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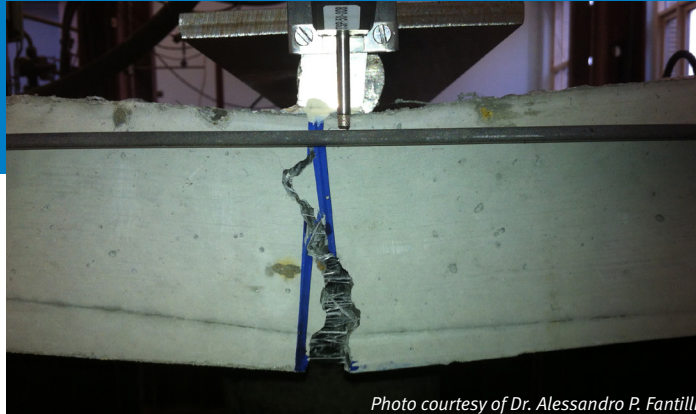


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Reduction of Crack Width with Fiber

Editors:

Corina-Maria Aldea and Mahmut Ekenel

SP-319



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Preface

Fiber reinforcement is the most effective way of improving the resistance of concrete to cracking, but little is known of the extent of the reduction of crack width with fiber. The papers included in this special publication discuss the role of fiber reinforcement in reduction of crack width and lay the foundation for Life Cycle Engineering Analysis with fiber reinforced concrete.

Recognizing the reduction of crack width with fibers in cement-based materials, ACI Committee 544 Fiber Reinforced Concrete, together with 544F Fiber Reinforced Concrete Durability and Physical Properties sponsored two technical sessions entitled Reduction of crack width with fiber at the Fall 2016 ACI Convention in Philadelphia. Papers were presented by invited international experts from Belgium, France, Germany, Italy, Portugal, United Arab Emirates and the United States of America.

This Symposium Publication (SP) contains eleven papers which provide insight on the state of the art of the topic in the academia, in the industry and in real life applications. The topics of the papers cover the reduction of crack widths in steel reinforced concrete bridge decks with fiber, 15 years of applying SFRC for crack control in design from theory to practice, the effectiveness of macro synthetic fibers to control cracking in composite metal decks, conventional and unconventional approaches for the evaluation of crack width in fiber reinforced concrete (FRC) structures, reduction of water inflow by controlling cracks in tunnel linings using fiber reinforcement, a review of Engineering Cementitious Composites (ECC) for improved crack-width control of FRC beams, tailoring a new restrained shrinkage test for fiber reinforced concrete, a model to predict the crack width of FRC members reinforced with longitudinal bars, a probabilistic explicit cracking model for analyzing the cracking process of FRC structures, toughening of cement composites with wollastonite sub micro-fibers and self healing of FRC: a new value of “crack width” based design. The papers included in this publication have been peer reviewed by international experts in the field according to the guidelines established by the American Concrete Institute.

On behalf of ACI Committee 544 Fiber Reinforced Concrete and committee 544F Fiber Reinforced Concrete Durability and Physical Properties, the editors would like to thank all the authors for their contributions and the reviewers for their assistance, valuable suggestions and comments.

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Reduction of Crack Widths in Steel Reinforced Concrete Bridge Decks with Fiber Addition

Anil Patnaik, Prince Baah, Perry Ricciardi, and Waseem Khalifa

Synopsis: Bridge deck cracking is a common problem in the United States, and affects the durability and service life of reinforced concrete bridges. Physical inspections of three-span structural slab bridges in Ohio revealed cracks wider than $\frac{1}{8}$ inch (3.2 mm). ACI 224R-01 recommends a maximum crack width of 0.007 inch (0.18 mm) for members exposed to de-icing chemicals. The primary objective of this study was to investigate the effects of fiber addition on crack resistance. In an attempt to minimize deck cracking, slab specimens with basalt MiniBar or polypropylene fiber were also investigated in the test program. Slab tests revealed that the specimens with longitudinal epoxy-coated bars developed first crack at smaller loads, exhibited wider cracks and a larger number of cracks, and failed at smaller ultimate loads compared to the corresponding test specimens with uncoated (black) bars. Test specimens with fiber exhibited higher cracking loads, smaller crack widths, smaller mid-span deflections and higher ultimate failure loads compared to identical specimens without fiber. Addition of fiber to concrete with no changes to internal steel reinforcement details is expected to reduce the severity and extent of cracking in reinforced concrete bridge decks demonstrating that fiber addition improves crack resistance of bridge decks.

Keywords: Bridge deck cracking; continuous span structural slab bridges; crack widths; epoxy coated bars; fiber reinforced concrete; load testing; deck slabs; crack resistance

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INTRODUCTION

Non-prestressed steel reinforced concrete solid structural slab bridges are commonly used by several Departments of Transportation (DOTs) in the United States. Structurally, a continuous deck slab runs parallel to the longitudinal axis of the bridge, and is supported by the abutments at the ends and piers at intermediate locations. The decks typically are between 11 inch (280 mm) and 27 inch (685 mm) in thickness.

One of the primary factors affecting concrete bridge durability is deck cracking. Cracks are caused primarily by low tensile strength of the concrete, volumetric instability, and/or deleterious chemical reactions. Crack openings and spacing are affected by bar size and the effective concrete area surrounding the bar (Soltani et al. 2013). Regardless of the causes, cracking on bridge decks is a serious concern, because cracks provide access to harmful, corrosive chemicals that deteriorate the reinforcing steel embedded within the concrete. Once chloride and other corrosive agents penetrate concrete, corrosion of the embedded steel can initiate and cause concrete spalling. Such deterioration can affect the shear and moment capacity of reinforced concrete bridge decks. Also, bridge deck cracks allow water and de-icing salts to flow down through the deck and can also damage to the substructure (Krauss and Rogalla 2003). According to a survey of 52 transportation agencies across North America, more than 100,000 bridges were found to crack early (McDonald et al. 1995), and in some cases, typically when concrete is just one month old (Patnaik and Wehbe 2015).

In 2002, corrosion of the reinforcing steel in concrete was estimated to have an annual direct cost to highway bridges of \$8.3 billion. However, the indirect cost to users due to traffic delays and lost productivity was estimated to be ten times as much (Yunivich et al. 2002). Replacement costs for bridge decks are a significant portion of that direct cost (Virmani and Clemena 1998). Cracks frequently form relatively early in the life of concrete bridge decks, at times well in advance of a bridge being open to traffic, and sometimes immediately following construction (Schmitt and Darwin, 1995, Patnaik and Wehbe, 2015). Concrete bridge deck cracking is influenced by several conditions, including construction practices, concrete mix proportions, material properties, structural design, and loading levels (Ramakrishnan and Patnaik 2006; Cavaliero and Durham 2011).

The addition of fiber to improve cracking behavior of reinforced concrete is gaining some recognition. One of the important features of fiber reinforced concrete is the ability of the fiber to bridge across cracks. Fibers, when added to concrete, modify the cracking mechanism from macro cracking to micro cracking. The results are that crack widths are reduced, and the ultimate tensile cracking strain capacity of the concrete is increased. The mechanical bond between the embedded fiber and binder matrix redistributes the stresses. Additionally, the ability to modify the cracking mode results in quantifiable benefits. Reduced micro cracking leads to reduced permeability and increased surface abrasion resistance, impact resistance and fatigue strength (Adhikari and Patnaik 2012;

Patnaik et al. 2015). There are many different metallic and non-metallic micro or macro fiber available for use in fiber reinforced concrete.

A new type of macro fiber known as MiniBars was recently made from basalt fiber and is gaining acceptance in the industry. The addition of fiber to concrete will also improve the bond strength between the reinforcing bar and the surrounding concrete (Banibayat and Patnaik 2014; Grace et al. 2011; Grace et al. 2012, Baah 2012). This is particularly useful for epoxy-coated bars (ECB) because these are demonstrated in this study to have larger cracking potential, maybe due to inferior bond strength. Patnaik (2011) and Patnaik et al. (2013) performed a comprehensive study on the mechanical and structural characterization of concrete reinforced with basalt MiniBars. Bagherzadeh et al. (2012) studied the influence of polypropylene fiber using different proportions and fiber lengths to improve the performance characteristics of the lightweight cement composites.

This paper presents the results of a recent bridge investigation on twelve, three-span continuous reinforced concrete structural slab bridges in Ohio. These bridges were carefully selected from the bridge inventory in the state. These bridges vary in terms of span lengths, roadway width, skew angle, deck thickness, number of lanes, reinforcement ratio, and geographic location within the state. The study focused on the wide transverse cracks primarily in the direction parallel to the intermediate pier supports. Section and crack width analyses for select bridges were performed to compare the field-recorded crack widths to the theoretical crack widths determined using the three most common equations for predicting crack widths. The Ohio Department of Transportation (ODOT) uses epoxy-coated bars (ECB) in bridge decks. An experimental program was set up, and tests were conducted to examine the flexural cracking behavior of laboratory-scale slabs representing the intermediate support region of typical bridge decks using both ECB and conventional uncoated reinforcing bars, commonly referred to in the construction industry as “black bars”. The effects of addition of fiber on concrete cracking were also investigated. Variables studied in the experimental investigations included type of bar (ECB or black), bar size, with or without fiber, and specimen preparation (precut or non-precut). DIC (Digital Image Correlation) was used to obtain crack width measurements for one bridge included in this study. DIC was also used in the field investigation to measure crack widening under both static and moving truck loads. Several DOTs routinely design, build, and maintain a large number of three-span continuous structural slab bridges. The sheer number of such bridges in the states necessitated a systematic study to determine the extent of the problem in structural slab bridge decks so that the causes of cracking can be identified and countermeasures established to minimize cracking in future bridge deck construction. Because several DOTs also use ECB in bridge decks, the need to investigate the effects of the ECB on cracking behavior of concrete was identified as an immediate need.

RESEARCH SIGNIFICANCE

One of the primary factors affecting concrete bridge durability is deck cracking. Despite significant research specifically studying the problem, cracking in reinforced concrete bridge decks is still a widespread concern in old and newly-constructed bridges. Previous research conducted by the authors focused mainly on shrinkage cracking of stringer supported bridge decks (Ganapuram et al. 2012; Patnaik and Wehbe 2015). However, limited research is available on the cracking behavior of structural slab bridge decks. This investigation addresses the cracking problems identified in continuous structural slab bridge decks with an emphasis on “non-shrinkage” cracks. To investigate a preventive measure to deck cracking, this study examines the effects of the addition of basalt MiniBar and polypropylene fiber to reinforced concrete on deck cracking. Addition of fiber to concrete without changing any steel reinforcing details is expected to cost-effectively reduce the severity and extent of cracking in reinforced concrete bridge decks.

Also, the effect of epoxy coating on bond and anchorage behavior of reinforcing bars has been studied by several investigators (Treece and Jirsa 1989). However, little research has been done on the influence of ECB on crack control (Mitchell et al. 1996). This study also addresses the effect of epoxy-coating on deck cracking.

The adequacy of merely satisfying the reinforcement spacing requirements given in AASHTO or ACI 318-14 to limit cracking below the ACI 224R-01 recommended maximum limit, even though all the relevant design requirements are otherwise met, is an important factor that remains to be fully understood.

ALLOWABLE CRACK WIDTH LIMITS

To minimize the adverse effects of cracks on reinforced concrete bridge decks, the design should ensure that the crack widths under normal service conditions are within recommended allowable limits. The cracking of a reinforced concrete slab at service loads should not impact the appearance of the structure or lead to corrosion of the embedded steel reinforcement. According to ACI Committee report 224, crack widths equal to or greater than 0.007 in. (0.18 mm) can reduce durability when bridge decks are exposed to de-icing chemicals (ACI 224R-01 2008). Similar limits are set for other exposure conditions (ACI 224R-01 2008). Crack widths in the range of 0.01 in.

(0.254 mm) to 0.015 in. (0.381 mm) are acceptable from aesthetic considerations (ACI 224R-01 2008). Table 1 shows a summary of the maximum allowable crack widths, as suggested in ACI 224 report.

Table 1 – Allowable crack widths (ACI 224R-01, 2008)

Exposure Condition	Maximum Allowable Crack Width in. (mm)
Dry Air	0.016 (0.41)
Humidity, Moist Air, Soil	0.012 (0.30)
Deicing Chemicals	0.007 (0.18)
Sea Water	0.006 (0.15)
Water Retaining Structures	0.004 (0.10)

ACI 318-2014 CRACK CONTROL REQUIREMENT

Currently, the ACI 318-14 requirements are based on the belief that it can be misleading to calculate explicit crack widths, given the inherent variability in cracking. The three important parameters in flexural cracking are steel stress, cover, and bar spacing; of these, steel stress is the most important parameter. The design basis has been switched in recent years to the premise that crack width is not directly related to long-term durability, with cover depth and concrete quality being of greater importance. It can be misleading to use a design method that purports to effectively calculate crack widths. A re-evaluation of crack width data (Frosch 1999) provided a new equation based on the physical phenomenon for the determination of the flexural crack widths of reinforced concrete members. This study showed that previous crack width equations are valid for a relatively narrow range of covers up to 2.5 in. ACI 318-14 does not make distinction between interior and exterior exposure. The ACI 318-14 equation for determining the maximum spacing of flexural reinforcement closest to the tension face to affect adequate crack control is given below:

$$s = 15 \left(\frac{40,000}{f_s} \right) - 2.5c_c \leq 12 \left(\frac{40,000}{f_s} \right) \quad (1)$$

Where: f_s = calculated stress in reinforcement closest to the tension face at service load, which is computed based on the unfactored moment. It is permitted to take f_s as $2/3f_y$; c_c = the least distance from surface of reinforcement to the tension face; and s = center-to-center spacing of flexural tension reinforcement nearest to the surface of the extreme tension face (in.)

AASHTO 2012 CRACK CONTROL REQUIREMENT

The American Association of State Highway and Transportation Officials (AASHTO 2012) equation for determining the maximum spacing of flexural reinforcement in the layer closest to the tension face to affect adequate crack control is given below:

$$S \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \quad (2)$$

$$\text{in which: } \beta_s = 1 + \frac{d_c}{0.7(h-d_c)} \quad (3)$$

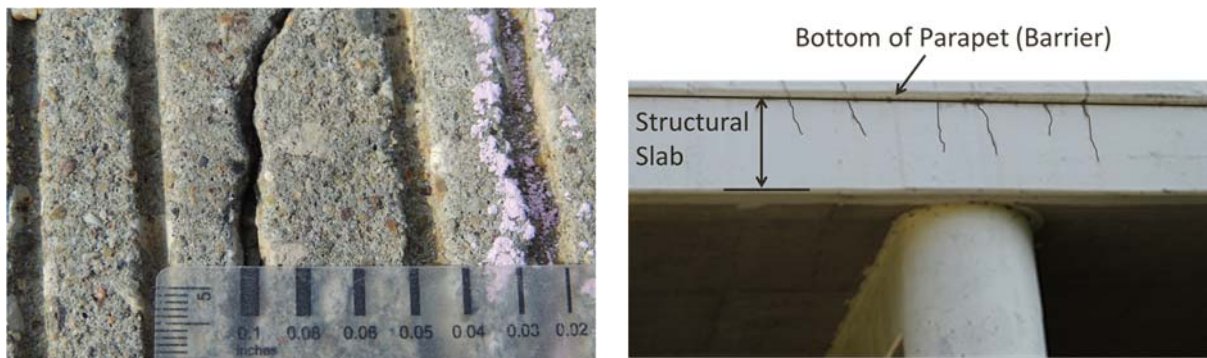
where γ_e is an exposure factor (equal to 1.00 for a Class 1 exposure condition or 0.75 for a Class 2 exposure condition); d_c = thickness of concrete cover measured from extreme fiber to the center of the flexural reinforcement located closest to the tension surface (in.); f_{ss} = the tensile stress in steel reinforcement at the service limit state (ksi); h = overall thickness or depth of the member (in.); and s = spacing of reinforcement.

FIELD INVESTIGATION

Twelve structural slab bridges with three sets of spans were studied during the field investigation. The three spans for the bridges investigated ranged from 24-30-24 ft (7.3-9.2-7.3 m) to 44-55-44 ft (13.4-16.8-13.4 m). The age of the bridge decks ranged from two to thirteen years. Crack patterns and maximum crack widths were recorded. With regard to the cracks on the top surface of the bridge deck, this study focused on the negative moment regions where cracks as wide as 0.03 in. (0.76 mm) to 0.125 in. (3.18 mm) were observed in the direction generally parallel to the intermediate pier supports. Design, material, and construction-related data are currently being collected for each bridge to determine if there is any correlation among these factors and the corresponding bridge deck cracking. The

bridges surveyed consisted of both skewed (up to 30 degrees) and non-skewed bridges, with bridge decks designed using different types of concretes. For some bridges, the maximum recorded crack width over the intermediate pier supports was more than 0.1 in. (2.54 mm), as can be seen in Figure 1 (a). These cracks, when observed from the side of the decks, extended in some cases for almost full depth through the deck, as shown in Figure 1 (b). It is worth repeating that with dead loads only, these crack width measurements are more than ten times the limit recommended in ACI 224 report.

