorv1@lsu.edu
[4] garcea1@lsu.edu
[5] marwa@lsu.edu
[6] tyson.rupnow@la.gov

Abstract: Engineered cementitious composites (ECC) are steady-state multiple cracking strain-hardening cementitious materials that significantly enhance ductility and tensile strength of traditional cement-based materials. This investigation focuses on the development of ECC utilizing locally available materials in the state of Louisiana. The influence of high contents of fly ash (up to 81% cement replacement) and low Polyvinyl Alcohol (PVA) fiber content (1.5% volume fraction) were investigated for cost-effectiveness of the composite. Compressive and third-point bending tests were conducted to characterize the mechanical properties of PVA-ECC mixes produced at different levels of matrix/interface tailoring. Experimental results demonstrated the feasibility of producing ECC exhibiting robust strain-hardening behavior with locally available materials in the state of Louisiana and low fiber content (1.5% volume fraction of PVA fibers). Furthermore, results suggested that increasing cement replacement with fly ash favored ductility of the composites. Yet, compressive, and flexural strength were reduced suggesting a trade-off between strength and ductility. Moreover, specimens with the highest cementitious matrix strength (specimens produced with the lowest fly ash content) did not exhibit strain-hardening behavior but a strain-softening performance similar to that of fiber reinforced concrete (FRC). Strain-softening behavior was attributed to an excessive matrix strength that did not allow the strength or energy criteria to be met.

1 INTRODUCTION

Concrete durability, cost-effectiveness, and versatility make it a superior material for construction of civil infrastructure. Yet, concrete is fragile and weak in tension, and its failure and deterioration largely relate to these attributes. For instance, when tested in tension or flexion, concrete will possess load carrying capacity until its first crack strength. However, once the first crack strength is reached and a crack is initiated, crack propagation accompanied with loss of load carrying capacity will occur in an almost instantaneous fashion. The development of fiber reinforced concrete (FRC) has shown substantial improvement in the fracture toughness of concrete by resisting the propagation of cracks. Yet, most FRC possess quasibrittle postpeak tension softening behavior (in tension and flexion) where an inferior load than the first peak load is sufficient for further crack opening and propagation after first crack formation. Consequently, the tensile strain capacity is only marginally changed, keeping it closely to that of ordinary concrete, i.e., about 0.01% (Mai 1995).

In recent years, many efforts have been conducted to transform the quasibrittle behavior of FRC to a ductile strain-hardening behavior similar to that of steel. Different approaches have been proposed resulting in a modern class of cement-based material called high performance fiber reinforced cementitious composites (HPFRCC). HPFRCC are characterized by tensile strain-hardening behavior after initiation of the first crack and its strain capacity can be as much as several hundred times that of FRC (Naaman 1996; Şahmaran et al. 2011). According to Naaman and Reinhardt, HPFRCC is described by way of an ultimate strength higher than the first cracking strength of the composite (controlled by the cementitious matrix fracture toughness) and formation of multiple cracking throughout the inelastic deformation system (Naaman 1996). While the apparent strain in FRC depends on the gage length, the deformation of HPFRCC is considered a pseudo-strain and is uniform and independent of the gage length on a macro-scale level (E. Yang 2008). The word “pseudo strain-hardening” and sometimes referred to as “pseudo ductility” is utilized to differentiate between strain-hardening behavior in metals and the behavior of crack-based deformation in HPFRCC because of dislocation micromechanics (Victor C. Li and Wu 1992a).

A specific type of micromechanically designed HPFRCC called engineered cementitious composites (ECC) has been developed utilizing a low volume fraction (about 1 to 2%) of short fibers (Victor C Li 1998). ECCs are unique fiber reinforced cementitious materials that possess a high composite ductility in the range of 100 to 500 times to that of ordinary concrete (strain capacity of 1 to 5% in tension) (M. L. and V. C. Li 2009). The main characteristic of ECC is the formation of multiple steady-state microcracks as tensile stress increases resulting in strain hardening behavior (Victor C. Li 2008). This characteristic of ECC overcomes many limitations of concrete. For instance, the marginal tensile strength, the limited deformation capacity and the deterioration through macro-cracking of regular concrete can be addressed (Fukuyama, Sato, and Li 2000).

