International Journal of Engineering Science Invention (IJESI)
ISSN (Online): 2319 – 6734, ISSN (Print): 2319 – 6726
www.ijesi.org ||Volume 7 Issue 6 Ver I || June 2018 || PP 35-42

Bond Performance of Mild Reinforcing Steel in Fiber Reinforced Cement-Limiting Concrete (FRCLC)

Trevor J. Looney[1], Jeffery S. Volz[1]
[1]*(Civil Engineering and Environmental Sciences/ University of Oklahoma, United States)*
Corresponding auther: Trevor J. Looney

Abstract: *Due to the emissions created during the production of cement, researchers have been developing new mix designs with less cement and comparable performance. One such mix design is called fiber reinforced cement-limiting concrete (FRCLC). FRCLC uses a lower paste volume, replacement of cement with fly ash, and polypropylenefibers to obtain comparable fresh and hardened properties to conventional concrete. This study focuses on the bond performance of FRCLC when compared to anOklahoma Department of Transportation Class AA conventional concrete (CC) mix design. Two different FRCLC mix designs were tested. The first, Eco-Bridge-Crete 1 (EBC1), contains 0.5 lb/yd^3 of micro-fibers. The second, Eco-Bridge-Crete 2 (EBC2), had the same dosageof micro-fibers as EBC1, along with 3.0 lbs./yd^3 of macro-fibers. All concrete mixes tested used 20% replacement of cement with Class C fly ash. Beam splice test specimens were used to evaluate the bond performance. Test results showed that EBC1 had comparable bond strengths to the CC mix design, while EBC2 was much weaker in bond.*

Keywords –*Cement-Limiting, Bond Performance, Mild Reinforcing, Fiber Reinforced Concrete, Optimized Aggregate Distribution*

Date of Submission: 18-05-2018 Date of acceptance: 04-06-2018

I. INTRODUCTION

Portland cement concrete is a very versatile construction material. Its simple ingredients, ease in mixing and placing, and high strength has made it one of the most popular construction materials around the world. The main ingredient in conventional concrete and the binding agent is the cement. Unfortunately, the production of cement emits large quantities of greenhouse gases into the environment. According to the United States Geological Survey, approximately 83.5 million tons of cement was produced in the United States in 2017 [1]. Since approximately every pound of cement produced leads to the emission of one pound of CO_2, cement production becomes a large emission source.

With greenhouse gas emissions becoming a rising concern, the construction industry is looking for ways to reduce the amount of cement production. Two approaches to making concrete a more sustainable material and lessening its environmental impact is to reduce the amount of cement used in the mix while maintaining mechanical properties similar to conventional concrete. This can be accomplished by either replacing the cement with supplementary cementitious materials, or by using less cement while optimizing the aggregate distribution.

Research has been conducted to prove the comparable performance of concrete mix designs using supplementary cementitious material to reduce the cement required. Mohamed et al. [2] studied the effects of incorporating varying percentage replacements of cement with fly ash. The studied replacement percentages were 0%, 20%, 40%, and 60%. The water-to-cementitious-material ratio was kept at 0.4 for all mixes to ensure the only change was the volume of fly ash used. The compressive strength was tested using cubes and the modulus of elasticity was determined using cylinders. The results showed that the compressive strength and the modulus of elasticity of the concrete with 40% fly ash had comparable strengths as the control mix design at 56 days[2].

Several studies have also been conducted to assess the bond performance of various concrete mix designs. One such study by Looney et al. [3] compared the bond strength of self-consolidating concrete (SCC) to conventional concrete. The bond strength of two different SCC mix designs, one normal strength and one high strength, were compared to conventional concrete. The high strength SCC used 20% replacement of cement with fly ash. The study consisted of testing three full-scale beam splice specimens for each type of concrete to compare bond strengths. The beams were 10 ft. long, with a 12 in x 18 in. cross section. The results showed that both SCC mix designs had comparable bond strengths to the conventional mix design [3].

Another study on bond strength comparisons was conducted on high-volume fly ash concrete (HVFAC) by Looney et al. [4]. The bond strength of two different HVFAC mix designs, both with 70%

replacement of cement with fly ash and one with a high paste volume and one with a low paste volume, were compared to the bond strength of a conventional concrete mix design. Three full-scale beam splice specimens were tested for each type of concrete to compare bond strengths. The beams were 10 ft. long, with a 12 in x 18 in. cross section. The researchers concluded that both HVFAC mix designs had a higher bond strength than the conventional concrete mix design, with the low paste volume HVFAC havinga lower bond strength than the high paste volume HVFAC [4].