Transverse cracks were also observed at the bottom of the bridge deck in the middle of the spans for some of the bridges surveyed. Bottom cracks (if any) were fewer than those on the top surfaces near supports and are not as critical from a point of view of the exposure to deicing salts. In addition to the cracks observed at the top and bottom of decks, cracks that were predominantly through-thickness cracks on the side faces confirmed that cracks extended deep into the deck thickness. One of the bridges was opened to traffic for just three months at the time of the inspection, and had developed cracks as wide as 0.03 in. (0.76 mm) on the top of the deck within three months. Typical crack surveys performed on the slab bridges revealed several wide cracks on the bridge decks and wide cracks over the piers in the negative moment region, as shown in Figure 2. Figure 3 shows a typical crack map on the top surface predominantly in the direction nearly parallel to one of the intermediate pier supports. Two crack surveys, conducted 22 months apart of one of the bridges, showed that both the crack widths and the extent of cracking were progressing with time (Baah 2014; Baah et al. 2014, Patnaik and Baah 2015; Patnaik et al. 2015).



(a) Typical maximum crack width, 0.1 in. (2.54 mm) (b) Through-thickness crack over an intermediate pier cap
Figure 1 – Typical maximum crack width and through-thickness cracks over an intermediate pier cap



Figure 2 – Extensive wide cracks over the piers of one of the surveyed bridges

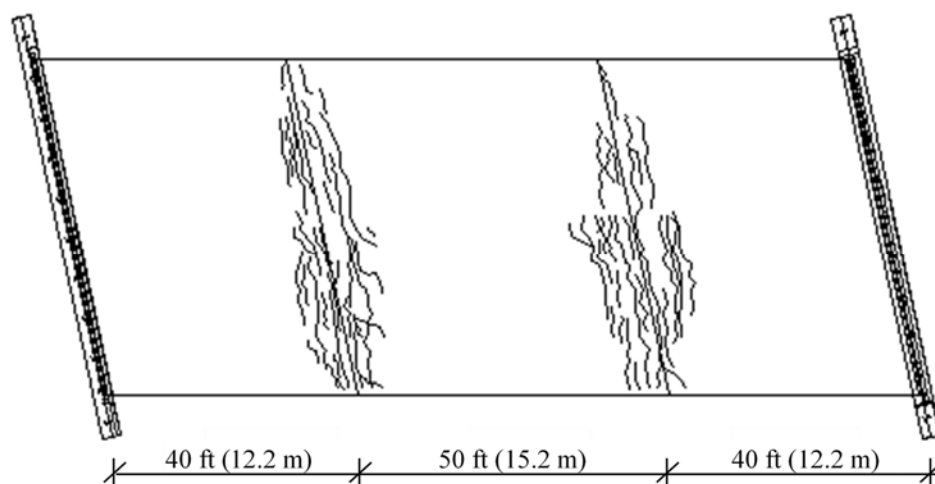


Figure 3 – Typical crack pattern

COMPARISON OF FIELD MEASUREMENTS WITH THEORETICAL PREDICTIONS

Different investigators have used either experimental and/or analytical methods to develop crack width prediction equations. Some of the available studies include Lan and Ding (1992), Chowdhury and Loo (2001), Rao and Dilger (1992), Beeby (1979), Frosch (1999), and Rasidi et al. (2013). Three of the methods were selected from this list to predict crack widths at service load moments over the piers. These three crack width equations are also briefly discussed below. Table 2 shows a comparison of the widest recorded and maximum predicted crack widths using these three equations for service dead load. Predicted crack widths due to just the live load ranged from 0.0059 in. (0.15 mm) to 0.0095 in. (0.24 mm) for these study bridges. The table also shows measured maximum crack widths for each bridge. Figure 4 shows the predicted crack width using Gergely and Lutz equation in comparison to the recorded crack width for a typical bridge.

Gergely and Lutz Equation

At the tension surface of a flexural member, the maximum crack width predicted by Gergely and Lutz (Gergely and Lutz 1968) is given by:

$$w_{\max} = 0.011 \beta f_s \sqrt[3]{d_c A} \times 10^{-3} \quad (4)$$

Where: $w_{\max}$ = most probable maximum crack width; β = ratio of the distance between the neutral axis and tension face to the distance between the neutral axis and centroid of reinforcing steel; f_s = steel stress; A = area of concrete symmetric with the closest reinforcing steel bar; and d_c = thickness of cover.

Chowdhury and Loo Equation

Two crack width equations were proposed by Chowdhury and Loo (Chowdhury and Loo 2001). The average crack width predicted is:

$$w_{cr} = \left(\frac{f_s}{E_s} \right) \left[0.6(c-s) + 0.1 \left(\frac{\phi}{\rho} \right) \right] \quad (5)$$

The maximum crack width is calculated by using the equation:

$$w_{\max} = 1.5 w_{cr} \quad (6)$$

Where: w_{cr} = average crack width; $w_{\max}$ = maximum crack width; ϕ = bar diameter; ρ = reinforcement ratio; c = concrete cover; s = average spacing between the reinforcing bars; E_s = modulus of elasticity of steel; and f_s = steel stress.

Oh and Kang Equation

The equation suggested by Oh and Kang (Oh and Kang 1987) for determining the crack width of a reinforced concrete member is:

$$w_{\max} = a_o (\varepsilon_s - 0.0002) RD \quad (7)$$

in which

$$a_o = 159 \left(\frac{t_b}{h_2} \right)^{4.5} + 2.83 \left(\frac{A_1}{A_{s1}} \right)^{\frac{1}{3}} \quad (8)$$

Where: A_1 = average effective area of concrete around each reinforcing bar; A_{s1} = area of each reinforcing bar; ε_s = axial tensile strain of bars; t_b = bottom cover measured from the center of lowest bar; h_2 = distance from the extreme tension fiber to the neutral axis; h_3 = distance from the centroid of steel to the neutral axis; $R = h_2/h_3$; and D = bar diameter.

Table 2 – Comparison of measured and predicted maximum crack width of structural slab bridges

Bridge No.	Year Built/ Rebuilt	Span Lengths (ft)	Deck Thickness (in.)	Max. Recorded Crack Width for Dead Load (in.)	Predicted Crack Width by Gergely and Lutz Equation for Dead Load (in.)	Predicted Crack Width by Chowdhury and Loo Equation for Dead Load (in.)	Predicted Crack Width by Oh and Kang Equation for Dead Load (in.)
1	2009	40-50-40	24.0	0.100	0.0078	0.0083	0.0051
2	2012	37-46.25-37	22.5	0.030	0.0078	0.0074	0.0043
3	2006	44-55-44	26.0	0.100	0.0081	0.0083	0.0055
4	2001	38-47.5-38	23.0	0.125	0.0074	0.0074	0.0047
5	2006	44-55-44	26.0	0.100	0.0081	0.0083	0.0055
6	2008	28-35-28	18.0	0.040	0.0062	0.0055	0.0028
7	2009	28-35-28	18.0	0.050	0.0062	0.0055	0.0028
8	2006	30-37.5-30	19.0	0.080	0.0062	0.0055	0.0028
9	2006	40-50-40	24.0	0.100	0.0078	0.0083	0.0051
10	2003	40-50-40	24.0	0.080	0.0078	0.0083	0.0051
11	1998	24.5-35-24.5	18.0	0.10	0.0054	0.0056	0.0023
12	1995	24-30-24	16.0	0.10	0.0040	0.0037	0.0018

Note: 1 in. = 25.4 mm; 1 ft = 0.305 m

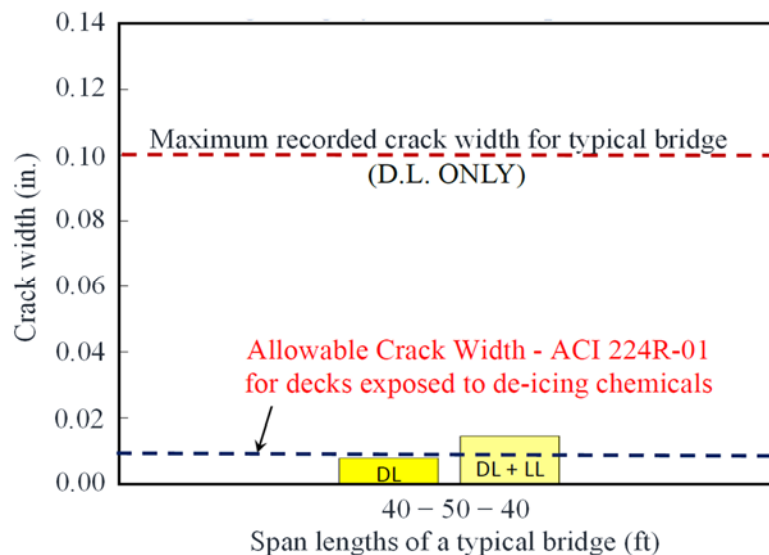


Figure 4 – Predicted crack width using Gergely and Lutz equation in comparison to the measured crack width for a typical bridge (Note: 1 in. = 25.4 mm; 1 ft = 0.305 m; DL = Dead Load; LL = Live Load)

EXPERIMENTAL INVESTIGATION

Material Properties and Test Specimen

Tests were conducted to study the flexural cracking behavior of eight, one-third scale reinforced concrete slabs representing the negative moment region over an intermediate pier of typical three-span continuous structural slab bridge decks. The slabs were instrumented and tested in a flexural configuration to investigate the effect of addition of fiber and epoxy coating on the crack width development in reinforced concrete. All the test slabs had the same cross section and reinforcement configuration with an effective span of 10 ft (3.1 m) and an overall depth of 8 in. (203 mm). The main top reinforcement comprised two No. 6 (19 mm diameter) bars at 6.6 in. (168 mm) on-center and bottom reinforcement of two No. 5 (16 mm diameter) bars at a spacing of 6.4 in. (163 mm). Four slab specimens were reinforced with epoxy-coated steel bars, whereas the remaining four slab specimens were reinforced with black bars. Two of the test slabs had two precut grooves (with a width of 0.125 in. (3.2 mm) and depth of 0.5 in. (12.7 mm)) near the midspan section with a distance of 7¼ in. (184 mm) between them. The grooves were made in the slabs in an attempt to initiate the cracks at those locations. The precut locations were used as the locations where crack width development can be monitored with LVDTs (linear variable displacement transducers). Concrete cylinders; 4 in. diameter and 8 in. long (100 mm diameter and 200 mm long) were also cast and tested at 28 days of age, and on the day of testing to determine the compressive strength of the concrete. The concrete compressive strength testing was conducted for the cylinders according to ASTM C39/C39M. The average compressive strengths determined for the concrete were 4,600 psi (32 MPa) on 28 days and 5,100 psi (35 MPa) on the day of testing. The cast slabs were cured for 7 days with wet burlap and plastic sheets.

The slabs were evaluated over the entire range of service, post-cracked and ultimate limit states. The slabs were subjected to varying levels of loading/unloading until failure. Details of the eight test slabs are shown in Table 3. A typical slab longitudinal section and cross-section are shown in Figure 5 and Figure 6, respectively. An equivalent fiber dosage based on 10 lb/yd³ (5.9 kg/m³) of fiber was used to make the test specimens in this study. One of the important features of fiber reinforced concrete is the ability of the fiber to bridge across cracks. Fibers when added to concrete modify the cracking mechanism from macro cracking to micro cracking. Figure 7 shows polypropylene fiber and basalt MiniBar fiber used in this study, and Table 4 provides information regarding the properties of these fiber.

Table 3 – Details of Test Slabs without Fibers

Specimen	With/Without Fiber	Rebar Type	Precut (⅛" thick; ½" deep)
SLAB U	No fiber	Uncoated	No
SLAB E	No fiber	Epoxy-coated	No
SLAB U-P	No fiber	Uncoated	Yes
SLAB E-P	No fiber	Epoxy-coated	Yes
SLAB U-BF	Basalt Minibar	Uncoated	No
SLAB E-BF	Basalt Minibar	Epoxy-coated	No
SLAB U-PF	Polypropylene	Uncoated	No
SLAB E-PF	Polypropylene	Epoxy-coated	No

Note: 1 in. = 25.4 mm

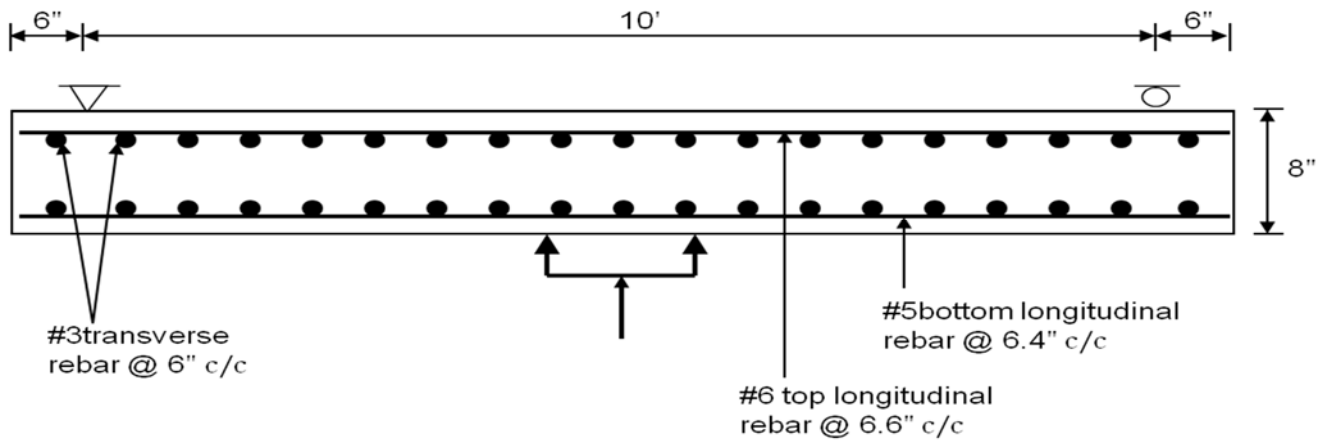


Figure 5 – Typical longitudinal section of the reinforced concrete test slab
(Note: 1 in. = 25.4 mm; 1 ft = 0.305 m; not to scale)

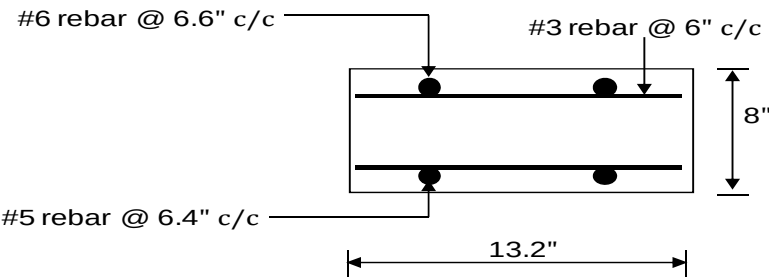


Figure 6 – Cross section details of the reinforced concrete slab
(Note: 1 in. = 25.4 mm; not to scale)



(a) Polypropylene fiber



(b) Basalt Minibar fiber

Figure 7 – Polypropylene and basalt minibar fiber

Table 4 – Properties of polypropylene and basalt minibar fiber

Properties	Polypropylene Fiber	Basalt Minibar Fiber
Length (mm)	38–54	42–45
Diameter (mm)	--	0.66
Density(gm/cc)	0.91	1.8
Rupture Strain	--	0.023
Tensile Strength (ksi)	83–96	157

Note: 1 in. = 25.4 mm; 1 ksi = 6.895 MPa

Instrumentation and Testing Procedure

Several sensors were installed at various stages of the experimental program to record strains, deflections, crack widths and forces. These sensors included strain gages, linear variable displacement transducer (LVDT), and load cells. A computer with a data acquisition (DAQ) interface system was used to monitor and record strain and LVDT data. Contrasting dots were painted onto the surfaces of the test slabs to allow digital image correlation (DIC) to be used for recording concrete strains and deflection at midspan. The displacement results were compared to the LVDT recordings at the midspan section. The flexural crack width test of the slabs was conducted to determine the cracking load, failure load, midspan deflection, steel strains, concrete strains and crack widths at various load levels. Simply supported slabs with an effective span of 10 ft (3 m) were tested under a four-point-loading setup. The two loading points of the steel spreader were 7.25 in. (184 mm) apart. The slabs were loaded from the bottom of the slabs. The main longitudinal reinforcement was placed at the top.

A service limit state test was performed first, where each slab was loaded until the first crack was observed. Several flexural cracks developed at the top of the slabs as the load was increased beyond the cracking load. Consequently, the slabs experienced a further reduction in their flexural stiffness with each successive loading/unloading cycle. The crack width and crack pattern were recorded and plotted at each loading level until failure. A schematic of the test setup for a typical slab specimen is shown in Figure 8. Each slab was subjected to incremental loading/unloading of 4, 6, 8, and 10 kips, (17.8, 26.7, 35.6, and 44.5 kN). The same slab was then loaded to failure after the incremental loading/unloading stages were completed. The ultimate load test for each slab specimen was performed in a force control mode with a uniform loading/unloading rate of 0.25 kip per minute (1.1 kN per minute). The test setup is shown in Figure 9.