Li et al. evaluated the performance of steel-reinforced ECC (R/ECC). It was observed that R/ECC possess a higher load carrying capacity than typical steel reinforced concrete due to the increased tensile strength and strain-hardening behavior of ECC (V C Li and Fischer 1999). Furthermore, increased shear capacity (limited by the tensile strength of the material) of R/ECC with potential for shear reinforcement reduction or even removal were reported (Kanda, T. and Li 1998). Improved ductility and structural damage tolerance of R/ECC due to enhanced deformation compatibility between ECC and steel are also among other benefits of ECC utilization (Victor C Li 2003). Moreover, ECC has also shown a great durability potential (Victor C. Li 2008). Several studies have concluded that ECC remains durable and preserves its mechanical performance against major types of concrete deterioration including sulfate attack, alkali silica reaction, corrosion, and freeze-thaw (Victor C. Li 2008; Liu et al. 2017). The enhancement is attributed to the superior ductility of ECC and tight crack width. In addition, self-healing properties of crack-damaged ECC have been reported due to its tight crack width (which allows for autogenous healing mechanisms of cementitious materials to be effective), resulting in recovery of mechanical and transport properties, further enhancing the durability potential of ECC (Şahmaran et al. 2015). In addition, ECC has been successfully applied in the field for repairing concrete structures, in bridge deck patches, and in bridge deck link slabs (Victor C. Li 2008; Victor C Li 2003).

2 OBJECTIVES AND SCOPE

This study focuses on the development of PVA-ECC utilizing locally available materials in the state of Louisiana. The implementation of local materials, and utilization of low fiber content and high replacements of cement with fly ash throughout this investigation aim to make ECC cost-effective and readily available for future applications in the transportation infrastructure in the state. To this end, the main objectives of this study were: (1) to develop PVA-ECC mixes utilizing locally available materials in the state of Louisiana; and (2) to evaluate and correlate compressive strength, flexural strength, and ductility of PVA-ECC mixes to different levels of matrix/interface tailoring.

3 BACKGROUND

The design concept of ECC and its characteristic tensile strain hardening behavior is built on the bases of fracture mechanics and micromechanics. In the early 1990's, the first micromechanical models correlating ECC pseudo strain-hardening performance to micromechanical parameters of the fiber, matrix and interface (between fiber and matrix) of the cementitious composite were proposed by Li et al. (Victor C. Li and Wu 1992b). Li et al. recommended fiber-bridging model for cementitious composites reinforced with short random fibers under uniaxial tensile strength exhibiting strain-hardening behavior and steady-state multiple cracking (Victor C. Li and Wu 1992b).

According to ECC design theory, two fundamental requirements need to be met for successful pseudo strain hardening to be achieved. These criteria are the strength criterion and the energy criterion (or steady-state cracking criterion) (Marshall and Cox 1988; Victor C. Li 1992). The consolidation of these criteria results in the guidelines for tailoring the matrix, fiber, and interface to achieve strain-hardening behavior with minimum fiber content.

The first condition for pseudo strain hardening, the strength criterion, guarantees satisfactory fiber-bridging capacity upon crack initiation from a defect location. The condition requires that the matrix first cracking strength (that is controlled by the initial flaw size and the cementitious matrix fracture toughness) does not exceed the maximum fiber-bridging strength on any potential crack plane according to the following equation (Victor C. Li 1992):

[1] $\sigma_0 \geq \sigma_{fc}$

Where,

σ_0= Maximum fiber-bridging strength; and

σ_{fc}= Matrix first cracking strength.

If this condition is not met, fibers will be ruptured or pulled out upon matrix first crack due to insufficient load carrying capacity of the cracked section.