This study focuses on using beam splice specimens to assess the bond performance of two fiber reinforced cement-limiting concrete (FRCLC) mix designs when compared to aClass AA conventional concrete (CC) mix design used by the Oklahoma Department of Transportation (ODOT).

II. EXPERIMENTAL PROGRAM

2.1 Concrete Materials

The control CC mix design used Type I/II portland cement manufactured by Ash Grove Cement Company (Chanute, KS) conforming to ASTM C150 . The coarse aggregate was a #57crushed limestone supplied by the Dolese Bros. Co. Davis Quarry (Davis, OK).The fine aggregate was a concrete sand also supplied by the Dolese Bros. Co. from their East Sand Plant (Oklahoma City, OK). Each aggregate conformed to ASTM C33. Chemical admixtures from BASF (Florham Park, NJ) were also used to improve workability and increase the air content of the concrete mix. The high-range water reducing admixture MasterGleniuim 7920 was used to improve workability. The air entraining admixture MasterAir AE 90 was also selected. Both admixtures were chosen due to their local popularity and superior performance.

Two FRCLC mix designs were used in this study and were labeled Eco-Bridge-Crete 1 (EBC1) and Eco-Bridge-Crete 2 (EBC2). Each mix used the same cement, sand, and chemical admixtures as the CC mix design. The supplementary cementitious material used for this mix was Class C fly ash supplied by Headwaters Resources (Jeffery Plant, St. Mary’s, KS). The coarse aggregate used was the #57 crushed limestone supplemented with a 3/8” chipped limestone supplied by Metro Materials (Norman, OK). Each FRCLC mix design used fiber reinforcement to enhance their mechanical properties. EBC1 used micro-fibers at a dosageof 0.5 lbs./yd^3 and EBC2 used the same micro-fiber dosageand a macro-fiber dose of 3 lbs./yd^3. The fibers used were both polypropylene and were supplied by BASF (Florham Park, NJ). The micro-fibers were called MasterFiber M 100 and the macro-fibers were called MasterFiber MAC Matrix. The properties of each fiber type are shown in Table 1. The final mix proportions used for each type of concrete tested are shown in Table 2.

2.2 Concrete Properties

The fresh and hardened properties of each mix design tested are shown in Table 3 along with the applicable ASTM procedure employed for testing. The compressive strength was determined using 4 in. x 8 in. cylinders. The modulus of rupture was determined using a prism with a 6 in. x 6 in. cross section and span length of 18 in. The split cylinder strength was determined using 6 in. x 12 in. cylinders. The tested strengths were intentionally less than the design strength to ensure bond failure of the beam splice specimens.

2.3 Beam Splice Specimen Design

Three beam splice test specimens were fabricated for each concrete tested in this study, for a total of nine beams. In order to assess the bond performance of the different concrete mix designs, a testing setup based on ACI 408R-2003 [5] and previous research was used in the design of the beam splice specimens in this study. A non-ASTM testing procedure was used based on the research of Wolfe [6], Looney [7], and Steele [8].The beams used were 10 ft. long with a cross section of 12 in. x 18 in.The bottom longitudinal reinforcing were three #6 bars that were spliced at midspan and terminated at the ends with a 90° hook. The splice length was determined using the design strength of each concrete then reducing the development length calculated using (1):

$$l_d = \left(\frac{3}{40} \frac{f_y}{\lambda \sqrt{f'_c}} \frac{\Psi_t \Psi_e \Psi_s}{\left(\frac{c_b + K_{tr}}{d_b} \right)} \right) d_b \tag{1}$$

where l_d is the development length, f_y is the specified yield strength of reinforcement, λ is the lightweight concrete modification factor, f'_c is the specified compressive strength of concrete, Ψ_t is the reinforcement location modification factor, Ψ_e is the reinforcement coating modification factor, Ψ_s is the reinforcement size modification factor, c_b is the smaller of the distance from center of a bar to nearest concrete surface or one-half the center-to-center spacing of bars being developed, K_{tr} is the transverse reinforcement index, and d_b is the nominal diameter of reinforcing bar [9]. The calculated bar development length was then reduced by 30%, based on bond research by Looney [7], to ensure a bond failure prior to the bars yielding.