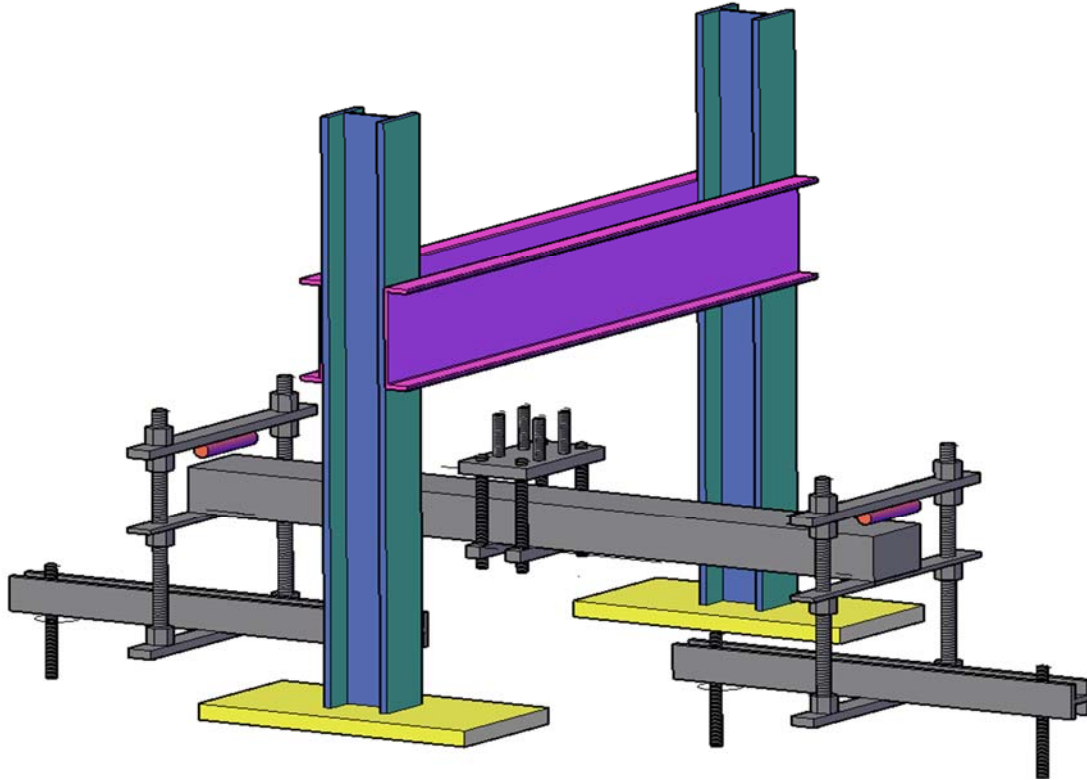


Figure 8 – Schematic diagram of flexural crack width test setup.



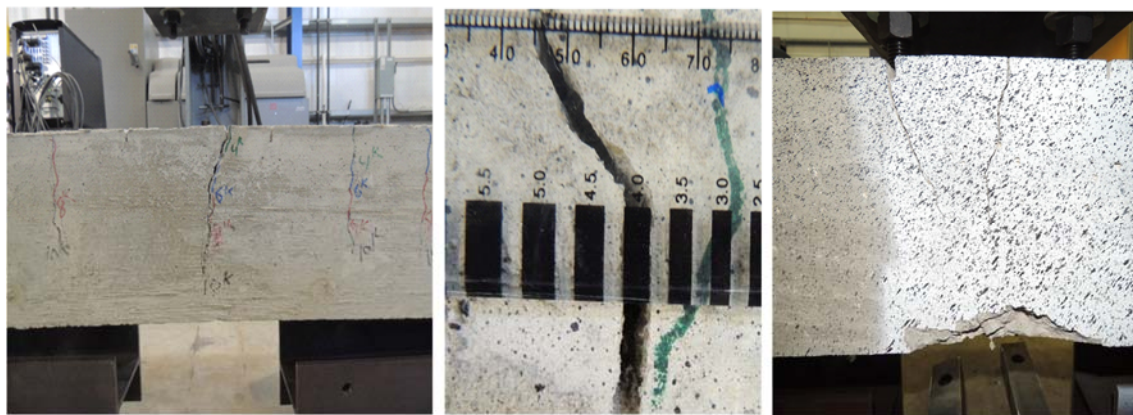
Figure 9 – Test setup for flexural crack width slab specimen

Cracking Behavior of Test Slabs

During flexural crack tests, the load was applied incrementally to the bottom of each test slab until the first flexural crack was observed at the top. The first crack was generally detected at a load level between 1.8 kips and 3.0 kips (8 kN and 13.3 kN) in all slabs without fiber. The propagation of cracks as the test progressed was also monitored, and a crack gage was used to measure the observed cracks. The slabs with fiber experienced higher cracking loads in

comparison to slabs without fiber. The first crack was detected for Slab U, Slab E, Slab U-P, and Slab E-P at loads of 3.0, 2.0, 2.8, and 1.8 kips (13.3, 8.9, 12.5, and 8.0 kN), respectively, and the corresponding measured crack widths were 0.008, 0.01, 0.006, and 0.008 in. (0.203, 0.254, 0.152, and 0.203 mm), respectively. The first crack was detected for Slab U-BF, Slab E-BF, Slab U-PF and Slab E-PF at loads of 3.6, 3.3, 3.4, and 3.2 kips (16.0, 14.7, 15.1, and 14.2 kN). The slabs with epoxy-coated bars withstood lower cracking loads compared to the corresponding test slabs with black bars. All the cracks were observed near the midspan section of the slabs. The cracked stage began with the initiation of the first flexural crack.

The slabs were subjected to incremental loading/unloading of 4, 6, 8, 10 kips, (17.8, 26.7, 35.6, 44.5 kN) and the loading/unloading continued until each slab failed. LVDTs were installed to monitor cracks on the top of the slabs. The crack width and crack pattern were recorded and plotted for each slab. More cracks developed progressively farther from the midspan as the load was increased. For Slab E, the maximum recorded crack widths at 4, 6, 8, 10 kip (17.8, 26.7, 35.6, 44.5 kN) and at failure were 0.016, 0.02, 0.03, 0.04, and 0.125 in. (0.406, 0.508, 0.762, 1.016 mm), respectively. Similarly, for Slab U, the maximum recorded crack widths at 4, 6, 8, 10 kips (17.8, 26.7, 35.6, 44.5 kN) and at failure were 0.01, 0.012, 0.016, 0.02, and 0.08 in. (0.254, 0.305, 0.406, 0.508, and 2.032 mm), respectively. Under various load levels, the two similar slabs (with and without precast grooves) with uncoated bars (Slab U and Slab U-P) experienced similar crack widths and overall flexural crack patterns. At the same load levels, the slabs with fibers exhibited smaller cracks compared to slabs without fibers. The same was observed for the two slabs with epoxy-coated bars (Slab E and Slab E-P). However, the recorded difference in crack widths for Slab U and Slab E were quite significant. For example, at load level of 10 kips, the maximum observed crack widths were 0.02 and 0.04 in. in Slab U and Slab E, respectively. A similar trend was observed in Slab E-P and Slab U-P. At a load level of 10 kips, the maximum observed crack widths were 0.025, and 0.05 in. (0.635 and 1.27 mm) for Slab U-P and Slab E-P, respectively. Slab E-P exhibited the largest crack width, followed by Slab E, Slab U-P, and Slab U. These test results indicate that the crack widths for slabs with epoxy-coated bars were twice as much as those with black bars. The wider crack widths may be attributed to inferior bonds between the epoxy-coated bars and the surrounding concrete. Slab U, Slab E, Slab U-P, and Slab E-P failed at loads of 14.4, 12.4, 13.8, and 12.6 kips (64.1, 55.2, 61.4, 56.0 kN), respectively, and the corresponding crack widths measured at failure using a crack gage were 0.08, 0.125, 0.08, and 0.16 in. (2.032, 3.175, 2.032, and 4.064 mm), respectively. Similarly, Slab U-BF, Slab E-BF, Slab U-PF and Slab E-PF failed at loads of 17.9, 16.2, 16.3, and 15.9 kips (79.6, 72.1, 72.5, 70.7 kN). In addition, the measured crack widths using LVDTs at failure were 0.078, 0.149, 0.060, and 0.199 in. (1.981, 3.785, 1.524, 5.055 mm) for Slab U, Slab E, Slab U-P, and Slab E-P, respectively. Fewer cracks were observed for the uncoated slab specimens when compared to the corresponding epoxy-coated specimens at various loading/unloading stages. The experimental average crack spacing observed for Slab U, Slab E, Slab U-P, and Slab E-P were 5.4, 4.8, 5.5, and 4.6 in. (137.2, 121.9, 139.7, and 116.8 mm), respectively. The recorded maximum crack width and failed test Slab E-P are shown in Figure 10. Flexural test slabs after testing to failure are shown in Figure 11.



*mm scale; Crack width = 4 mm = 0.16 in.

Figure 10 – Failure mode of Slab E-P

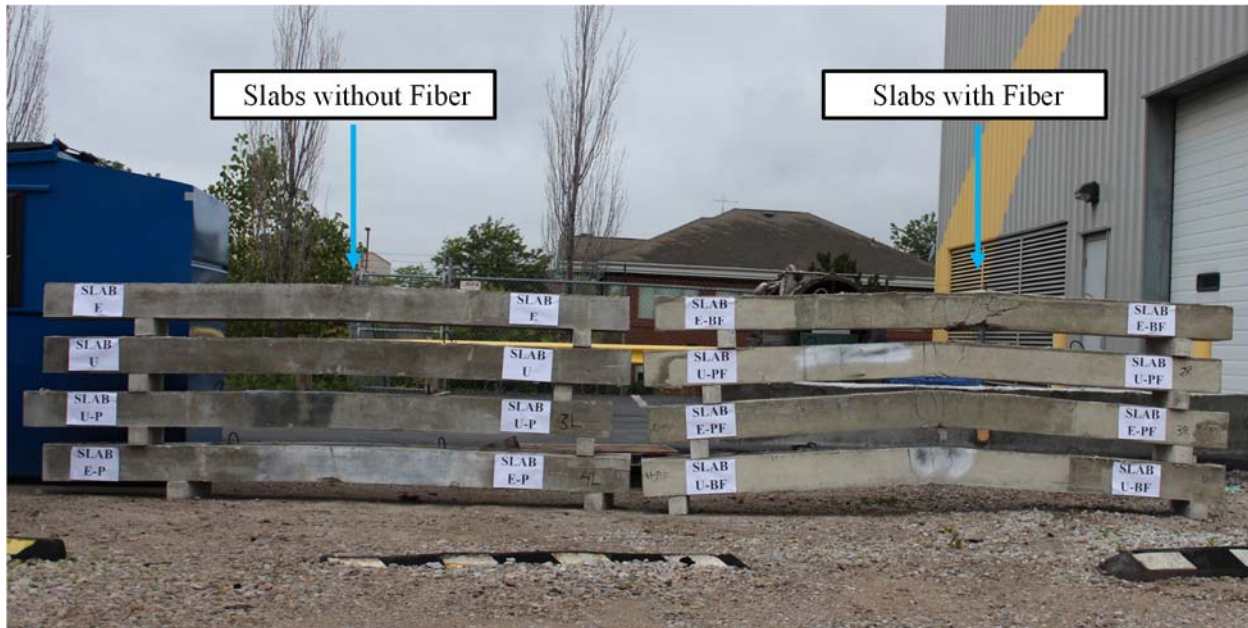


Figure 11 – Flexural test slabs after testing to failure

Load versus Mid-span Deflection Response

LVDTs installed at the midspan of each slab were used to record the deflections. In addition, DIC (Digital Image Correlation) was used to measure the displacement of the slab at the midspan sections. The initiation of the first flexural crack was marked by an obvious change in the slope in the load deflection curves. Several flexural cracks developed in the slabs after the load was increased beyond the cracking load. The residual deflection was recorded for each loading/unloading deflection. All slabs exhibited similar load deflection response until failure. The load deflection response for all the slabs was nearly linear until approximately 10 kips (44.5 kN). At the yielding of steel bars in the slabs, the flexural cracks seem to progressively widen, and the slabs exhibited a rapid increase in the midspan deflection. The maximum observed deflection was 1.84, 1.87, 1.81, and 1.80 in. (46.7, 46.5, 46.0, and 45.7 mm) in Slab U, Slab E, Slab U-P and Slab E-P, respectively. These deflections are reasonably close to each other. Slabs with fiber experienced higher ductility and deflection in comparison to slabs without fiber. For Slab U-BF, Slab E-BF, Slab U-PF and Slab E-PF, the maximum observed deflection was 5.79, 4.79, 4.83, and 4.98 in. (147.1, 121.7, 122.7, and 126.5 mm), respectively. The DIC displacement results were compared to the LVDT recordings at the mid-span section, and the resulting values were found to be reasonably close. Figure 12 shows typical DIC displacement results for Slab E-P. The load deflection response until failure for typical Slab U is presented in Figures 13. Figure 14 shows the load versus midspan deflection for the slabs at ultimate load.

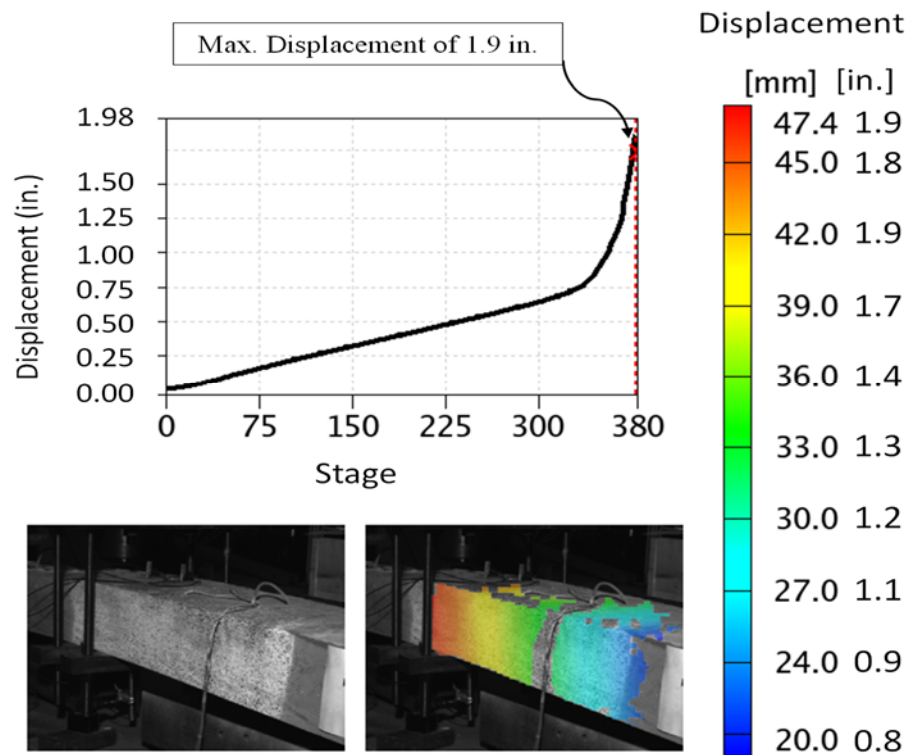


Figure 12 – Digital image correlation deflection results for Slab E-P
(Note: 1 in. = 25.4 mm)

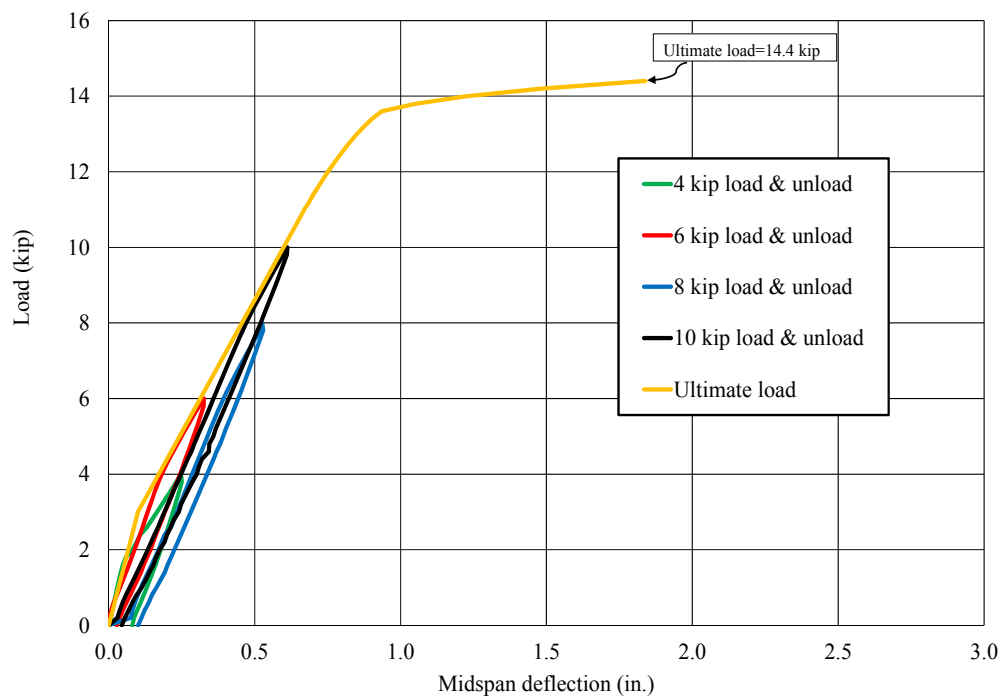


Figure 13 – Load versus midspan deflection for Slab U
(Note: 1 in. = 25.4 mm; 1 kip = 4.45 kN)