The second condition for steady-state multiple cracking strain-hardening is the energy criterion (or steady-state crack propagation criterion). According to Li et al., steady-state crack propagation means that "a crack increases in length at constant ambient tensile stress σ_{ss} while maintaining a constant crack opening δ_{ss} (a flat crack, with the exception of a small region near the crack tip)" (Victor C. Li et al. 2002). The energy criterion is satisfied when the complementary energy of the fiber bridging relation does not exceed the crack tip matrix toughness according to the following equation (Victor C. Li 2008; Victor C. Li et al. 2002):

[2] $J'_b = \sigma_0\delta_0 - \int_0^{\delta_0} \sigma(\delta)\, d\delta \ \geq \ J_{tip} = \sigma_{ss}\delta_{ss} - \int_0^{\delta_{ss}} \sigma(\delta)\, d\delta \ \approx \frac{{K_m}^2}{E_m}$

Where,

J'_b= Complementary energy of the fiber bridging relation;

J_{tip}= Crack tip matrix toughness;

δ_0= Crack opening corresponding to σ_0;

σ_{ss}= Steady-state cracking stress;

δ_{ss}= Crack opening corresponding to σ_{ss};

$\sigma(\delta)$= Fiber bridging relationship;

K_m= Matrix fracture toughness; and

E_m= Matrix Young's Modulus.

If the crack tip matrix toughness J_{tip} (sensitive to the details of the cementitious matrix design such as water/cement ratio, fly ash replacement and sand particle size and content), is too high or inadequate energy absorption occur in the increasing phase of the σ-δ curve, then, steady-state crack propagation is hard to be satisfied (Victor C. Li et al. 2002; Pan et al. 2015). Figure 1a presents a graphical representation of the J'_b and J_{tip} in a typical fiber bridging stress curve. The hatchet area is representative of J'_b while the shaded area is illustrative of J_{tip} (which approximates $K_m{}^2/E_m$ at low fiber contents as shown in Eq. 2) (Victor C. Li et al. 2002).

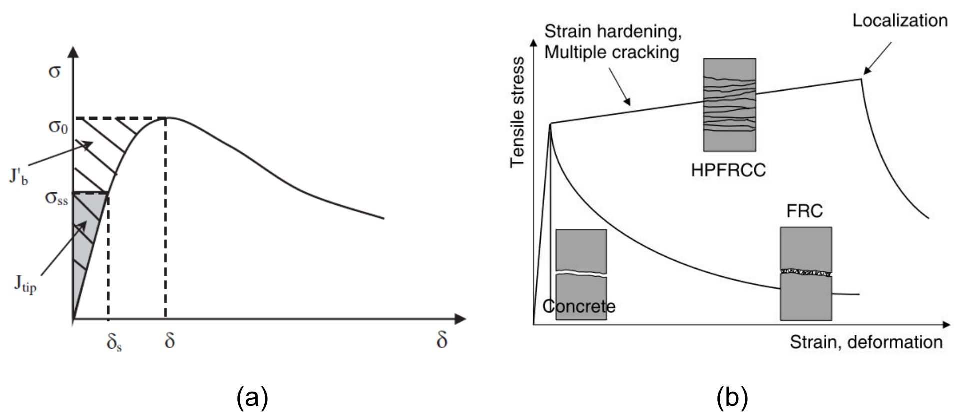


Figure 1: (a) Typical Curve of Fiber Bridging Stress σ vs. Crack Opening Width δ (Ma et al. 2015) (b) Schematic Stress–Strain Behavior of Cementitious Materials in Tension (Fischer and Li 2007)

From Eqs. (1) and (2), successful design of ECC is achieved when both strength and energy criteria are satisfied. Consistent with the conditions for pseudo strain hardening presented above, if the ratios J_b'/J_{tip} and σ_0/σ_{fc} are greater than one, both, strength and energy criteria will be met. Otherwise, if any of the two ratios is less than one, the tensile-softening behavior of fiber reinforced concrete will prevail (as shown in Figure 1b). Furthermore, based on theoretical and experimental experience, Kanda and Li suggested that the pseudo strain-hardening parameters of J'_b/J_{tip}> 3 and σ_0/σ_{fc} > 1.45 correlate to robust strain hardening and multiple cracking behavior (Kanda and Li 1998).