Strain gages were placed on each bar at the end of the splice to measure the strain and the measured strain was used to calculate the stress at failure. That calculated stress was taken as the bond stress upon bond failure. The transverse shear reinforcing were #3 U-shaped stirrups that were terminated with a 180° hook. All reinforcing bars used in this study was Grade 60, deformed mild steel bar conforming to ASTM A615. The reinforcing bar was subject to a direct tension according to ASTM E8 to determine that the bar yield stress was 78.2 ksi. A plan view and section view of the beam splice specimens are shown in Fig. 1 and Fig. 2, respectively.

2.4 Fabrication And Curing Procedure

The concrete mix design used for this study was sent to a local ready-mix concrete plant, where it was proportioned and mixed then delivered to Fears Laboratory at the University of Oklahoma. After casting, all the beam test specimens and associated mechanical property specimens were covered in wet burlap and plastic to maintain a moist environment. The specimens were removed from their molds after two days and subsequently left under burlap and plastic for a total of seven days. The test specimens were then cured at ambient temperature and humidity until they reach an appropriate strength for testing. Each set of specimens was tested before reaching its respective design strength to ensure a bond failure.

2.5 Beam Splice Specimen Test Procedure

Each test specimen was subjected to third-point loading, creating a constant moment region between the load points where the reinforcing splice is located. A diagram of the test setup is shown in Fig. 3. The span length of the test setup was nine feet and the load points were three feet apart and three feet from each simple support. The actual test setup is shown in Fig. 4. A spreader beam was used to transfer the force from the hydraulic jack to the load points. A 100 kip load cell was placed on the spreader beam to monitor the applied load. A string pot was used to measure the midspan deflection and the strain gages installed on the reinforcing bars were used to measure strain in the longitudinal reinforcement. The load was applied in 10 kip increments until failure and cracks were marked on the beam as they formed.

III. TEST RESULTS AND DISCUSSION

All splice specimens exhibited bond failure prior to yielding of the reinforcing steel. Figure 5 shows an example bond failure of a beam splice specimen. The peak load reached for each beam splice test is shown in Table 4. The peaked measured and normalized stresses are shown in Table 5. To compare the bond performance of the two Eco-Bridge-Crete mixes to the control mix design, the data was normalized using the square root normalization method and the fourth root normalization method. Using the development length equation from ACI 318-14 (Equation 25.4.2.3a) [9], the development length is indirectly proportional to the compressive strength of the concrete. Since the three concrete mixes all had different design strengths and test day strengths, the measured peak stress in the steel was multiplied by the ratio of the square root of the design concrete strength to the test day concrete strength.

Load deflection plots for the CC, EBC1, and EBC2 beams are shown in Fig. 6, Fig. 7, and Fig. 8, respectively. Fig. 9 shows the average square root normalized peak bond stress for each concrete type. The maximum measured steel stresses in each of the concrete mixes was well below the 78.2 ksi yield stress of the longitudinal reinforcement. This verifies all beam splice specimens exhibited bond failures. The EBC1 bond strength compared to the CC was 8.9% higher when comparing the square root normalization values and 5.4% higher when comparing the fourth root normalization. The EBC2 bond strength compared to the CC was 23.6% lower based on the square root normalization and was 22.8% lower based on the fourth root normalization.

The beam splice results from this study were compared to the ACI Committee 408 database for bond testing. Figure10 shows the beam splice data is in the higher range of data points for their respective concrete compressive strengths. However, there is no clear trend in the database to come to a definitive conclusion on the bond performance of the tests.

IV. CONCLUSIONS

A total of nine beam splice specimens were tested under third-point loading to determine the bond performance of FRCLC compared to anODOT Class AA concrete mix. The following are the findings of this study:

- EBC1 and EBC2 performed at the same level as the ODOT Class AA mix design in terms of fresh and hardened properties.
- All beam splice test specimens failed in bond prior to the reinforcing bars yielding.
- EBC1 was reinforced with only micro-fibers and provided a comparable bond strength to the ODOT Class AA mix.
- EBC2 was reinforced with both micro-fibers and macro-fibers and had a much lower bond strength.

- Since EBC1 and EBC2 only differed in the fiber type, the macro-fibers appear to have a negative impact on bond strength, potentially due to their size.
- The macro-fibers could have created some honeycombing around the reinforcing bar due to their size, even though there was no evidence of this on the concrete surface.