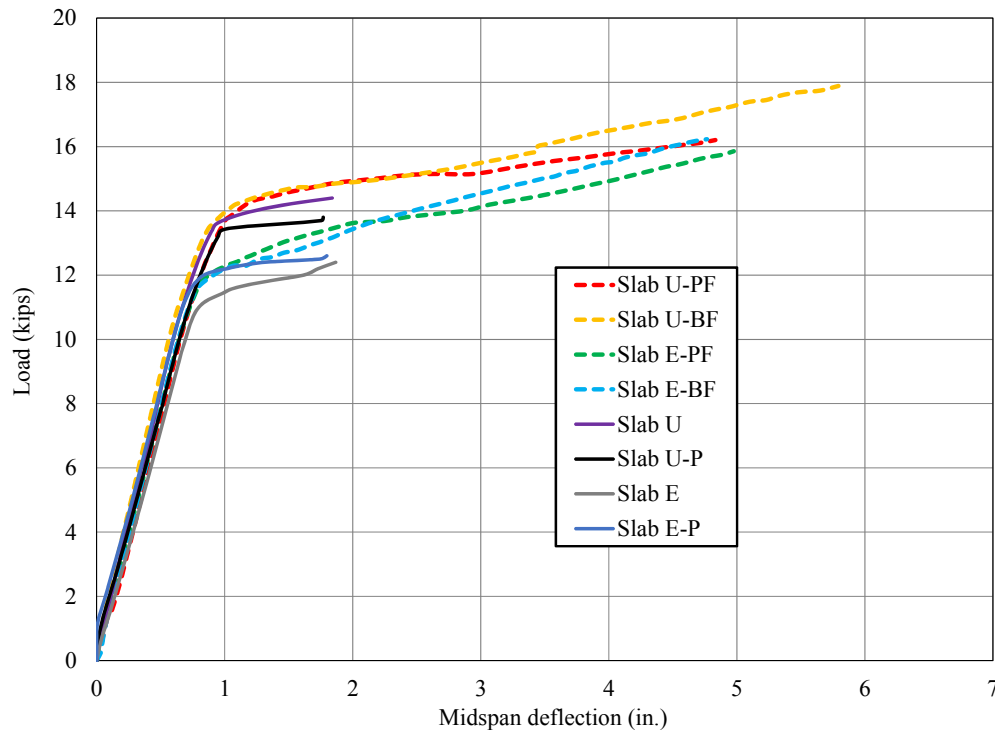


Figure 14 – Load versus midspan deflection for slabs at ultimate load
(Note: 1 in. = 25.4 mm; 1 kip = 4.45 kN)

Comparison between Predicted and Experimental Results for the Test Slabs

In general, the experimental failure load, cracking load, and average crack spacing in the slabs were found to match the values predicted using various equations. The results of the theoretical crack widths determined from Gergely and Lutz (1968), Chowdhury and Loo (2001), and Oh and Kang (1987) were close until approximately a load of 10 kips (44.5 kN), beyond which a sharp increase was observed in measured crack widths that lead to a significant difference between the experimental and theoretical values. This could be attributed to the loading/unloading patterns for which the slabs were subjected to until ultimate loading. The experimental average crack spacings of 5.4, 4.8, 5.5, and 4.6 in. (137.2, 121.9, 139.7, and 116.8 mm) for Slab U, Slab E, Slab U-P, and Slab E-P, respectively are similar to the theoretical average crack spacings of 4.9 in. and 5.1 in. (124.5 mm and 129.5 mm) using the Chowdhury and Loo (2001) and the Frosch (1999) equations, respectively. The determined theoretical cracking load was 2.7 kips compared to experimental cracking loads of 3.0, 2.0, 2.8, and 1.8 kips (13.3, 8.9, 12.5, and 8.0 kN) for Slab U, Slab E, Slab U-P, and Slab E-P, respectively. The predicted failure load was 14 kips compared to the experimental failure load of 14.4, 12.4, 13.8, and 12.6 kips (64.1, 55.2, 61.4, and 56.0 kN) for Slab U, Slab E, Slab U-P, and Slab E-P, respectively. All the test slabs with epoxy-coated bars cracked and failed at loads lower than their corresponding predicted values. The maximum load carrying capacities of the slab specimens with uncoated bars were about 12% higher than the corresponding epoxy-coated specimens. Figure 15 shows load versus crack widths for the test slabs for experimental to theoretical comparison. These test results seem to be the most notable findings in terms of explaining larger crack widths and greater extent of cracking observed on the slab bridge decks with epoxy-coated bars.

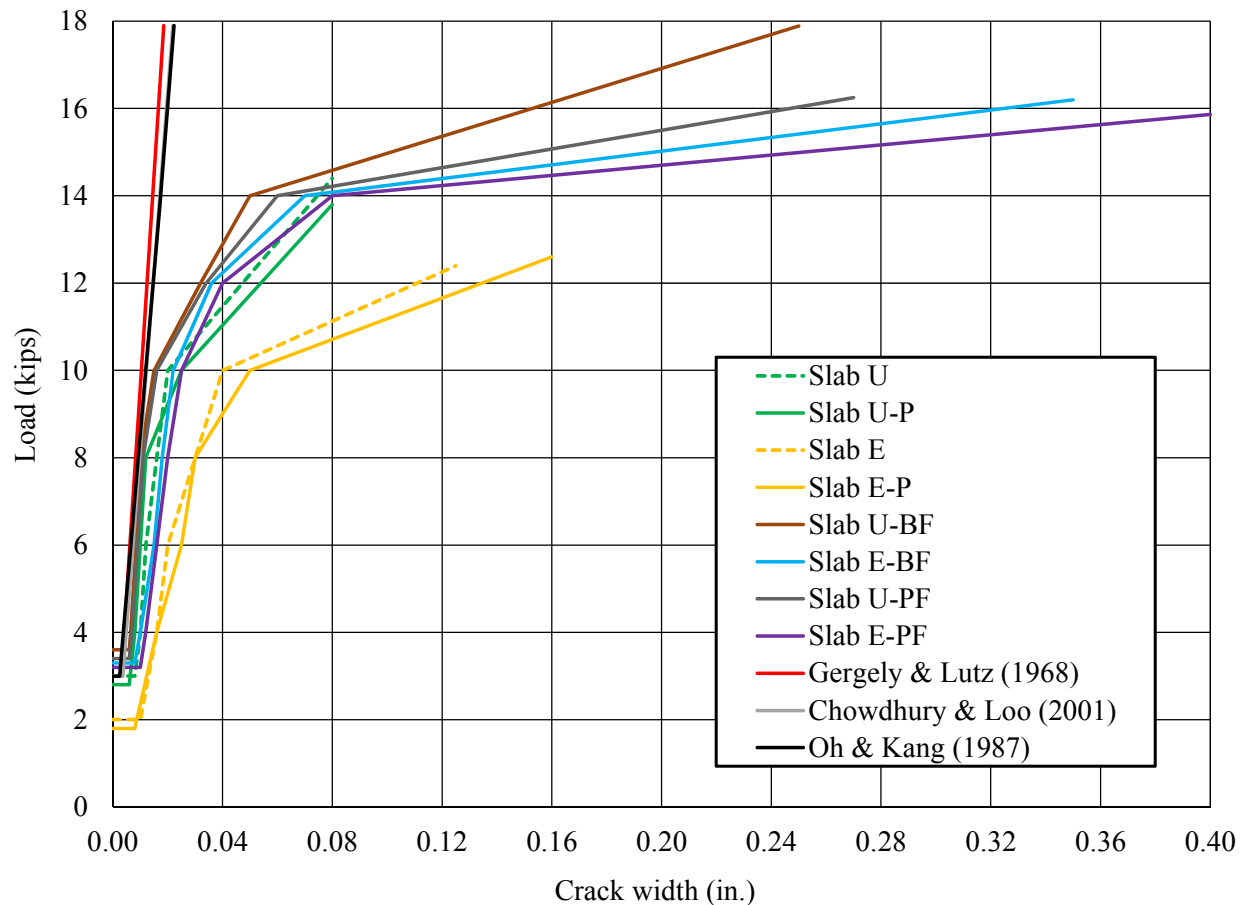


Figure 15 – Load versus crack width: experimental and theoretical comparison
(Note: 1 in. = 25.4 mm; 1 kip = 4.45 kN)

Summary of Flexural Crack Width Test Results

Addition of fiber to the concrete that was used to make the test slab specimens significantly reduced the crack widths. In comparison to slabs without fiber, slabs with basalt MiniBar or polypropylene fiber experienced (i) smaller crack widths over the entire loading range, (ii) higher cracking loads, (iii) higher ultimate failure loads, and (iv) maximum mid-span deflections at failure (better ductility). Consequently, the addition of fiber to the concrete mix used for making reinforced concrete decks is expected to improve cracking behavior of reinforced concrete bridge decks. A summary of results for flexural crack width tests with/without fiber are presented in Table 5. Table 6 and Table 7 present the summary of the effect of addition of fibers on test slabs with uncoated bars and epoxy-coated bars, respectively.

Reduction of Crack Widths in Steel Reinforced Concrete Bridge Decks with Fiber

Table 5 – Summary of results for flexural crack width test

	Slab U	Slab U-P	Slab E	Slab E-P	Slab U-BF	Slab E-BF	Slab U-PF	Slab E-PF
Experimental cracking load (kips)	3.0	2.8	2.0	1.8	3.6	3.3	3.4	3.2
Predicted cracking load (kips)	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
Average crack spacing (in.)	5.4	5.5	4.8	4.6	4.6	5.1	4.5	4.8
Predicted average crack spacing – Chowdhury & Loo, (in.)	4.9	4.9	4.9	4.9	4.9	4.9	4.9	4.9
Experimental ultimate load (kips)	14.4	13.8	12.4	12.6	17.9	16.2	16.3	15.9
Predicted ultimate load (kips)	13.99	13.99	13.99	13.99	13.99	13.99	13.99	13.99
Experimental max. midspan deflection (in.)	1.84	1.81	1.87	1.8	5.79	4.79	4.83	4.98

Note: 1 in. = 25.4 mm; 1 kip = 4.45 kN

Table 6 – Summary of the effect of addition of fibers on test slabs with uncoated bars

	Comparison of Slab U, Slab U-BF and Slab U-PF				
Load Level (kip)	Crack width for Slab U (in.)	Crack width for Slab U-BF (in.)	Crack width for Slab U-PF (in.)	Improvement for Slab U-BF (%)	Improvement for Slab U-PF (%)
6	0.02	0.015	0.016	25	20
7	0.025	0.0165	0.018	34	28
8	0.03	0.018	0.02	40	33
9	0.035	0.02	0.0225	43	36
10	0.04	0.022	0.025	45	38
Ave. crack width improvement (%)				43	28

Note: 1 in. = 25.4 mm; 1 kip = 4.45 kN

Table 7 – Summary of the effect of addition of fibers on test slabs with epoxy-coated bars

	Comparison of Slab E, Slab E-BF and Slab E-PF				
Load Level (kip)	Crack width for Slab E (in.)	Crack width for Slab E-BF (in.)	Crack width for Slab E-PF (in.)	Improvement for Slab E-BF (%)	Improvement for Slab E-PF (%)
6	0.012	0.0085	0.009	29	25
7	0.014	0.0072	0.01	49	29
8	0.016	0.0082	0.011	49	31
9	0.018	0.0093	0.0125	48	31
10	0.02	0.012	0.015	40	25
Ave. crack width improvement (%)				37	31

Note: 1 in. = 25.4 mm; 1 kip = 4.45 kN

CONCLUSIONS

The following conclusions were drawn from this study:

- (a) The crack widths measured on representative bridges are as large as eight to ten times the allowable maximum crack widths recommended in ACI 224R-01 for bridge decks exposed to de-icing chemicals. The crack widths and the extent of cracking of such decks increased with time.
- (b) Laboratory-scale slab flexural tests representing the intermediate support region of typical bridge decks revealed that maximum recorded crack widths of slabs with epoxy-coated bars can be as much as twice the crack widths of similar slabs that are reinforced with uncoated (black) bars.
- (c) In comparison to slabs with black bars, the slabs with epoxy-coated bars experienced (i) larger crack widths, (ii) lower cracking loads, (iii) lower ultimate loads, (iv) smaller average crack spacing and (v) similar mid-span deflections.
- (d) The inferior cracking performance of slab specimens reinforced with epoxy-coated bars compared to those reinforced with black bars is identified as an issue that must be evaluated more closely.
- (e) Adding fiber to concrete significantly reduces crack widths in concrete. In comparison to slabs without fiber, slabs with basalt MiniBar or polypropylene fiber experienced (i) smaller crack widths in the entire loading range, (ii) higher cracking loads, (iii) higher ultimate loads, and (iv) maximum mid-span deflections at failure (better ductility). Consequently, the addition of fiber to the concrete mix used for making reinforced concrete decks is expected to improve cracking behavior of reinforced concrete decks.
- (f) The ACI 318-14 (2014) requirements for crack control were recently switched from the determination of explicit crack widths to prescriptive maximum spacing of flexural reinforcement. However, satisfying these spacing requirements or similar requirements given in AASHTO LRFD were found to be inadequate to control cracking in three-span continuous structural slab bridges. The reinforcement details of all the bridges included in this study satisfy the relevant requirements for crack control given in AASHTO and ACI-318. The current specifications were developed in the 1960s, before the use of epoxy-coated bars was adopted and therefore, are inaccurate by a large margin when applied to bridge decks reinforced with epoxy-coated bars. It is anticipated that the addition of fiber to concrete would alleviate the problem of cracking in bridge decks to an acceptable level.

ACKNOWLEDGMENTS

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A Model to Predict the Crack Width of FRC Members Reinforced with Longitudinal Bars

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Synopsis: A hybrid analytical/numerical approach for the evaluation of the moment-rotation behavior of a cross section of fiber reinforced concrete (FRC) elements flexurally reinforced with longitudinal bars is briefly described. This model is applied to FRC elements failing in bending, and considers the constitutive laws of the constituent materials, where a special focus on the simulation of the post-cracking tensile behavior of FRC was given, as well as the bond behavior between flexural reinforcement and surrounding FRC. The predictive performance of the proposed model is assessed by simulating experimentally tested FRC beams of different geometry, fiber content, and longitudinal reinforcement ratio. Furthermore, the predictive performance of RILEM TC 162 TDF and fib Model Code 2010 design guidelines for the prediction of the crack width in FRC elements failing in bending is also discussed in the present work. The potentiality of the developed model is then explored for the assessment of the influence of toughness classes of FRC and the bond stiffness between flexural reinforcement and surrounding FRC on the moment-crack opening response of FRC flexural members.

Keywords: Crack width, fiber reinforced concrete (FRC), flexural elements, reinforced FRC elements

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INTRUDUCTION

The long-term durability of reinforced concrete (RC) structures is noticeably influenced by cracking response of RC elements. Penetration of adverse agents through early stage cracking of RC elements is responsible for a huge number of damaged RC structures due to corrosion of steel reinforcement (SR). As a consequence of the SR corrosion, the sound cross sectional area decreases, the concrete cover spalls, the concrete cross sectional area diminishes, as well as the concrete strength. These detrimental effects lead to a decrease of the load carrying capacity of RC members, which can affect the structural safety. Therefore, all the strategies that can protect the reinforcement systems of concrete elements against corrosion are of paramount importance in terms of decreasing the maintenance costs of these structures, and extending their durability and life cycle.

In this regard, the use of fiber reinforced concrete (FRC) is an effective approach to improve durability of reinforced FRC (R/FRC) members. The randomly distributed fibers in a FRC offer resistance to the degeneration of micro-cracks in macro-cracks, and fibers bridging the formed cracks restrain the crack opening propagation due to the fiber pullout mechanisms. The relatively high post-cracking tensile strength that FRCs present is the reflex of these mechanisms, which contributes to increase the load-carrying capacity of R/FRC members [1-4]. Available research also reveals that discrete fibers can eliminate steel stirrups in RC beams [4-6], and punching reinforcement [7]. Due to the noticeable contribution of discrete fibers on cracking control of FRC members, it is of paramount relevance the evaluation of the influence of the post-cracking response of FRC on width of cracks formed in R/FRC flexural elements. An analytical/numerical model developed to determine the moment-crack width relationship of R/FRC elements failing in bending is presented in this paper. The predictive performance of existing design guidelines for the evaluation of this relationship is also assessed and compared.