4 EXPERIMENTAL PROGRAM

4.1 Test Materials

One of the objectives of this study was to produce ECC mixes with locally available materials (except PVA fibers and admixture) and to evaluate its mechanical properties. For this purpose, the following locally available materials where utilized: Ordinary Portland Cement (OPC) Type I, Class F Fly Ash (FA), and river sand with a maximum particle size of 1.18 mm and a fineness modulus of 1.96. Chemical compositions of cement and fly ash are presented in Table 1 according to XRF analysis.

Table 2: PVA Fibers Properties

Material	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	SO_3	K_2O	TiO_2	Na_2O
Cement	19.24	4.75	3.35	65.81	2.2	3.61	0.54	0.21	-
Fly ash	62.08	18.56	8.22	5.69	1.69	0.37	1.42	1.03	0.35

Fibers utilized throughout the investigation were non-oil coated RECS15 polyvinyl alcohol (PVA) fibers supplied by Kuraray Co. Ltd in Japan. Table 2 and Figure 3b illustrate the properties and shape of the PVA fibers utilized, respectively. Moreover, a polycarboxylate-based high range water reducer (HRWR) was utilized as an admixture.

Table 2: PVA Fibers Properties

Fiber type	Length (mm)	Diameter (μm)	Young's Modulus (GPa)	Tensile Strength (MPa)	Elongation (%)
RECS15	12	39	40	1456	5.7

4.2 PVA-ECC Specimen Preparation

Three cylindrical and three prismatic specimens were cast to evaluate the compressive and flexural strength of each mixture in this study; see Figure 2. Three levels of cement replacement with fly ash were tested with 55%, 69% and 81% replacement (by weight). Water to Binder ratio (W/B), Sand to Binder (S/B) ratio, and fiber content were kept constant at 0.27, 0.36, and 1.5%, respectively. The details of the different mix proportions are summarized in Table 3. Specimens prepared were demolded after 24 hours of casting (specimens were covered with plastic to prevent moisture loss), and then allowed to cure for 28 days in a moist room (23 ± 2 °C, > 95% Relative Humidity [RH]) according to ASTM C192 (ASTM Standard C 192 2002).

Figure 2: Preparation and Casting of PVA-ECC

For the mixing procedure, dry powder components (Cement and Fly Ash) were mixed in a Hobart mixer for 3 minutes. Then, sand was combined with the dry powders and mixed for three additional minutes. Subsequently, water and HRWR were added and mixed for three more minutes. Finally, PVA fibers were introduced slowly to the wet mix (for 3 min) and mixed for an additional 7 minutes (Figure 2).

Table 3: ECC Mix Design Proportions by Weight

Mix ID	Proportions by Weight								
	Cement	Fly Ash	Water	Sand	HRWR*	Fibers (Vol%)	FA/C	W/B	S/B
Mix 1	1	1.2	0.6	0.8	0.625	1.5	1.2	0.27	0.36
Mix 2	1	2.2	0.87	1.16	0.286	1.5	2.2	0.27	0.36
Mix 3	1	4.4	1.47	1.96	0.227	1.5	4.4	0.27	0.36

*HRWR dosage by weight of cement

4.3 Compressive Strength Test

Compressive strength of ECC mix designs was evaluated according to ASTM C 39 (Compressive Strength of Cylindrical Concrete Specimens) on 101.6 x 203.2 mm (4 in x 8 in) cylindrical specimens (ASTM Standard C 39 2003). Three specimens were prepared for each mixture to assess the compressive strength of ECC at age of 28 days. The experimental tests were performed by means of hydraulic pressure with a constant loading rate of 0.25 MPa/s.

4.4 Third-Point Bending Test

A third-point bending testing procedure similar to ASTM C 1609 (Flexural Performance of Fiber-Reinforced Concrete) was conducted by utilizing a closed-loop, servo-controlled hydraulic universal testing system to assess flexural strength and deformation capacity of ECC mixtures (ASTM Standard C 1609 2005). Three prismatic specimens having dimensions of 101.6 x 101.6 x 355.6 mm (4 x 4 x 14 in.) were cast for each ECC mix design. The load was applied at a rate of 0.075 mm/min. The span length of 300 mm with center span length of 100 mm was used for flexural loading. Mid-span beam net deflection and load were recorded on an automated information recording system during third-point bending test. Testing setup is shown in Figure 3. To measure the flexural deflection of ECC specimens, two linear variable displacement transducer (LVDT) were attached to the testing set-up.