The results of this study shows that an eco-friendly, cement-limiting concrete can perform as well as a conventional concrete in terms of bond strength. However, further research is necessary to determine how the macro-fibers effect bond strength.

Acknowledgements

The authors gratefully acknowledge the financial support provided by the Tier 1 University Transportation Center at Missouri University of Science and Technology. The authors would also like to thank the support staff in the School of Civil Engineering and Environmental Science at the University of Oklahoma. The conclusions and opinions expressed in this paper are those of the authors and do not necessarily reflect the official views or policies of the funding institutions.

REFERENCES

[1]. United States Geological Survey, "Mineral Commodities Summaries," January 2018.
[2]. Mohamad, N., Zulaika, M.S., Samad, A.A.A., Goh, W.I., Hadipramana, J., Wirdawati, A., "Fresh State and Mechanical Properties of Self Compacting Concrete Incorporating High Volume Fly Ash," MATEC Web of Conferences, 47 (1001), 2016, pp. 1-7.
[3]. Looney, T.J., Arezoumandi, M., Volz, J.S., Myers, J.J., "An Experimental Study on Bond Strength of Reinforcing Steel in Self-Consolidating Concrete," International Journal of Concrete Structures and Materials, 6 (3), Oct. 2012, pp. 187-197.
[4]. Looney, T.J., Arezoumandi, M., Volz, J.S., Myers, J.J., "An Experimental Study on Bond Strength of Reinforcing Steel inn High-Volume Fly Ash Concrete," Advances in Civil Engineering Materials, 1 (1), Nov. 2012, pp. 1-17.
[5]. ACI 408R, "Bond and Development of Straight Reinforcing Bars in Tension," American Concrete Institute, 2003, Farminton Hills, MI.
[6]. Wolfe, M. "Bond Strength of High-Volume Fly Ash Concrete," M.S. Thesis, 2011, Missouri University of Science and Technology, Rolla, MO.
[7]. Looney, T.J., "Bond Behavior of High-Volume Fly Ash and Self-Consolidating Concrete," M.S. Thesis, 2012, Missouri University of Science and Technology, Rolla, MO
[8]. Steele, A.R., "Bond Performance of Recycled Aggregate Concrete," M.S. Thesis, 2014, Missouri University of Science and Technology, Rolla, MO
[9]. ACI 318, "Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary," American Concrete Institute, 2014, Farminton Hills, MI.

Table 1: Fiber Reinforcement Properties

Property	MasterFiber M 100	MasterFiber MAC Matrix
Specific gravity	0.91	0.91
Absorption	Negligible	Negligible
Tensile strength, ksi.	70	85
Nominal length, in.	0.75	2.1
Nominal diameter, in.	0.00047	0.03
Material	Polypropylene	Polypropylene
Notes: 1 in. = 25.4 mm; 1 ksi = 6.89 MPa		

Table 2: Concrete Mix Proportions per Cubic Yard

	CC	EBC1	EBC2
Type I/II cement lbs.	470	414	414
Class C fly ash lbs.	118	103	103
w/cm	0.4	0.4	0.4
#57 limestone, lbs.	1857	989	989
3/8" chip, lbs.	—	565	565
Concrete sand, lbs.	1323	1415	1415
Micro-fiber, lbs.	—	0.5	0.5
Macro-fiber, lbs.	—	—	3
HRWR, fl. oz.	26.7	36.19	36.19
AEA, fl. oz.	4.4	2.59	2.59
Notes: 1 lb. = 0.454 kg; 1 fl. oz = 29.5 mL			

Table 3: Fresh and hardened properties of tested concrete

Property	ASTM	CC	EBC1	EBC2
Slump, in.	C143	3.25	3.00	3.75
Air Content, %	C231	6.4	6.0	5.6
Unit Weight, lbs./ft^3	C138	146	144	144
Split Cylinder Strength, psi	C493	290	340	550
Modulus of Rupture, psi	C78	630	545	670
Compressive Strength, psi	C39	4350	4220	5490