NUMERICAL/ANALYTICAL MODEL

A segment of an R/FRC beam is shown in Fig. 1 under pure bending moment condition. When the most tensioned concrete surface (in Fig. 1 is the bottom surface) attains the concrete strain at crack initiation, $\varepsilon_{ct} = \varepsilon_{cr}$, i.e., cracking bending moment is installed ($M = M_{cr}$), flexural cracks are initiated. For the sake of simplicity it can be assumed that the crack spacing (L_{cs}) is constant along the pure bending zone. Therefore, the length of the pure bending zone is subdivided in prisms of a length L_{cs} , as schematized in Fig. 1. The L_{cs} can be determined based on equilibrium considerations leading to the following fundamental equation governing the bond between a steel bar and the surrounding concrete [8]:

$$\frac{d^2 s(x)}{dx^2} - \left(\frac{L_{ps}}{A_s E_s} + \frac{L_{ps}}{E_c A_{c,ef}} \right) \tau(x) = 0 \quad (1)$$

where A_s , L_{ps} and E_s are the cross section area, the perimeter and the elasticity modulus of the reinforcement, respectively, and E_c is the Young's modulus of FRC that can be obtained from the recommendations of *fib* Model Code 2010 [9]:

$$E_c = 21.5 \left(\frac{f_{cc}}{10} \right)^{1/3} \quad (\text{in GPa}) \quad (2)$$

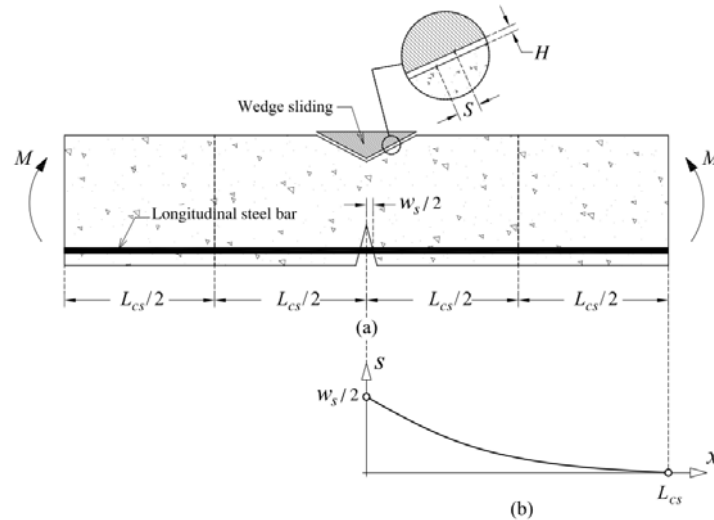


Fig. 1- Idealized representation of flexural deformation of R/FRC beams and crack spacing.

In Eq. (1), $A_{c,ef} = 2.5cb$ is the effective area of concrete in tension [10] where c is the concrete cover and b is the width of the cross section. The $A_{c,ef}$ can also be obtained by the MC2010 recommendations. Assuming a linear bond stress-slip relationship, $\tau(x) = k_{bs} s(x)$, and respecting the boundary conditions of Eq. (3) for the bar-concrete sliding between two consecutive cracks, L_{cs} is obtained from Eq. (4) [8]:

$$\begin{cases} x = 0 : & s = w_s / 2 \\ x = L_{cs} : & s = 0 \\ x = L_{cs} : & ds / dx = 0 \end{cases} \quad (3)$$

$$L_{cs} = \frac{2}{\beta_s} \quad (4)$$

where

$$\beta_s = \sqrt{k_{bs} \left(\frac{L_{ps}}{A_s E_s} + \frac{L_{ps}}{E_c A_{c,ef}} \right)} \quad (5)$$

being k_{bs} the bond stress-slip stiffness. The overall deformational response of the beam can be obtained by using the moment-rotation response of each prism depicted in Fig. 2. A symmetric cross section that can have a width varying along its depth (Fig. 2b), and a height h , is discretized in n layers in order to take into account the appropriate constitutive law for each concrete layer during the loading procedure. The width, the thickness and the depth of the i^{th} layer (with respect to the beam's top surface) is designated by, respectively, b_i , t_i , and d_i . For the concrete layer at the level of the reinforcement ($d_i = d_s$) the total width of this layer (b_i) is decomposed in the part corresponding to the reinforcement b_s ($= A_s / t_i$) and to the concrete b_c ($= b_i - b_s$).

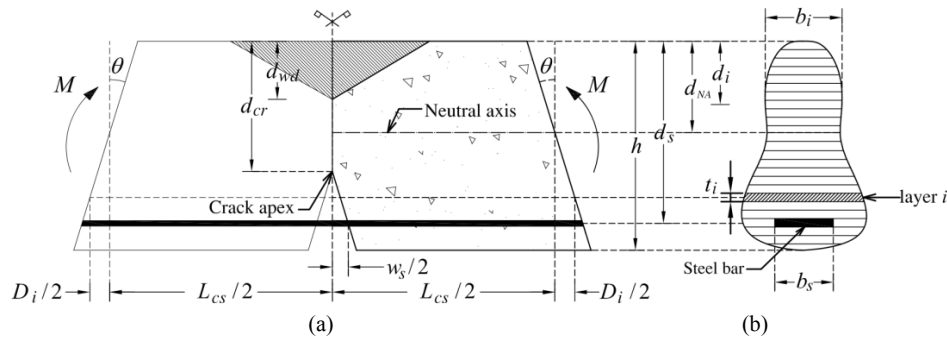


Fig. 2- (a) Cracked R/FRC segment of length L_{cs} submitted to pure bending, (b) layered approach to discretize the cross section.

CONSTITUTIVE LAWS OF THE INTERVENING MATERIALS

Tensile behavior of FRC

Fig. 3 represents the constitutive laws adopted to simulate the tensile behavior of FRC. Up to crack initiation, the FRC is simulated by the following stress-strain equation:

$$\sigma_{ct}(\varepsilon_{ct}) = E_c \varepsilon_{ct} \quad (0 \leq \varepsilon_{ct} \leq \varepsilon_{cr}) \quad (6)$$

where $\varepsilon_{cr} = f_{ct}/E_c$ is the strain at crack initiation, and f_{ct} is the tensile strength of FRC that can be obtained from the recommendations of *fib* Model Code 2010 [9]:

$$f_{ct} = \begin{cases} 0.30 f_{cc}^{2/3} & (\text{for } \leq C50/60) \\ 2.12 \ln(1 + f_{cc}/10) & (\text{for } > C50/60) \end{cases} \quad (7)$$

being f_{cc} the concrete compressive strength in MPa. After crack initiation the FRC is simulated by a stress-crack opening diagram that can be formed by multi-linear segments (Fig. 3b) in order to have the potential of capturing, with high accuracy, the behavior of strain-softening and strain-hardening FRCs [11]:

$$\sigma_{ct}(w) = \left[\alpha_i + (\alpha_{i+1} - \alpha_i) \frac{w - w_i}{w_{i+1} - w_i} \right] f_{ct} \quad (\text{for } w_i < w \leq w_{i+1}) \quad (8)$$

($i = 0$ to 6 , $\alpha_0 = 1$, $w_0 = 0$, $\alpha_6 = 0$, $w_6 = w_u$)

where $\alpha_i = \sigma_i/f_{ct}$ is the normalized stress parameter corresponding to crack width w_i , and w_u is the ultimate crack width. The shape of the σ - w diagram can be determined by performing uniaxial tensile tests with notched FRC specimens [12], or by inverse analysis by fitting with a target accuracy the force-crack mouth opening displacement (CMOD) registered in notched FRC beam bending tests [13]. The *fib* Model Code 2010 [9] also defines a strategy to determine the σ - w constitutive law for FRC by using the flexural tensile strength parameters obtained in three point FRC notched beam bending tests.

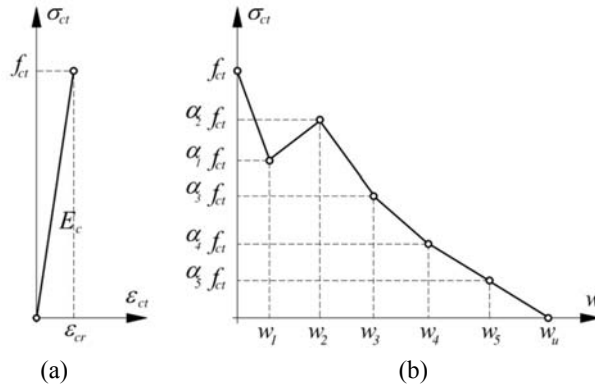


Fig. 3- Tensile behavior: (a) stress-strain before cracking, (b) Post-cracking stress-crack width response.

Compressive behavior of FRC

The compressive response is decomposed in two phases: a pre-peak phase characterized by a stress-strain law ($\sigma_{cc} - \varepsilon_{cc}$), as represented in Fig. 4a; and a post peak phase simulated by a stress-deformation relationship ($\sigma_{cc} - u$), as shown in Fig. 4b. To simulate the pre-peak stress-strain response the equation proposed by Vipulanandan and Paul [14] was adopted:

$$\sigma_{cc}(\varepsilon_{cc}) = f_{cc} \frac{\varepsilon_{cc} / \varepsilon_{cc,p}}{(1-p-q) + q(\varepsilon_{cc} / \varepsilon_{cc,p}) + p(\varepsilon_{cc} / \varepsilon_{cc,p})^{\frac{1-q}{p}}} \quad (9)$$

$$q = 1 - p - \frac{E_{c,sec}}{E_c}, \quad p + q \in]0, 1[, \quad \frac{1-q}{p} > 0 \quad (10)$$

where $\varepsilon_{cc,p}$ is the strain corresponding to the concrete compressive strength (f_{cc}), $E_{c,sec} = f_{cc} / \varepsilon_{cc,p}$ is the secant modulus of elasticity of concrete, and p is a parameter ranging between 0 to 1. For hooked ends steel fibers similar to the ones used in the experimental programs considered in the present work for the assessment of the

predictive performance of the proposed model, Barros and Figueiras [15], based on experimental research and inverse analysis procedure, proposed the following equations for the evaluation of $\varepsilon_{cc,p}$ and p parameters:

$$\varepsilon_{cc,p}^{SFRC} = \varepsilon_{cc,p}^{PC} + 0.0002W_f \quad (11)$$

$$p = 1.0 - 0.919 \exp(-0.394W_f) \quad (12)$$

where W_f is the fiber weight percentage, and $\varepsilon_{cc,p}^{PC}$ is the strain at compressive strength of the plain concrete of the same strength class of the FRC [9]. The superscripts *SFRC* and *PC* in Eq. (11) indicate that the entity is measured in specimens of SFRC and PC of the same strength class, respectively.

The post-peak compressive behavior of FRC is simulated by the following equation:

$$\sigma_{cc} = (\tan \gamma + \cot \gamma) \left(\frac{-30.142 \left(\frac{u}{\cos \gamma} \right) \left(\frac{f_{cc}}{30} \right)^{0.91} + 0.497 f_{cc}}{1 - \left(51.623 \frac{\tan \gamma}{f_{cc}} \right) \left(\frac{u}{\cos \gamma} \right) \left(\frac{f_{cc}}{30} \right)^{0.91}} \right) \quad (13)$$

where u is the axial displacement component of the shear sliding occurred in the contour of the wedge concrete submitted to compression (see Figs. 1 and 5), and γ is the concrete internal frictional angle that ranges between 37° and 57.5° for ordinary concrete depending on the concrete resistance [16]. Since no reliable information is available on the effect of the fiber reinforcement on the γ value, the same interval is also assumed for FRC.

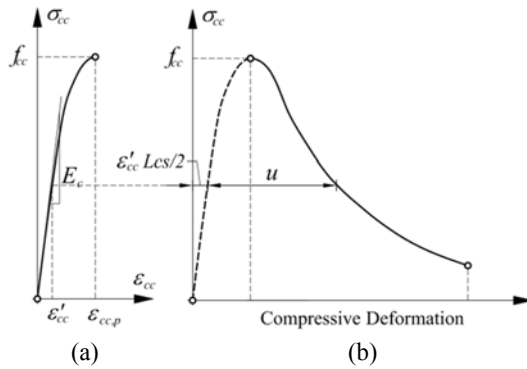


Fig. 4- Compressive behavior: (a) pre-peak stress-strain, (b) post-peak stress-deformation response.

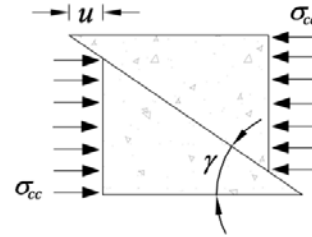


Fig. 5- Wedge sliding mechanism in compression.

Tensile behavior of steel reinforcement

The elastic-perfectly plastic stress-strain response represented in Fig. 6 was considered to simulate the tensile behavior of the steel bars in tension (the same graph was assumed for reinforcement in compression):

$$\sigma_{st}(\varepsilon_{st}) = \begin{cases} E_s \varepsilon_{st} & \varepsilon_{st} \leq \varepsilon_{sy} \\ f_{sy} & \varepsilon_{st} > \varepsilon_{sy} \end{cases} \quad (14)$$

where f_{sy} and ε_{sy} are the yielding stress and strain, respectively, and ε_{su} is the ultimate tensile strain.

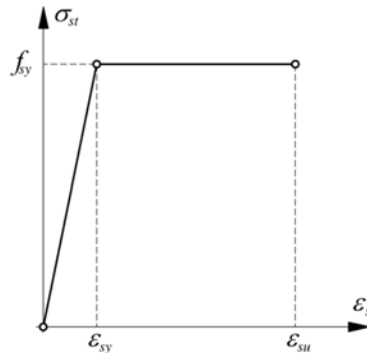


Fig. 6- Elastic-perfectly plastic response to simulate the tensile behavior of steel bars.

ALGORITHM TO PREDICT THE MOMENT–ROTATION RESPONSE OF FRC REINFORCED CROSS SECTION

In the developed incremental-iterative algorithm, the rotation of the beam in a k^{th} generic step of the computation is imposed by considering a constant increment of $\Delta\theta$:

$$\theta_k = k\Delta\theta \quad (15)$$

The increment is the rotation, while the iterative procedure is executed up to assure force equilibrium according to an adopted force tolerance. For this rotation, the axial displacement of the i^{th} layer, D_i^k , is determined by considering its position, d_i , and the distance of the neutral axis (d_{NA}):

$$D_i^k = \theta_k |d_i - d_{NA}| \quad (16)$$

while its corresponding effective strain is obtained from the following equation (the superscript k representing the loading step will be not indicated hereafter in order to simplify the notation):

$$\varepsilon_i^{ef} = \frac{2D_i}{L_{cs}} \quad (17)$$

where L_{cs} is obtained from Eq. (4). If $d_i < d_{NA}$, the compressive force in the i^{th} layer, F_i^c , is obtained from:

$$F_i^c = \begin{cases} \sigma_{cc}(\varepsilon_i^{ef}) b_i t_i & \text{if } \varepsilon_i^{ef} \leq \varepsilon_{cc,p} \end{cases} \quad (18a)$$

$$F_i^c = \begin{cases} \sigma_{cc}(u_i) b_i t_i & \text{if } \varepsilon_i^{ef} > \varepsilon_{cc,p} \end{cases} \quad (18b)$$

where u_i is the horizontal component of sliding in the i^{th} layer, which is determined by solving the following system of equations:

$$\begin{cases} \varepsilon_i^{ef} - \varepsilon'_{cc,i} - (2u_i / L_{cs}) = 0 \end{cases} \quad (19a)$$

$$\begin{cases} \sigma_{cc}(u_i) - \sigma_{cc}(\varepsilon'_{cc,i}) = 0 \end{cases} \quad (19b)$$

where $\varepsilon'_{cc,i}$ is the compressive strain of the layer when subjected to the axial displacement u_i (see Fig. 4b).

If $d_i \geq d_{NA}$, the tensile force in the i^{th} layer, F_i^t , can be obtained from the following equation:

$$F_i^t = \begin{cases} \sigma_{ct}(\varepsilon_i^{ef}) b_i t_i & \text{if } \varepsilon_i^{ef} \leq \varepsilon_{cr} \end{cases} \quad (20a)$$

$$F_i^t = \begin{cases} \sigma_{ct}(w_i) b_i t_i & \text{if } \varepsilon_i^{ef} > \varepsilon_{cr} \end{cases} \quad (20b)$$

where the tensile stress, $\sigma_{ct}(\varepsilon_i^{ef})$, is obtained from the diagram represented in Fig. 3, while w_i is the crack width determined by solving iteratively the following equation:

$$w_i = \left[2D_i - \frac{\sigma_{ct}(w_i)}{E_c} L_{cs} \right] \quad (21)$$

This equation assumes that the crack width of the formed crack, plus the deformation of the concrete along the prism ($\sigma_{ct}(w_i) L_{cs} / E_c$), is equal to the total displacement of the cracked concrete at this level ($2D_i$). To evaluate the force installed in the reinforcement (F_i^s), two conditions should be considered:

i) when the reinforcement is not crossed by the crack ($d_s \leq d_{cr}$, see Fig. 2a): in this case perfect bond conditions are assumed for the reinforcement, thereby the force is determined from the following equation:

$$F_i^s = b_s t_i \sigma_{st}(\varepsilon_i^{ef}) \quad (22)$$

ii) when the reinforcement is crossed by the crack ($d_s > d_{cr}$, see Fig. 2a): in this case the F_i^s is obtained by the following equation, whose deduction can be found elsewhere [8]:

$$F_i^s = \frac{A_s E_s [(w_s / 2) \beta_s E_c + 0.76 \sigma_{ct}(w_s)]}{0.76 E_c} \quad (23)$$

By imposing the equilibrium of the axial forces in the cross section, the depth of the neutral axis, d_{NA} , is determined. For the equilibrium stage corresponding to the k^{th} loading step, the bending moment is evaluated by:

$$M_k = \sum_{i=1}^n F_i^c d_i + F_i^s d_s \quad (24)$$

where n is the number of concrete layers of the cross section. Eqs. (15) and (24) define a point of the moment-rotation relationship, $M_k - \theta_k$. The flowchart of the model's algorithm is described in Fig. 7.