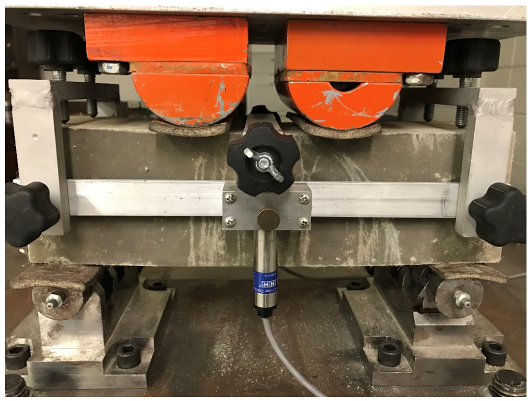


Figure 3: Third-point Bending Testing Setup

5 RESULTS AND ANALYSIS

5.1 Compressive Strength

The average compressive strength test results of ECC mixture M1, M2 and M3 at the age of 28 days are summarized in Figure 4. Replacement of fly ash with cement from 55%, 69% and 81% decreased the compressive strength of the specimens proportionally. The highest compressive strength of 54.3 MPa was achieved by 55% replacement of cement (FA/C = 1.2), which is significantly higher than the nominal compressive strength of normal concrete (30 MPa). Even replacement of 69% fly ash with cement (FA/C = 2.2) resulted in a compressive strength of 31.2 MPa that is slightly higher than that of normal concrete. On the other hand, increasing the replacement of fly ash up to 81% (FA/C = 4.4) decreased the compressive strength to an average of 16.2 MPa.

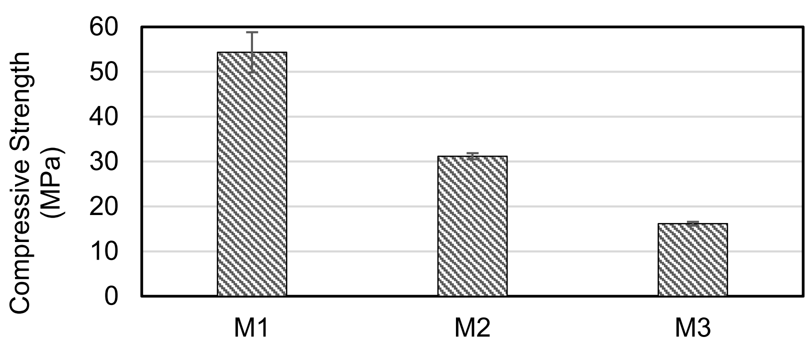


Figure 4: Average Compressive Strength (After 28 days of Curing)

Generally, the strength development of fly ash in concrete is achieved at later ages due to its pozzolanic properties (Mehta 1985). In addition, fly ash improves the mixture workability leading to better dispersion of fibers, and modifying the fiber/matrix interface (E. H. Yang, Yang, and Li 2007). However, the decrease in compressive strength of ECC mixtures by increasing the content of fly ash could be attributed to two reasons. First, relatively high proportion of fly ash to cement may limit the secondary hydration reaction of fly ash, which may partially act as filler in the matrix (decreasing fracture toughness of the matrix) even after long periods of curing. Second, the low water/binder ratio (W/B=0.27) may lead to inadequate amount of water to promote the secondary hydration reaction between cement and fly ash (E. H. Yang, Yang, and Li 2007).