Design Strength, psi	---	5000	5500	6000
Notes: 1 in. = 25.4 mm; 1 ksi = 6.89 MPa				

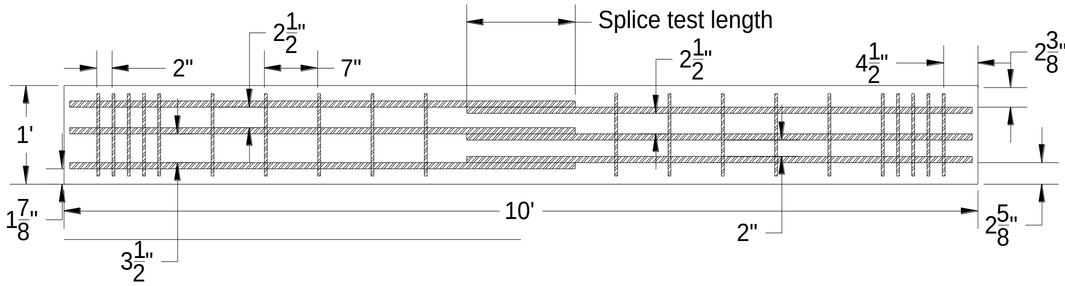

Figure 1: Plan view of beam splice specimen reinforcing

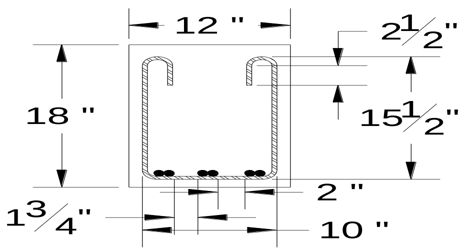

Figure 2: Section view of beam splice specimen reinforcing

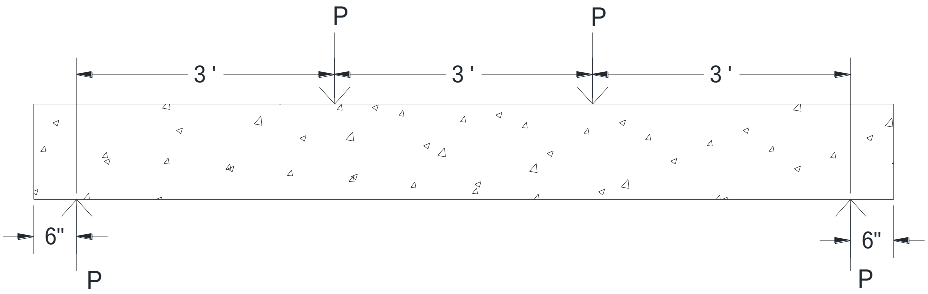

Figure 3: Third point loading on beam splice specimens

Figure 4: Beam splice test setup

Table 4: Summary of failure load of bond specimens

	Failure Load (kips)
CC-1	52.6
CC-2	52.2
CC-3	54.2
EBC1-1	45.1
EBC1-2	47.5
EBC1-3	43.7
EBC2-1	49.5
EBC2-2	40.9
EBC2-3	49.5

Table 5: Summary of measured and normalized bond stress at failure

Specimen	Steel Stress (ksi)	Design Strength (psi)	Test Day Strength (psi)	Square Root Normalization (ksi)	Average Square Root Normalization (ksi)	Fourth Root Normalization (ksi)	Average Fourth Root Normalization (ksi)
CC-1	69.5	5000	4353	74.5	71.1	72	68.7
CC-2	65.7			70.4		68.1	
CC-3	63.8			68.3		66	
EBC1-1	62.4	5500	4222	71.3	77.4	66.7	72.4
EBC1-2	73.3			83.7		78.4	
EBC1-3	67.6			77.1		72.2	
EBC2-1	56.7	6000	5487	59.3	54.3	58	53.1
EBC2-2	43.5			45.5		44.5	
EBC2-3	55.6			58.2		56.9	