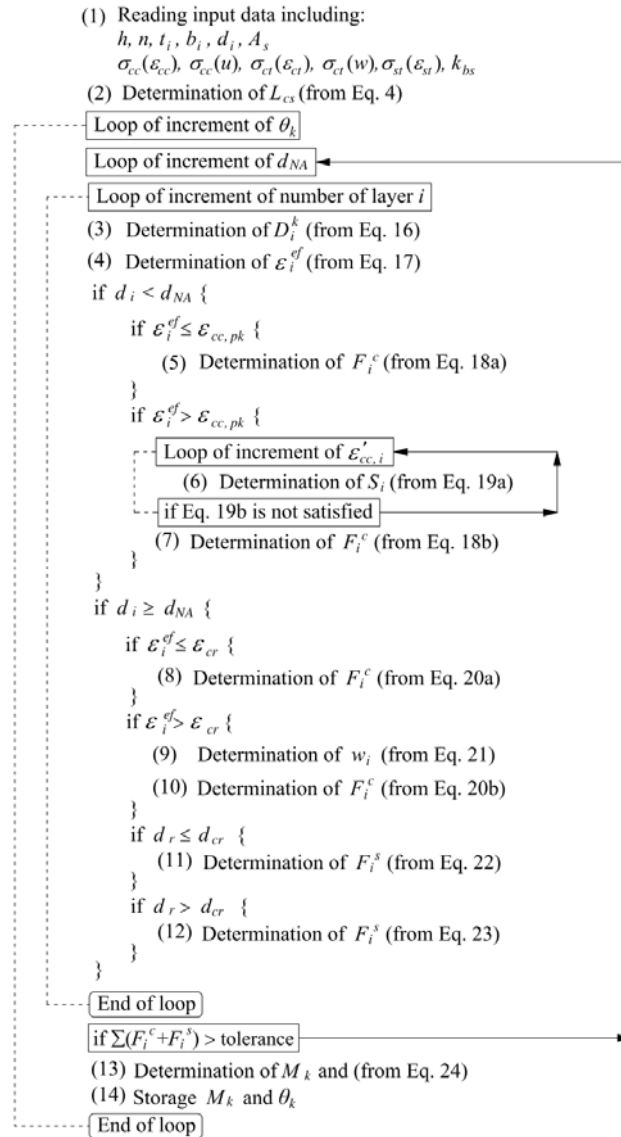


Fig. 7- Flowchart of the algorithm of the model.

MODEL ASSESSMENT

The predictive performance of the proposed model was evaluated by simulating the moment-crack width (at the level of reinforcement) of three R/FRC beams of a cross section of a width of 150 mm (5.90 in) and a height of 110 mm (4.33 in), reinforced flexurally with a steel bar of 8 mm (0.31 in) diameter. The beams were tested in four point bending configuration (Fig. 8). They are designated by B1, B2, and B3, corresponding to the content of 45, 60, and 90 kg/m³ (2.81, 3.74, and 5.62 lb/ft³) of hooked end steel fibers used in the corresponding steel fiber reinforced self-compacting concrete (SFRSCC), respectively. These SFRSCCs are designated by C15f45, C25f60, and C45f90, made of concrete strength class of 15, 25, and 45 MPa (2175.45, 3625.75, and 6526.35 psi) and reinforced by 45, 60, and 90 kg/m³ (2.81, 3.74, and 5.62 lb/ft³) of fibers, respectively. The mix compositions are summarized in Table 1. The average compressive strength of the SFRSCCs at the age of 28 days (f_{cm}) is indicated in Table 2. The f_{cm} was obtained by performing compression tests according to EN 206-1 [17]. The average tensile strength (f_{ctm}) and the average modulus of elasticity of the SFRSCCs (E_{cm}), determined by Eqs. (7) and (2), respectively, are also included. To obtain the average crack width in the central pure bending zone of the beam, the elongation of this zone, measured by LVDT1 (Fig. 8), was divided by the number of cracks formed at the stabilized cracking process (the elastic deformation of concrete between cracks was neglected). The average total number of cracks formed in this zone was 8, 8.33, and 9 in case of B1, B2, and B3 beams, respectively.

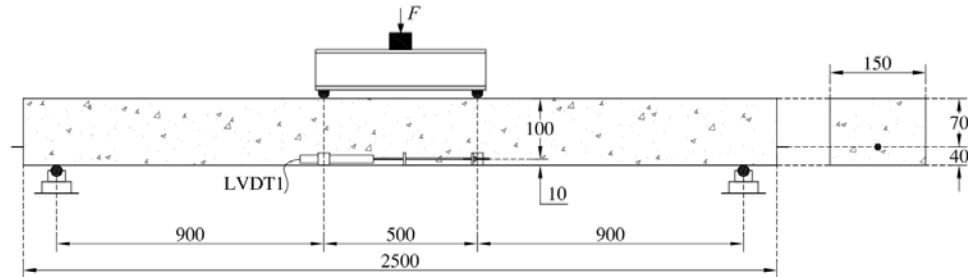


Fig. 8- Four point bending test setup and position of LVDT to measure the horizontal elongation in the pure bending region at the level of the reinforcement (dimensions in mm; 1 mm=0.039 in)

Table 1: Compositions of the developed SFRSCCs (1 kg=2.20 lb).

Designation of SFRSCC	Cement [kg]	Water [kg]	Fly ash [kg]	Crushed granite [kg]	River Sand [kg]	Fine Sand [kg]	Fibers [kg]	SP ¹ [kg]	VMA ² [kg]
C15f45	220.0	105.0	100.0	615.35	693.19	437.44	45	6.08	0.022
C25f60	350.0	160.0	150.0	579.97	697.93	232.97	60	9.50	0.022
C45f90	422.82	143.76	362.42	491.08	671.16	219.95	90	5.92	-

¹SP: Superplasticizer Sika 3005, ²VMA: Viscosity-modifying admixture

Table 2: Mechanical properties of the SFRSCCs used in the experimental program (1MPa=145.03 psi).

Designation	FRC indication	f_{cm} [MPa]	f_{ctm} [MPa]	E_{cm} [GPa]	$f_{R1,m} / f_{R1,k}$ [MPa]	$f_{R3,m} / f_{R3,k}$ [MPa]	Toughness class
B1	C15f45	13.12	1.31	23.31	4.02 / 2.97	3.20 / 2.05	2c
B2	C25f60	23.57	1.74	28.62	7.36 / 4.14	6.44 / 3.54	4e
B3	C45f90	44.42	2.53	35.23	11.59 / 8.73	9.70 / 7.40	8d

To characterize the toughness class of the developed SFRSCCs, three-point notched beam bending tests (TPBT) were performed on beams of 600×150×150 mm (23.62×5.9×5.9 in), following the recommendations of *fib* Model Code 2010 [9]. The obtained average and characteristic values of flexural residual strengths corresponding to the crack mouth opening displacement (CMOD) of 0.5, nominated by $f_{R1,m}$ and $f_{R1,k}$, and the one corresponding to CMOD of 2.5 mm (0.02 and 0.1 in), designated by $f_{R3,m}$ and $f_{R3,k}$, respectively, and the associated toughness class of the SFRSCCs are also included in Table 2. The post-cracking response of the used SFRSCCs was characterized by inverse analysis applied to the force-deflection responses registered in the performed TPBT. The adopted inverse approach is detailed elsewhere [18]. The post-cracking response of the SFRSCCs is depicted in Fig. 9, according to which the parameters that define the stress-crack width constitutive law of Fig. 3 are indicated in Table 3.

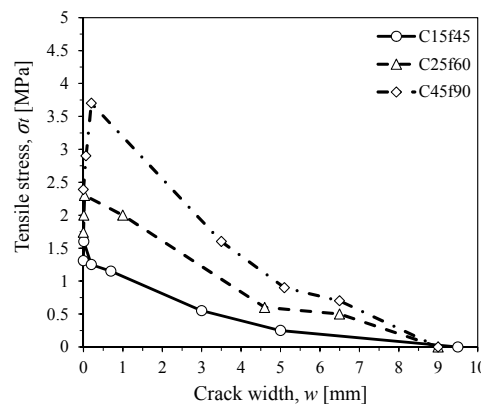


Fig. 9- Tensile stress-crack width relationship of the SFRSCCs (1MPa=145.03 psi and 1 mm=0.039 in).

Table 3: Mechanical properties of SFRSCCs (1MPa=145.03 psi and 1 mm=0.039 in).

		C15f45	C25f60	C45f90
f_{cm}	[MPa]	13.12	23.57	44.42
f_{ctm}	[MPa]	1.31	1.74	2.39
α_1	[-]	1.22	1.15	1.21
α_2	[-]	0.95	1.32	1.55
α_3	[-]	0.88	1.15	0.67
α_4	[-]	0.42	0.34	0.38
α_5	[-]	0.19	0.29	0.29
w_1	[mm]	0.01	0.01	0.065
w_2	[mm]	0.2	0.03	0.2
w_3	[mm]	0.7	1.0	3.5
w_4	[mm]	3.0	4.6	5.1
w_5	[mm]	5.0	6.5	6.5
w_u	[mm]	9.5	9.0	9.0

Direct tensile tests were conducted according to the recommendations of ASTM A370 [19] for obtaining the modulus of elasticity (E_s), the stress and strain at yielding (σ_{sy} and ε_{sy}) and ultimate stress and strain (σ_{su} and ε_{su}) of steel bars, whose results are summarized in Table 4.

Table 4: Mechanical properties of steel bars (1MPa=145.03 psi).

E_s	σ_{sy}	ε_{sy}	σ_{su}	ε_{su}
[GPa]	[MPa]	[%]	[MPa]	[%]
204.8	575	2.8	575	32

In Figs. 10a to 10c the average moment-crack opening relationship predicted by the proposed model is compared to the one obtained experimentally (B1, B2, and B3), and it can be concluded about its high predictive performance. Representative experimental tests conducted by Vandewalle [20] and Tan *et al.* [21] were also simulated. The compressive strength of SFRCs, and the geometry and content of hooked-end steel fibers adopted in these tests are summarized in Table 5. Due to lack of information, the post-cracking response of SFRCs was estimated by following the recommendations of *fib* Model Code 2010 [9], according to which the linear stress-crack width diagram represented in Fig. 11 is adopted for modeling the FRC in tension. In this figure $f_{Fts,m}$ and $f_{Ftu,m}$ are, respectively, the average serviceability and ultimate residual strength of FRC determined from the following equations:

$$f_{Fts,m} = 0.45f_{R1,m} \quad (25)$$

$$f_{Ftu,m} = f_{Fts,m} - \frac{w_u}{CMOD_3} (f_{Fts,m} - 0.5f_{R3,m} + 0.2f_{R1,m}) \geq 0 \quad (26)$$

being $f_{R1,m}$ and $f_{R3,m}$ the average residual flexural tensile strength of FRC at a $CMOD_1 = 0.5$ mm (0.02 in) and $CMOD_3 = 2.5$ mm (0.1 in) evaluated from the force-CMOD relationship determined in three-point notched beam bending tests carried out according to the recommendations of RILEM TC 162-TDF [10]. In Eq. (26) w_u is the ultimate value of crack opening that depends on the level of required ductility, considered equal to 2.5 mm (0.1 in) in the evaluation of the flexural capacity [9]. Since the flexural residual strength parameters are not available in [18] and [19], they were estimated according to the following equations [22]:

$$f_{R1,m} = 7.5(V_f l_f / d_f)^{0.8} \quad (27)$$

$$f_{R3,m} = 6.0(V_f l_f / d_f)^{0.7} \quad (28)$$

whose values are included in Table 5. In these two experiments the SFRC beams were tested according to the four-point bending loading configuration schematized in Fig. 12, where the corresponding data is indicated in Table 6.

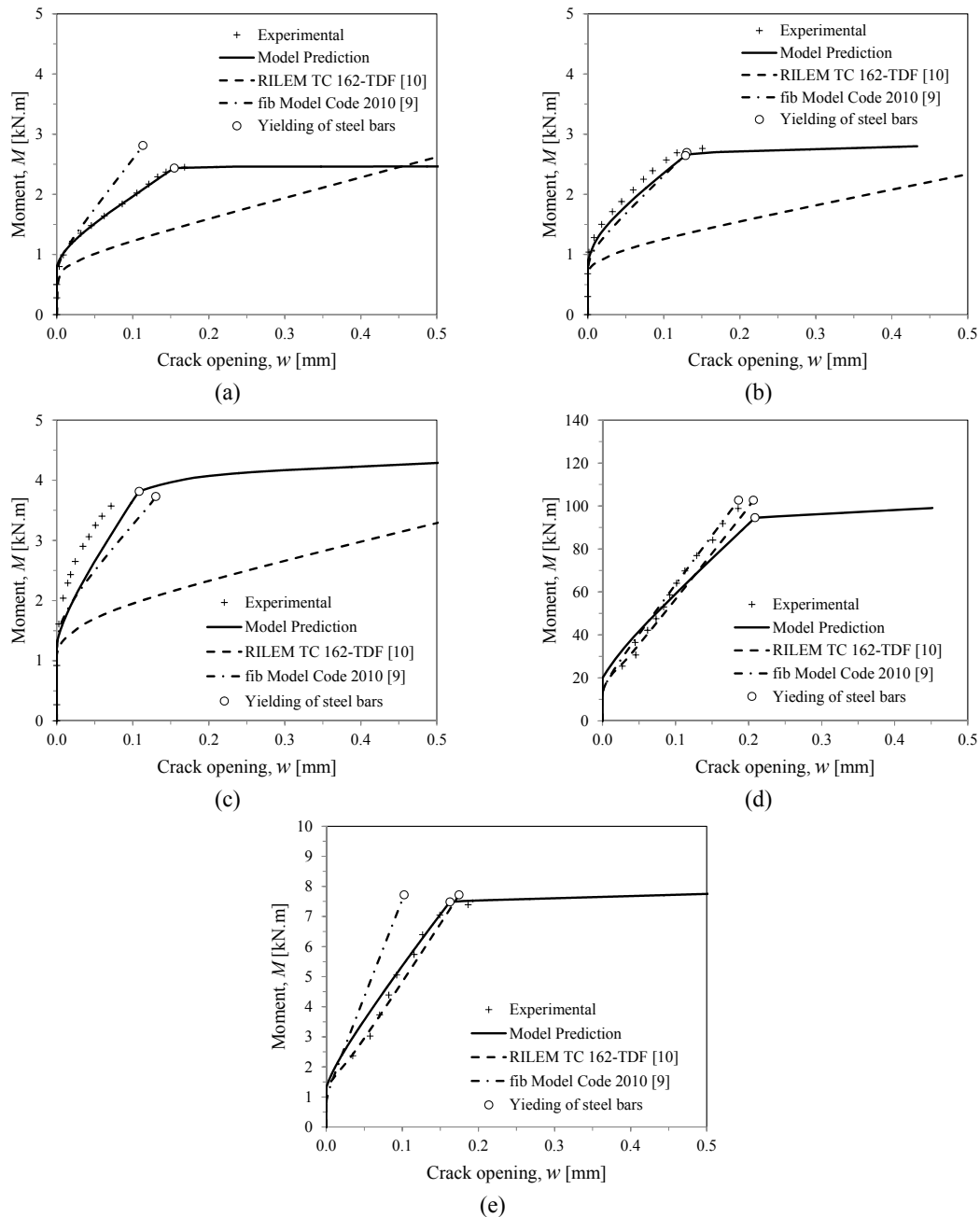


Fig. 10 - Moment-crack width response of (a) B1, (b) B2, (c) B3, (d) B4, and (e) B5 R/SFRSCC beams (1 kN.m=737.56 lb.ft and 1 mm=0.039 in).

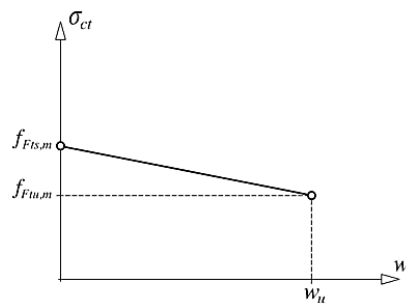


Fig. 11- Tensile stress vs. crack opening diagram recommended by *fib* Model Code 2010 [9].