5.2 Flexural Performance

After 28 days of curing, the flexural performance of ECC beams was assessed according to third-point bending test similar to ASTM C 1609 (Flexural Performance of Fiber-Reinforced Concrete). Figure 5 presents the Flexural Stress vs. Deformation curves associated to each mix design. From the mixes evaluated in this study, M1 was the only mixture to exhibit a flexural behavior similar to that of FRC. While a peak load higher than the first-peak strength was observed in M1 (accompanied by a dramatic drop in flexural stress right after the first-peak strength), right after the second peak (peak strength), an almost complete loss of load carrying capacity occurred in a sudden manner with a single localized macro-crack failure as shown in Table 4. The strain-softening behavior of M1 can be attributed to an excessive strength of the cementitious matrix. The excessive strength of the matrix increased the crack tip toughness (J_{tip}) and the matrix first-cracking strength (σ_{fc}) to the extent of producing a reduction on J'_b/J_{tip} and σ_0/σ_{fc} parameters leading to not meeting either the strength or energy criteria. It is important to notice that J'_b and σ_0 will also be affected by the strengthening or weakening of the matrix (stronger matrix should tend to decrease J'_b and increase σ_0) as the fiber/matrix interface will be altered, yet the largest effects will occur on J_{tip} and σ_{fc}.

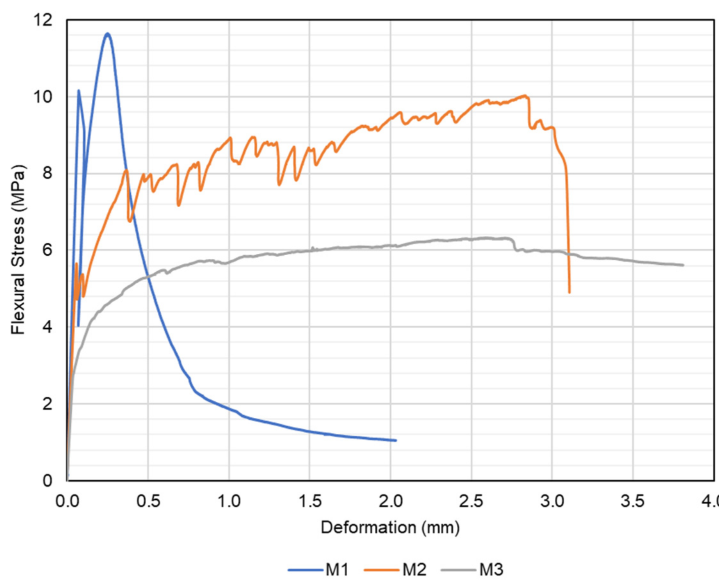
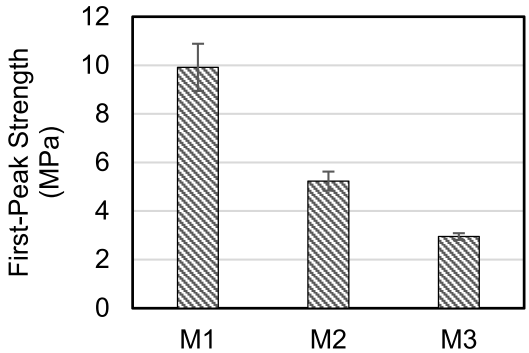


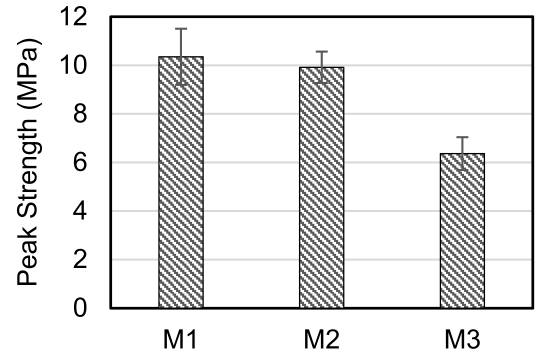
Figure 5: Flexural Stress vs. Deformation Curve (After 28 days of Curing)

As shown in Figure 5, both M2 and M3 exhibited a robust pseudo strain-hardening behavior exhibiting a first-peak strength with subsequent increase in load carrying capacity (accompanied by multiple steady-state micro-crack formation as shown in Table 4). In contrast to M1, the average first-peak strength (as presented in Figure 6a) of M2 (5.23 MPa) and M3 (2.93 MPa) were significantly lower than that of M1 (9.91 MPa) likely producing higher J'_b/J_{tip} and σ_0/σ_{fc} parameters and consequently enabling strain-hardening behavior.