Figure 5: Typical bond failure in beam splice specimen

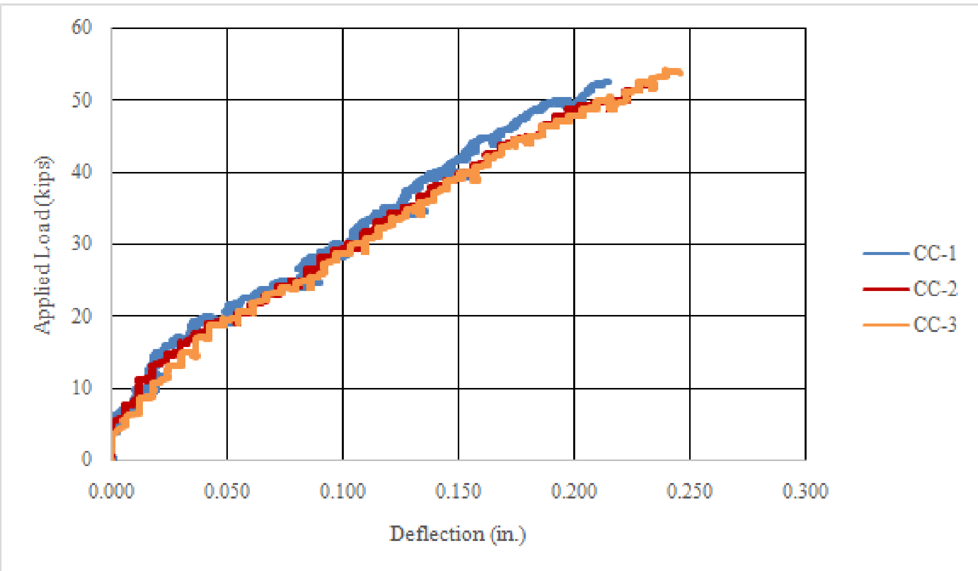

Figure 6: Load vs. deflection plot of CC beams

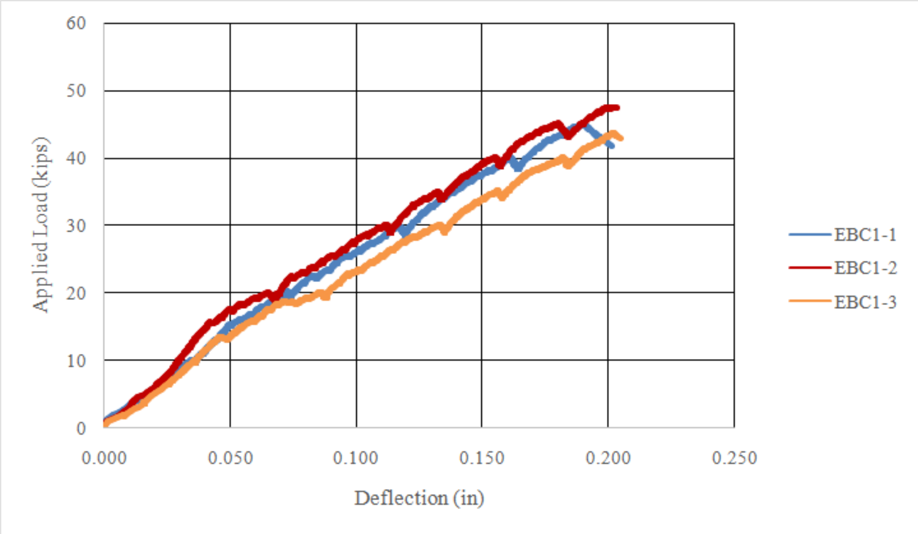

Figure 7: Load vs. deflection plot of EBC1 beams

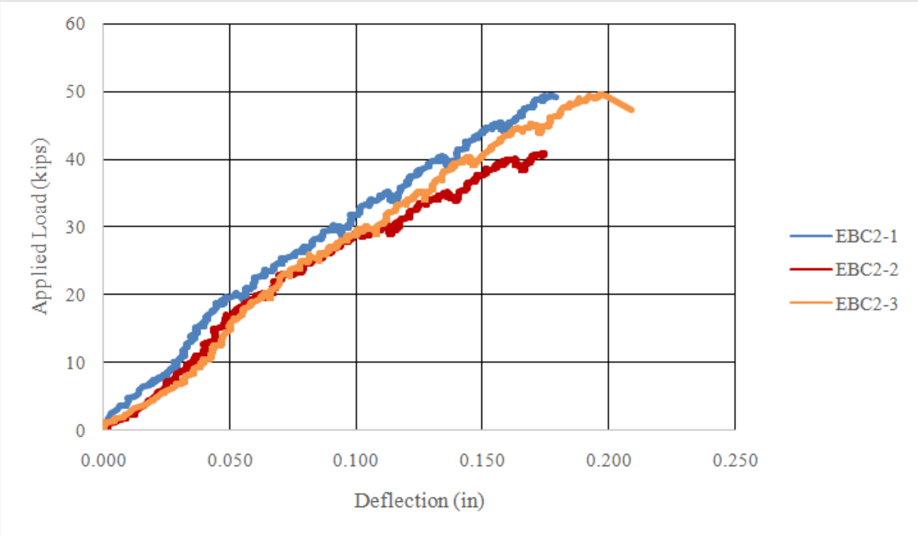

Figure 8: Load vs. deflection plot of EBC2 beams

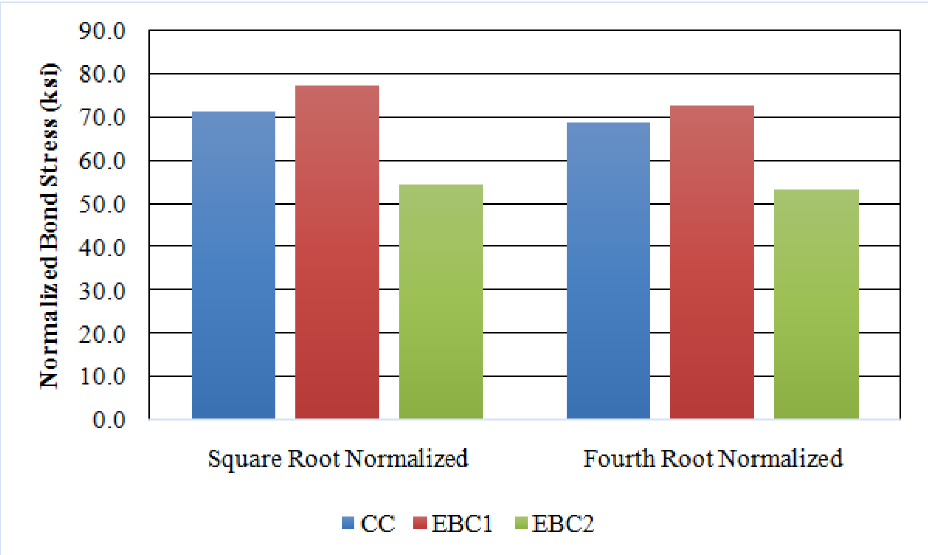