Material properties of SFRC of the beams are indicated in Table 7. Figs. 10d and 10e show that the model predicts with good accuracy the results obtained in these experiments.

Table 5: Characteristics of concrete and fibers used in the referred test programs (1MPa=145.03 psi and 1 mm=0.039 in).

Reference	Designation	f_{cm} [MPa]	V_f [%]	l_f [mm]	d_f [mm]	$f_{R1,m}$ [MPa]	$f_{R3,m}$ [MPa]
Vandewalle [20]	B4	37.5	0.6	35	0.54	3.18	3.01
Tan <i>et al.</i> [21]	B5	34.5	0.5	30	0.50	3.10	2.63

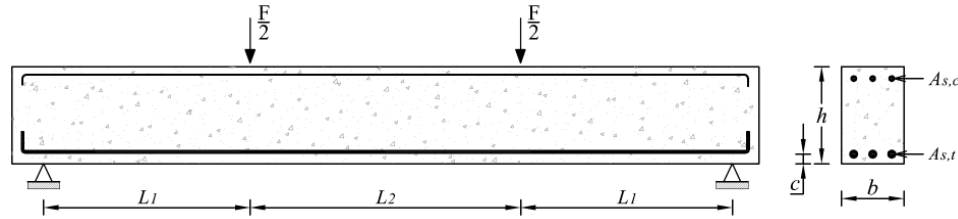


Fig. 12- Reinforced SFRC beam under four point loading configuration (data included in Table 6).

Table 6: Geometric properties of the beams (1 mm=0.039 in).

Designation	b [mm]	h [mm]	c [mm]	L_1 [mm]	L_2 [mm]	$A_{s,t}$ [mm ²]
B4	200	350	35	750	1750	628.3
B5	100	125	25	665	665	157.1

Table 7: Mechanical properties of intervening materials of the simulated beams ($E_s = 200$ GPa,

1MPa=145.03 psi and 1 mm=0.039 in).

Beam series	f_{cc} [MPa]	f_{ct} [MPa]	α_1 -	α_2 -	α_3 -	α_4 -	α_5 -	w_1 [mm]	w_2 [mm]	w_3 [mm]	w_4 [mm]	w_5 [mm]	w_u [mm]	f_{sy} [MPa]	ϵ_{su} [‰]
B4	37.5	2.49	1.11	0.57	0.49	0.40	0.31	0.016	0.40	1.10	1.80	2.49	2.55	500	15
B5	34.5	2.49	1.11	0.56	0.48	0.39	0.31	0.011	0.42	1.09	1.76	2.45	2.55	500	15

PREDICTIVE PERFORMANCE OF AVAILABLE DESIGN GUIDELINES

The moment-crack width response of the beams determined by following recommendation of RILEM TC 162-TDF [10] and *fib* Model Code 2010 [9] are also indicated in Figs. 10. According to RILEM TC 162-TDF [10], up to the yield initiation of the steel reinforcement the design value of the crack width (w_d) for R/FRC members subjected principally to flexure or tension is estimated by the following equation:

$$w_d = k_1 s_{rm} \epsilon_{sm} \quad (29)$$

where k_1 is a coefficient relating the average crack width to the design value ranging between 1.3 and 1.7 depending on the minimum dimension of cross section, and s_{rm} is the average final crack spacing calculated from:

$$s_{rm} = \left(50 + 0.25 k_2 k_3 \frac{\phi_s}{\rho_{s,ef}} \right) \left(\frac{50}{l_f / d_f} \right) \quad (\text{in mm}) \quad (30)$$

where k_2 is a coefficient to take into account the bond properties of the longitudinal bar, equal to 0.8 and 1.6 for high bond and plain bars, respectively, while k_3 is a coefficient equal to 0.5. In Eq. (30) ϕ_s is the bar diameter,

l_f / d_f is the fiber aspect ratio, being l_f and d_f the fiber length and diameter, respectively. Furthermore, $\rho_{s,ef}$ is the effective flexural reinforcement ratio:

$$\rho_{s,ef} = \frac{A_s}{2.5cb} \quad (31)$$

where A_s is the cross sectional area of the longitudinal reinforcement contained within the effective SFRC area in tension ($A_{c,ef} = 2.5cb$). In Eq. (29) ε_{sm} is the average strain in the reinforcement between cracks:

$$\varepsilon_{sm} = \frac{\sigma_{st}}{E_s} \left[1 - k_4 k_5 \left(\frac{\sigma_{sr}}{\sigma_{st}} \right)^2 \right] \quad (32)$$

where k_4 is equal to 1.0 and 0.5 for, respectively, high bond and plain bars, while k_5 is equal to 1.0 and 0.5 for single-short term and sustained-repeated loading, respectively. In Eq. (32) $\sigma_{st} (= E_s \varepsilon_s)$ is the stress in the steel reinforcement up to the yield initiation, where $\varepsilon_s (= \chi(d - x))$ is the strain of the steel reinforcement, being χ and x , respectively, the corresponding curvature and the depth of the neutral axes, and d the effective depth of this reinforcement. Furthermore σ_{sr} is the maximum steel stress at crack section obtained from the following equation:

$$\sigma_{sr} = \frac{(f_{ctm} - f_{Fts,m})}{\rho_{s,ef}} \cdot (1 + n_s \rho_{s,ef}) \quad (33)$$

where $n_s = E_s / E_c$, f_{ctm} is the average value of the tensile strength, and $f_{Fts,m}$ is determined from Eq. (25). In the application of this approach the following values were adopted: $k_1 = 1.3$; $k_2 = 0.8$; $k_3 = 0.5$; $k_4 = k_5 = 1.0$.

More recently the *fib* Model Code 2010 [9] has proposed the following equation to determine the w_d for R/FRC members:

$$w_d = \frac{2}{E_s} \left[k_6 c + \frac{\phi_s (f_{ctm} - f_{Fts,m})}{4 \rho_{s,ef} \tau_{bm}} \right] (\sigma_{st} - k_7 \sigma_{sr} - k_8 \varepsilon_s E_s) \quad (34)$$

where τ_{bm} is the average bond strength between reinforcing bars and surrounding concrete:

$$\tau_{bm} = 1.8 f_{ctm} \quad (35)$$

In Eq. (34), σ_{st} and ε_s are the stress and strain of steel reinforcement at the onset of cracking, and σ_{sr} is determined by Eq. (33). In the evaluation of the σ_{st} and ε_s the SFRC in tension was considered resisting to a constant value of $f_{Fts,m}$, with a linear stress-strain diagram in the compression zone. $\rho_{s,ef}$ is the effective reinforcement ratio equal to the maximum of $A_s / (2.5(h - d))$ and $A_s / ((h - x) / 3)$, being h the total height of cross section. Furthermore, k_6 is an empirical coefficient for simulating the influence of concrete cover that can be assumed equal to 1.0. k_7 is also an empirical coefficient to assess the mean strain between two adjacent cracks, considered equal to 0.6 for instantaneous loading, and k_8 is a coefficient for considering the shrinkage contribution, which can be considered equal to zero for the short-term loading.

The predictive performance of the RILEM TC 162-TDF [10] and MC2010 [9] approaches is assessed by determining the moment vs. crack opening relationship of the B1 to B5 beams, and the results are compared to the corresponding experimental ones in Fig. 10. When FRCs with $f_{Fts,m} (= 0.45 f_{R1,m})$ greater than f_{ctm} (Table 2) are used, such is the case of B1, B2 and B3 beams, the term $f_{ctm} - f_{Fts,m}$ in Eq. (33) and (34) is considered equal to zero. Fig. 10 shows that when the RILEM TC 162-TDF [10] was applied to the B1, B2, and B3 beams a crack width larger than the one obtained with the other approaches was determined (Figs. 10a, 10b, and 10c). Nevertheless, the RILEM TC 162-TDF [10] formulation has predicted with noticeable accuracy the crack width evolution in case of B4 and B5 beams (Figs. 10d and 10e). This can be justified by the reason that this approach was mainly calibrated based on the experimental results of [18] and [19]. Apart B1 and B5 beams, where the crack width was underestimated, in the remaining beams the MC2010 [9] approach has predicted with high accuracy the crack opening evolution. These two beams have the common characteristics of being shallow and with FRC of relatively low strength and toughness classes (Tables 2, 5 and 6), indicating the approach requires improvements for this type of FRC elements.

PARAMETRIC STUDY

By using the proposed model, the influence of the post-cracking performance of FRC on the cracking behavior of R/FRC beams was evaluated. For this purpose, and adopting the toughness classification for FRC proposed by

MC2010 [9], four toughness classes of FRC were adopted, whose post-cracking responses are shown in Fig. 13. According to the MC2010, the toughness class of a FRC is defined by f_{R1k} (representing the post-cracking strength interval) and by a letter a, b, c, d or e (representing the f_{R3k} / f_{R1k} ratio), being f_{R1k} and f_{R3k} the characteristic values of flexural residual strength of FRC corresponding to the CMOD of, respectively, 0.5 and 1.5 mm (0.02 and 0.058 in) determined in TPBTs [9]. Therefore the f_{R1k} and f_{R3k} / f_{R1k} that define the FRC toughness class are the variables investigated in this parametric study. This parametric study is applied to a simply supported R/FRC beam of 2500 mm (98.42 in) span length and rectangular cross section of 300×150 mm (11.81×5.90 in). A longitudinal reinforcement ratio of 0.5% was adopted by using steel bars of 16 mm (0.63 in) diameter in the tension region, with a concrete cover of 50 mm (1.97 in). An FRC of compressive strength of 60 MPa (8701.8 psi) was used, while for the longitudinal steel bars a yield stress (f_{sy}) of 500 MPa (72515 psi), a modulus of elasticity (E_s) of 200 GPa (29006 ksi), and ultimate strain (ϵ_{su}) of 1.15% were considered.

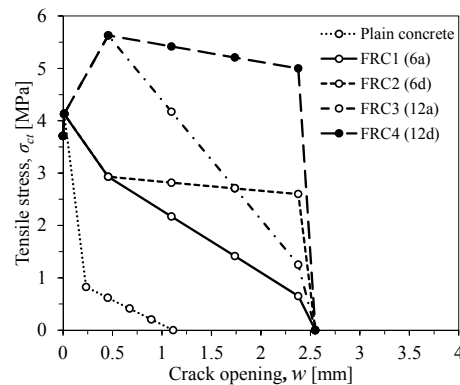


Fig. 13- Post-cracking diagrams of FRC of different toughness classes (1MPa=145.03 psi and 1 mm=0.039 in).

The influence of the toughness classes of FRC on the moment-crack width response of the beam is represented in Fig. 14. Two vertical dashed lines are included, corresponding to the crack width limits of 0.1 and 0.3 mm (0.0039 and 0.0117 in) imposed by *fib* Model Code 2010 [9] for attending the serviceability limit state, SLS, due to cracking in RC structures. According to Fig. 14 by varying the toughness class from 6a to 6d, and also from 12a to 12d, has no effect on the flexural capacity at the SLS (moment at a crack width limit of 0.3mm (0.0117 in), Fig. 14). This means that the increase of the f_{R3} / f_{R1} ratio for a FRC of a certain f_{R1} has negligible effect in terms of flexural capacity at SLS conditions. However, for the present R/FRC beams, an increase of 15.8% was obtained for the moment corresponding to a crack width of 0.3mm (0.0117 in) when a FRC of f_{R1} =12MPa (f_{R1} =1740.36 psi) is used instead of a FRC of f_{R1} =6MPa (f_{R1} =870.18 psi). When compared to a plain concrete of the same strength class, an increase of 37.6% and 59.4% was obtained in terms of flexural capacity at the SLS conditions due to cracking when a FRC of f_{R1} equal to, respectively, 6 and 12 MPa (870.18 and 1740.36 psi) is used.

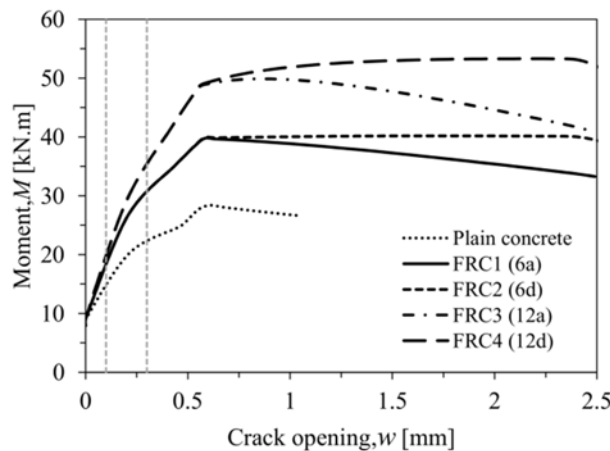


Fig. 14- Influence of the FRC toughness class on the moment-crack opening responses of R/FRC beams (1 kN.m=737.56 lb.ft and 1 mm=0.039 in).

The influence of the k_{bs} bond stiffness on the moment-crack width relationship was evaluated in case of the beam made of FRC3 by adopting a very stiff value ($k_{bs}=80$ MPa/mm ($k_{bs}=297497.6$ psi/in)), which simulates high bond conditions, and two other smaller values simulating higher sliding between reinforcement and surrounding concrete ($k_{bs}=15$ and 7 MPa/mm ($k_{bs}=55780.8$ and 26031.04 psi/in)). As shown in Fig. 15, the flexural capacity for this interval of crack width (0.1 to 0.3mm (0.0039 to 0.0117 in)) increases with k_{bs} . The increase in the bond stiffness can be attained by including micro- and meso-fibers in the FRC composition, capable of minimizing the crack opening and propagation in the concrete surrounding the tensile steel bars [23]. In the present parametric study it is verified that the increase of k_{bs} from 7 MPa/mm (26031.04 psi/in) to 80 MPa/mm (297497.6 psi/in) has provided an increase of 39.7% and 54.2% on the flexural capacity for a crack width of, respectively, 0.1 and 0.3 mm (0.0039 and 0.0117 in).

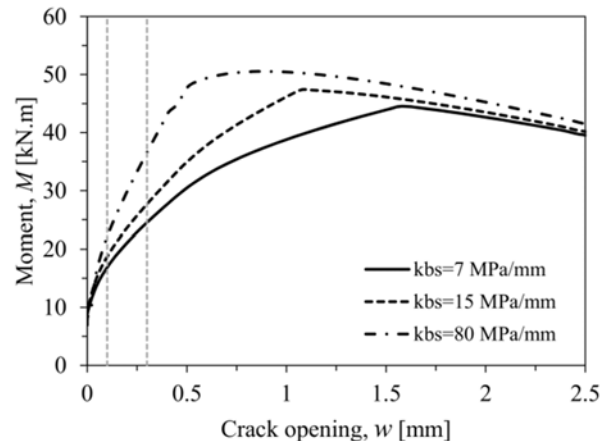


Fig. 15- Effect of the k_{bs} bond stiffness on moment-crack width responses of R/FRC beams (1 kN.m=737.56 lb.ft and 1 mm=0.039 in).

CONCLUSIONS

In the present paper, a model was developed to predict the moment-rotation response of FRC elements reinforced flexurally with conventional bars, which is designated by R/FRC member. The model is capable to consider the constitutive laws of FRC in compression and tension, as well as the bond behavior between flexural reinforcement and surrounding FRC. The predictive performance of the proposed model was assessed simulating experimental tests conducted on five R/FRC beams of different geometry and FRC material properties. The model has predicted with good accuracy the relationship between the bending moment vs crack width. This predictive performance was better than the ones obtained by applying the recommendations of RILEM TC 162-TDF and *fib* Model Code 2010. The RILEM TC 162-TDF demonstrated a tendency to overestimate the crack width, mainly in shallow beams, while the *fib* Model Code 2010 has shown acceptable predictive performance, except in shallow beams of relatively low strength and toughness classes, therefore deeper research is required for this type of R/FRC elements.

By using the developed model, a parametric study to evaluate the influence of toughness class of FRC and the bond stiffness on the flexural capacity of R/FRC beams was carried out. It was verified that in R/FRC elements of a longitudinal reinforcement ratio of 0.5% (a convenient ratio when fibers and longitudinal tensile bars are used), the flexural capacity for crack width limits imposed by actual design guidelines for the accomplishment of the serviceability limit states has increased significantly with the increase of the residual flexural tensile strength parameter for a CMOD=0.5 mm (CMOD=0.078 in), f_{R1} . However, the other parameter that defines the toughness class of a FRC according to the and *fib* Model Code 2010, f_{R3} / f_{R1} , had only a beneficial effect on the flexural capacity for crack width values above the one corresponding to the yield initiation of the longitudinal reinforcement. Finally, it was observed that the bond stiffness has a quite favorable effect on the flexural capacity and on the arrestment of the crack opening up to the yield initiation of the longitudinal reinforcement.