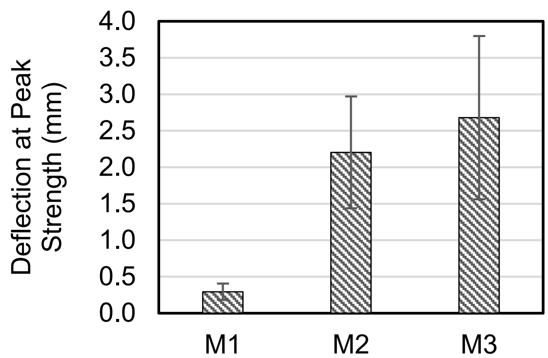
It is worth noting that while both M2 and M3 exhibited strain-hardening, M2 achieved a higher average peak-strength (9.92 MPa for M2 compared to 6.36 MPa for M3) but a lower average deformation capacity at peak strength (2.20 mm for M2 compared to 2.68 mm for M3) than that of M3 (as shown in Figures 6b and 6c). This behavior is explained by the differences in matrix strength between M2 and M3. As discussed above, a lower matrix strength favors the increase in the parameters controlling pseudo strain-hardening performance thus producing a more ductile composite with more and smaller steady-state microcracks like M3. However, there is a tradeoff between flexural strength of the composite and ductility, which highlights the implications of matrix/interface tailoring in the overall performance of ECC.



(a)



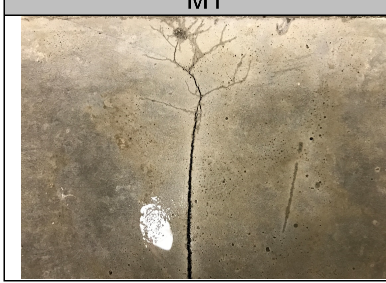
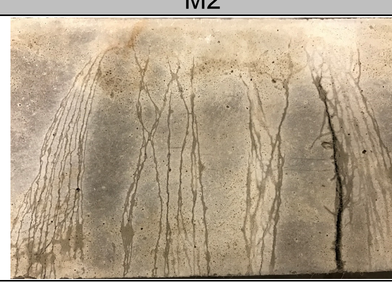
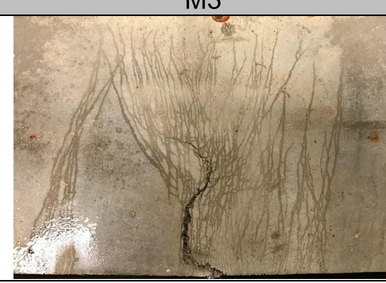
(b)



(c)

Figure 5: (a) Average First-Peak Strength (b) Average Peak Strength (c) Average Deflection at Peak Strength

Table 4: Beam Crack Patterns at Failure

M1	M2	M3
		

6 SUMMARY AND CONCLUSIONS

The development of PVA-ECC with locally available materials in the state of Louisiana was successfully achieved. Furthermore, the correlation between compressive strength, flexural strength, and ductility of PVA-ECC mixes to different levels of matrix/interface tailoring was investigated. The following conclusions can be drawn from the experimental results:

- The compressive strength of ECC mixtures was significantly affected by the fly ash content in the mix. Higher fly ash contents produced lower strengths as these limited the pozzolanic reaction of fly ash, which partially acted as a filler. The highest compressive strength reported of 54.3 MPa was achieved at 55% replacement of cement with fly ash (FA/C = 1.2) followed by 31.2 MPa and 16.2 MPa at 69% (FA/C = 2.2) and 81% (FA/C = 4.4) fly ash replacement, respectively.
- Third-point bending test results demonstrated that the increase in fly ash replacement was in favor of multiple cracking and strain hardening. Yet, a tradeoff between flexural strength and ductility was observed. M1 mixture, having the lowest fly ash content, did not exhibit strain-hardening likely due to failure to meet the strength and/or energy criterion, which allowed strain-softening behavior similar to that of FRC to prevail. On the other hand, M2 and M3 exhibited a robust strain hardening behavior with average peak flexural strengths of 9.92 MPa and 6.36 MPa, and average deflection at peak flexural strengths of 2.20 mm and 2.68 mm, respectively.

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