Figure 9: Average normalized peak bond stress for each concrete

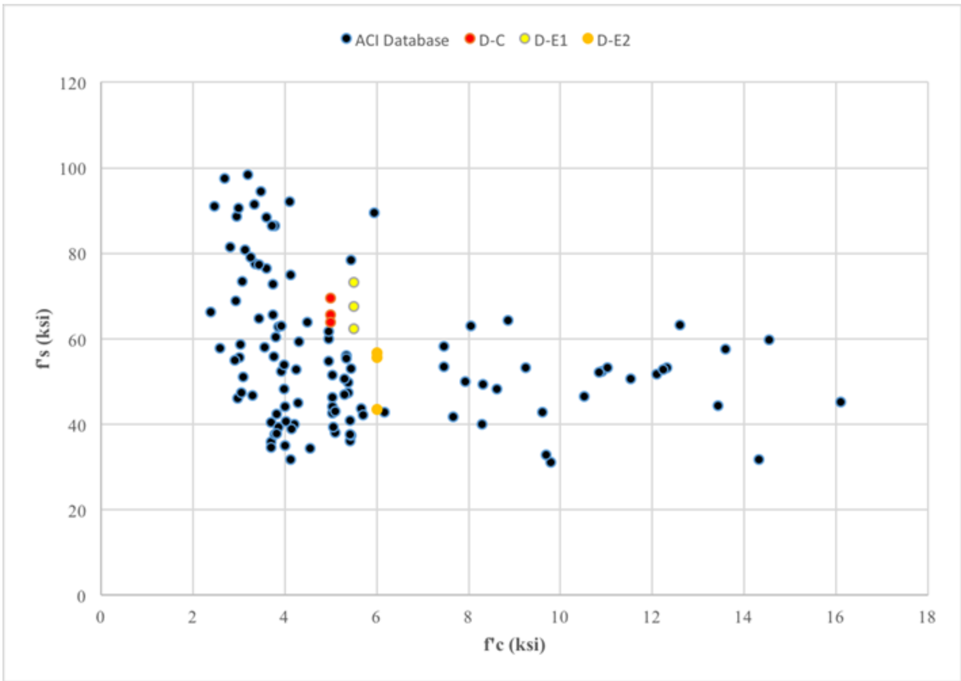

Figure 10: ACI 408 database comparison

Trevor J. Looney "Bond Performance of Mild Reinforcing Steel in Fiber Reinforced Cement-Limiting Concrete (FRCLC) "International Journal of Engineering Science Invention (IJESI), vol. 07, no. 06, 2018, pp 35-42