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Effectiveness of Macro Synthetic Fibers to Control Cracking in Composite Metal Decks

Salah Altoubat and Klaus-Alexander Rieder

Abstract: This paper presents results of an ongoing experimental program to study the effectiveness of macro synthetic fibers to control cracking in composite metal slabs. Both short- and long-term performance is being investigated in this experimental program. Two types of experiments for composite slabs on corrugated steel deck are conducted: restrained shrinkage tests and large-scale loaded composite continuous slabs. The restrained shrinkage test provides data on crack width caused by shrinkage, while the large-scale continuous slab was intended to monitor the crack width development across the middle support caused by the load, shrinkage and creep. The crack width measurements of both experiments indicate that the investigated fiber can provide comparable performance in terms of long-term crack control to conventional steel mesh reinforced concrete specified by the standards. Crack width measurements in the restrained shrinkage test over a period of 250 days of drying suggest that macro synthetic fibers at the minimum dosage specified by the ANSI/SDI can provide similar crack control as the minimum steel mesh. Long-term monitoring of load-induced cracking in the slab at the middle support over a period of up to 5 years indicate that the crack width for both reinforcing systems (fibers and steel mesh) increased asymptotically with loading time and stabilized thereafter. The results indicated that creep across the crack occurred for both reinforcing systems suggesting that the creep deformation across the crack is not only related to the type of reinforcing materials and the creep of the fiber/cement paste interface but also by creep of concrete section in compression.

Keywords: Macro Synthetic Fibers, Composite Slabs, Steel Deck, Restrained Shrinkage, Creep, Crack width

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INTRODUCTION

Shrinkage-induced surface cracking, or any form of micro-cracking, has been proven to lead to concrete macro-cracking, compromising serviceability and strength performance if not well controlled (Altoubat, 2000 and 2004). Both fibers and the more conventional welded wire fabric are currently being used contently to control shrinkage cracking in slabs and composite metal decks. However the debate of which is better or more durable continues.

Fibers in general are divided into two categories based on the size and they are referred as microfibers and macrofibers. Synthetic micro fibers are relatively fine with a typical diameter in the range of 0.01 to 0.3 mm (0.0004 to 0.012 inch) and a length in the range of 6 to 25 mm (.25 to 1 inch). Synthetic macro fibers have a typical diameter in the range of 0.3 to 1.0 mm (0.012-0.04 inch) and a length in the range of 19 to 65 mm (0.75 – 2.5 inch). Microfibers are used in relatively small dosage rates (0.03 to 0.1% by volume) to control plastic shrinkage cracking while synthetic macro fibers are used at higher dosage rates to control shrinkage and thermal cracking and to impart toughness and post-cracking strength to the concrete. One study conducted by Trottier et al. (2002) compared the performance of a variety of commercially available low-denier-fibrillated and monofilament-synthetic fibers, at low fiber content (0.03 to 0.1% by volume), to that of a conventional, commonly used 150 x 150 mm 3.4 mm/3.4 mm gage (6 x 6 in. 10/10 gage) welded wire fabric (WWF). The fibers tested comprised of one monofilament and five fibrillated low-denier-synthetic fibers as well as a high-denier monofilament self-fibrillating fiber and a deformed steel fiber. To test against plastic shrinkage, the study was conducted on rectangular plate specimens 900 x 600 x 50 mm, restrained with metallic and wooden fixings systematically placed at the bottom and sides of the molds. The specimens were evaluated for the total number of cracks, the average crack widths and the total area of cracking – corresponding to the sum of the length of each crack multiplied by its average width. As an overview of the results, the study demonstrated that the use of low-denier monofilament and fibrillated-synthetic fibers at low fiber dosage rates (0.3 to 0.9 kg/m³) is considerably more effective in controlling plastic shrinkage cracking than the conventional WWF. In addition, this study also investigated the post-cracking performance of the hardened specimens and the fibers proved they had little to offer at low fiber contents; and the WWF reinforced concrete was superior.

Another study conducted by Shah et al. (1994) also compared the crack controlling performance of WWF and fibers in restrained concrete rings. This study however focused on drying shrinkage induced cracking and the ring test seemed suitable since it simulated the uniaxial state of stress that was typically assumed for temperature and shrinkage stress analysis. The comparison involved concrete specimens reinforced with hooked-end-steel fibers and fibrillated-polypropylene fibers at dosages of 0.25% and 0.5% by volume of concrete as well as two types of welded wire fabric: 150mm x 150mm with a diameter of 4.76mm and a similar 75mm x 150mm. The specimens were subjected to a drying environment after four hours of moist curing and cracking was observed between four hours and forty-two days. The age of first crack, number of cracks, maximum crack width and the average crack width were recorded. The results clearly favored the fibers at the higher dosage with respect to their ability to prolong the time of first crack. However, with respect to reducing the crack width, both WWF and fibers compared favorably and similarly, reducing the maximum crack widths by about 70%. The researchers however did acknowledge that the effectiveness of the WWF was overestimated in this study due to the small thickness of the specimens.

Voigt et al. (2004) confirmed the results above with a similar, albeit more comprehensive study, whereby six different types of steel fibers, three different polypropylene fibers, WWF (W1.4 x W1.4 – 6 x 6 in.), and plain concrete specimens were tested using a similar ring test. The specimens included single fiber type specimens as well as two-fiber blend specimens. All specimens were checked for visible cracks every 24 hours and the width of each crack was measured at forty-two days after casting. The results again clearly showed that the age of first visible crack increased with increasing fiber volume in all cases while the WWF in comparison had no effect on the cracking age. In terms of maximum crack width, the crack width decreased with increasing fiber volume for any of the individual fiber types. The WWF also proved effective, reducing the crack width to about 16% of that of plain concrete – a performance similar to that of the tested steel fibers at fiber volumes of 0.25%. This result was also obtained in the study discussed above by Shah et al. (1994), raising the question of whether steel fibers at a dosage rate of 0.25% could be a viable replacement for WWF.

For steel composite decks in particular, the ANSI-SDI-C2011 permits the use of any of WWF, reinforcing bars, steel or macro synthetic fibers to control shrinkage cracks. The WWF or reinforcing bars limited to a minimum requirement of 0.00075 times the area of concrete above the deck but not less than the area of a 6 in. x 6 in. – W1.4 x W1.4 mesh; the steel fibers to a dosage not less than 14.8 kg/m³ (25 lb/yd³) and the macro synthetic fibers to a dosage not less than 2.4 kg/m³ (4 lb/yd³). To put things into perspective, the minimum requirement for the steel reinforcement by the SDI is nearly 60% lower than the temperature and shrinkage steel limits of 0.18% - 0.20% specified by the ACI 318 building code (2014). These ACI 318 requirements are designed to keep the crack width within an acceptable range. For very tight crack control, ACI 224 (2001) requires an area of steel percentage of about 0.60%.

Although, the current ANSI-SDI-C2011 permits the use of fibers and steel mesh to control shrinkage cracks, the existing literature is still lacking sufficient data on direct comparison of crack width between fibers and steel mesh in composite metal deck applications. In this paper, the results of an on-going experimental program to investigate the effectiveness of one type of macro synthetic fiber and conventional steel mesh to control shrinkage cracking in composite metal deck concrete slabs are presented.

RESEARCH SIGNIFICANCE

The current ANSI/SDI C2011 standard permits the use of macro synthetic fibers and welded wire mesh as a reinforcing material to control drying shrinkage cracking of composite concrete slabs on steel deck. Experimental data on short- and long- term performance of these reinforcing materials to control cracking is still scarce and in particular the long-term performance of synthetic macro fibers relative to the conventional steel mesh. This paper provides comparative data on shrinkage crack width development under restrained conditions of composite slabs. Of particular interest, is the comparative long-term data on a combined shrinkage and load-induced crack width development, which provides information on the creep behavior of these different reinforcing materials.

EXPERIMENTAL PROGRAM

The experimental program is aimed at studying the ability of macro synthetic fibers to control cracking in composite metal slabs. Both short term and long term performance are being investigated in this experimental program. For this purpose two types of experiments were designed and some initial results are presented and discussed in this paper.

Restrained shrinkage test

The current ANSI/SDI standards permits the use of macro synthetic fibers to replace the conventional steel mesh used to control shrinkage cracking in composite metal decks. The effectiveness of fibers to control shrinkage cracking relative to the conventional steel mesh was examined through a restrained shrinkage test and the shrinkage crack width was monitored over time. The crack width was used to assess the effectiveness of fibers relative to the conventional steel mesh in this type of application. The experiment consists of four specimens cast at the same time

over a corrugated steel strip, one specimen (S1) reinforced with conventional steel mesh; the other three specimens (S2, S3, S4) were reinforced with macro synthetic fibers at dosage rates of 2.4, 3.0, and 3.6 kg/m³ (4.0, 5.0, and 6.0 lb/yd³), respectively. Additional concrete prisms were cast for measurement of free shrinkage according to ASTM C157. The prism dimensions were 76.2 x 76.2 x 285.75 mm (3 x 3 x 11.25 in) and they were subject to identical conditions as the restrained test specimens.

The restrained specimen consists of a relatively large-scale slab strip restrained at the bottom and at both ends to provide restrained conditions similar to those occurring in elevated concrete slab on steel corrugated sheet. The slab specimen consists of a "dog-bone" shape concrete specimen with enlarged cross-sectional area cast on corrugated steel deck which provided the bottom restraint, while the ends were fastened with anchors and the rigid steel frame provided the end restraint. The specimens had a cross-section of 200 mm width and 150 mm depth in the middle portion. The width gradually increased at the end zone to 300 mm width. The enlargement of the specimen cross section reduced the stress concentration at the restrained ends. The whole assembly was exposed to normal indoor drying conditions. Figure 1 shows the four specimens in the restrained drying shrinkage test.

Materials—Normal weight concrete with a 28 day cylinder compressive strength of 20 N/mm² (3000 psi) was used as it has been widely used as topping concrete. The cement used was ordinary Portland cement Type I conforming to ASTM C150, crushed washed aggregate with 10 mm (3/8 inch) maximum size, and specific gravity of 2.83, water absorption of 0.7%, 5mm washed sand with specific gravity of 2.67, and water absorption of 1.3%, dune sand with specific gravity of 2.63, and water absorption of 1.2%. The concrete mix was designed with a targeted w/c ratio of 0.64, which is higher than normal but was selected in order to increase the drying shrinkage strain in the experiment and to increase the probability for shrinkage cracking. Table 1 presents the concrete mix proportions which were used for the four slabs in this study.

The macro synthetic fibers used for these tests (Figure 2a) are composed of polypropylene and polyethylene. They have a rectangular cross section with a nominal length of 40 mm (1.57 inch), an average width of 1.4 mm (0.55 inch), an average thickness of 0.105 mm (0.04 inch), and an aspect ratio of 90. The specified tensile strength of the fibers is 620 MPa (90 ksi) and a modulus of elasticity of 9.5 GPa (1378 ksi).

The minimum fiber content of 2.4 kg/m³ (4 lb/yd³) conforming to the requirement of ANSI/SDI –C 2011 standard and ASTM subcommittee C09.42 was tested in this study. The fiber content for the three fiber reinforced slabs {S2, S3, S4} was 2.4 kg/m³ (4 lbs/yd³), 3.0 kg/m³ (5lbs/yd³), 3.6 kg/m³ (6 lbs/yd³) , respectively. This is equivalent to a fiber volume fraction 0.26%, 0.33%, and 0.39% respectively. In the welded wire mesh slab (S1) one layer of 6 inch x 6 inch-W1.4 x W1.4 (152 x 152 mm - MW9 x MW9) welded wire fabric (Figure 2b) placed on the bottom of the upper third of the slab (50 mm (2 inch) from the slab top surface) was provided as shrinkage and temperature reinforcement to meet the minimum allowed by ANSI/SDI –C 2011 standard, which is equal to 59.8 mm²/m (0.028 in²/ft).

Table 1— Concrete Mix Proportions

Material	kg/m ³	lb/yd ³
Cement Type I	320	540
10 mm Aggregate	920	1550
5 mm Sand	670	1130
Dune sand	300	505
Free water (L)	206	347
W/C	0.64	0.64



Figure 1—A photograph of the four slab specimens

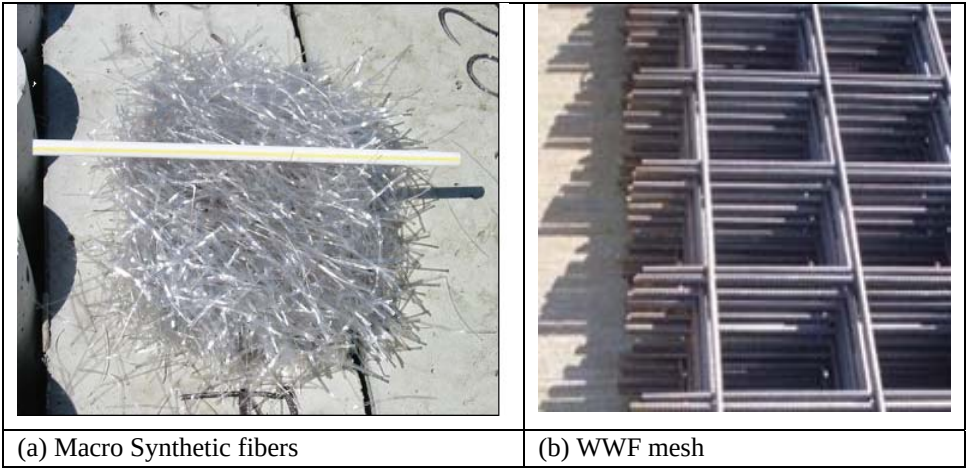


Figure. 2— Secondary reinforcement in composite slabs

Crack observations and measurements—The surface of the specimen slabs were inspected daily after casting for the first two weeks, and every two days after that. Once a crack appeared it was given a code and the crack width was measured using a hand-held graduated microscope with an accuracy of 0.02 mm (0.0007 inch) (see Figure 3). The crack width was measured at several locations along the crack path. The average of these measurements was recorded as the crack width at that age.

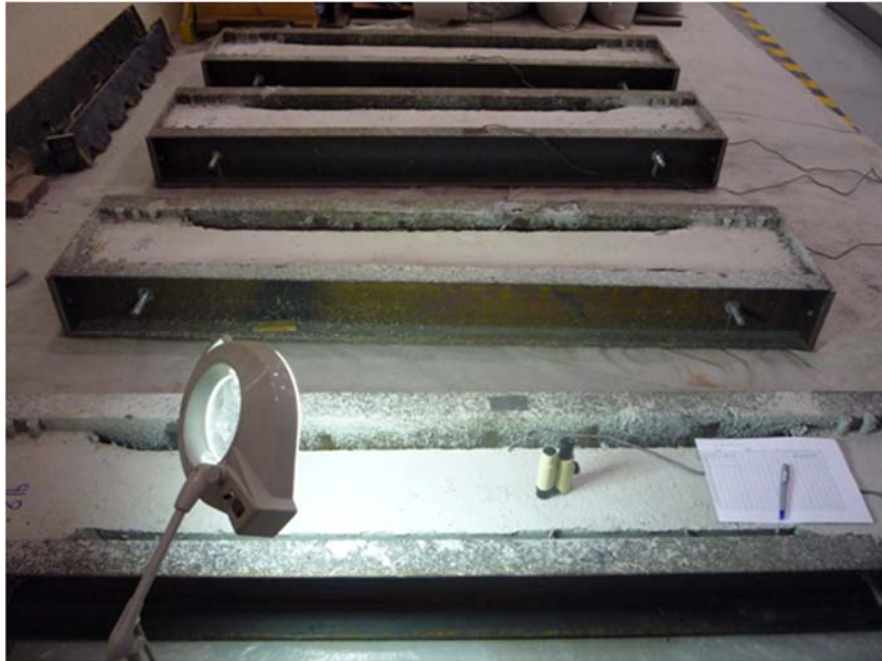


Figure 3— Crack observation

Large-Scale Composite Slab Testing

The purpose of this experiment was to assess the long-term ability of synthetic macro fibers to control cracking in composite floor systems caused by a combined effect of shrinkage and load induced stresses. In this experiment, two-span continuous floor systems were constructed and loaded to its maximum rated capacity. The slab width was 912 mm (3 ft.) which is equivalent to one complete corrugated steel sheet. Two sets of experiments were conducted. Each set consisted of two continuous slabs cast side by side, one with macro synthetic fiber reinforcement and the second one with conventional steel mesh reinforcement. Each slab consisted of two continuous spans with a clear span of 2.3 m (7.5 ft) as shown in Figure 4.

Materials and Slab Parameters—Normal weight concrete with targeted compressive strength of 27.6 MPa (4000 psi) and w/c ratio of 0.5 was used for these experiments. The corrugated steel sheet used was of Gage 20 manufactured by VULCRAFT, with a depth of 2 inch (50.8 mm), and marketed as 2VLI20. The macro synthetic fiber used in the experiment was the same as described in the restrained shrinkage test and shown in Figure 2a. The fiber content was in one case 2.4 kg/m^3 (4 lb/yd³) conforming to the requirement of ANSI/SDI –C 2011 standard and in another case 2.1 kg/m^3 (3.5 lb.yd³). Two sizes of steel mesh were used based on the requirements of the ANSI/SDI standard. Table 2 presents details of the concrete deck, the reinforcement and loading conditions.