Long Term Durability of Structural Materials
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APPROACHES TO ENHANCING CONCRETE BRIDGE DECK DURABILITY

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ABSTRACT

This paper reviews recent results on the mechanisms of durability enhancement in concrete bridge decks via the use of fiber reinforced cementitious composite (FRCC). The deterioration of concrete bridge decks due to shrinkage crack formation and fatigue crack propagation is briefly reviewed. As an approach to enhancing fatigue resistance, fiber addition, and the mechanism of fatigue crack propagation in FRCC is studied. Crack bridging degradation phenomenon is discussed and a fatigue life prediction model based on crack bridging and bridging degradation in FRCC under cyclic loading is presented. Second, a ductile strip concept is investigated for reducing and/or eliminating shrinkage cracks in concrete bridge decks. This approach is introduced and some preliminary experimental results are presented.

KEYWORDS: Durability, Concrete bridge decks, Fatigue, Fiber reinforced cementitious composite, Ductile strip, Shrinkage crack

INTRODUCTION

According to information provided by FHWA, out of 583,349 bridges in the 1996 National Bridge Inventory (NBI), 333,641 have cast-in-place concrete decks and another 38,844 have precast concrete deck panels. The average life of a concrete deck is determined by many factors including initial design, material properties, traffic, environment, salt application, presence and effectiveness of protective systems and maintenance practices among others. Deterioration of the deck is the most common cause requiring repair, rehabilitation or replacement of bridge superstructures. Extensive cracking and large potholes directly affect traffic safety. Therefore, determining the mechanisms of deterioration and developing efficient technologies for resisting and/or eliminating such mechanisms are very important research needs for durability enhancement of concrete bridge decks. Concrete slabs are subjected to considerable fatigue loads. Average daily truck traffic (ADTT) can vary from site-to-site, from less than 500 to over 5,000 trucks per lane, or 200,000 to 2 million trucks per year. Passage of each axle or closely spaced group of axles can be considered as a load cycle. Over the years, a bridge deck slab can be subjected to multi-millions of load cycles. Durability can be considered as the ability to retain an original property, or resistance against long-term deterioration. Often this terminology is used in connection with different kinds of deterioration in materials and structures, under a complex combination of environmental and mechanical loads. For example, concrete durability is considered against chloride ions, carbonation, alkali-aggregate reaction, freeze-thaw cycles, and fatigue, and durability of steel rebars is considered against corrosion and fatigue. Reinforced concrete structures are subjected to these multiple deterioration factors, and structural durability is dependent on each of these factors, as well as their combined effects.

Recent studies show that the service life of reinforced concrete (RC) bridge decks is controlled not only by the corrosion of steel reinforcements, but also by fatigue cracking of concrete slabs [1-4]. The failure mechanism of RC bridge decks is revealed by fatigue loading tests with a moving wheel. The failure progresses through the following five stages [1]. First, cracks are developed on the bottom face of a deck in a transverse direction to traffic. These cracks are mainly due to concrete shrinkage and temperature changes which develops tensile stress (due to restrain) in the longest dimension of the deck. Shrinkage and temperature induced stress together with bending stress due to traffic loading can combine to form these cracks, but, in some occasions, shrinkage stress by itself is high enough to form cracks. Second, longitudinal cracks are developed on the bottom while transverse cracks are developed on the top. On the bottom face of the deck, due to the transverse cracks formed in the previous stage, the deck slab loses load transfer in the longitudinal direction so that flexural cracks are formed in the longitudinal direction. Together with the first set of shrinkage induced transverse cracks, this new set of longitudinal cracks forms a network of cracks. On the top face of the deck, the repeated traffic loading leads to the initiation and growth of transverse cracks starting from the location of girders to the middle of the deck. Since these top transverse cracks are formed in weak sections, they are certain to join the bottom transverse cracks, forming through cracks. Third, water penetrates into the through cracks. The asphaltic topping does not drain easily but tends to retain the water for a long period (e.g. as long as one week after one hour of raining). The water migrates downward through the cracks, creating efflorescence on the bottom surface of the deck. Fourth, the through cracks are gradually worn out under repeated traffic load. The loss of aggregate interlocking leads to the loss of load transfer in the longitudinal direction. As a result, the deck slab does not behave as a plate any longer, but acts as transverse ‘beams’. The presence of water accelerates the wearing out of the cracks. Finally, the transverse ‘beams’ fail in shear fatigue due to the insufficient amount of transverse steel reinforcement of the deck. This leads to the spalling of concrete, and the depression of the deck slab takes place, leading to service termination. Shear punching failure was also observed in the fatigue test conducted on FRP reinforced concrete deck slabs [4].

The above studies indicate that, in RC bridge decks, the importance of fatigue durability is as important as that of corrosion durability. Furthermore, previous investigations [1, 5] show that the integrity of RC bridge decks is actually the key for improving the durability of bridges. Therefore, the prevention of fatigue failure in RC bridge decks is crucial, and the damage sequence described above has to be interrupted before final deck failure. Specifically, the failure mechanism of RC bridge decks shows that the formation of through cracks under repeated traffic loading plays a major role in the sequence of five stages. The formation of through cracks is completed relatively early in the service life, and the rest of the life is spent for erosion and wearing-out of concrete cracks and fatigue of steel rebars [6]. This implies that increased fatigue crack resistance of concrete leads to improvement of the service life of bridge decks, since the progressive crack growth is caused by low fatigue crack resistance of concrete. Therefore, research efforts are needed to investigate and improve the fatigue durability of RC bridge decks and, in turn, the fatigue crack resistance of concrete materials.

Fiber reinforced cementitious composites (FRCCs), typically fiber reinforced concrete (FRC) and fiber reinforced mortar (FRM), are promising materials for fatigue resistant structural elements. With fiber addition, improvements on various mechanical properties, including toughness, impact resistance and fatigue strength have been experimentally demonstrated, e.g. [7-13]. These studies suggest that the use of fiber produces significant improvement not achievables with adjustments of the concrete mix design itself. In the present paper, first recent theoretical and experimental studies on FRC fatigue resistance are reviewed. Second, a newly developed technique for eliminating shrinkage and temperature cracks in concrete bridge decks by inserting ductile strips which are made of FRCC is presented.

FATIGUE RESISTANCE OF FRCs

Experimental Findings

Fatigue of FRCs has been investigated experimentally using the Stress-Life Approaches, and FRCs are shown to have improved fatigue performances. Stress level and fatigue life diagrams (S-N curves) have been obtained for many kinds of FRCs: steel [7-10], polypropylene [11], carbon [12], polyethelene [13], and hybrid (hooked end steel + polypropylene) [10]. The effect of fiber addition to concrete on fatigue strength is positive. An FRC showed 4 times increase in fatigue strength with 3% of straight steel fibers [7]. Hooked end steel FRCs showed 2-3 times increase with less than 1% fiber content [8, 9]. Polypropylene FRC improved fatigue strength by 1.2 times with 0.32% of fiber content [11]. A typical comparison on the fatigue resistance of plain concrete and FRC under bending load is shown in Fig. 1 in terms of maximum flexural stress and fatigue life diagrams [10].

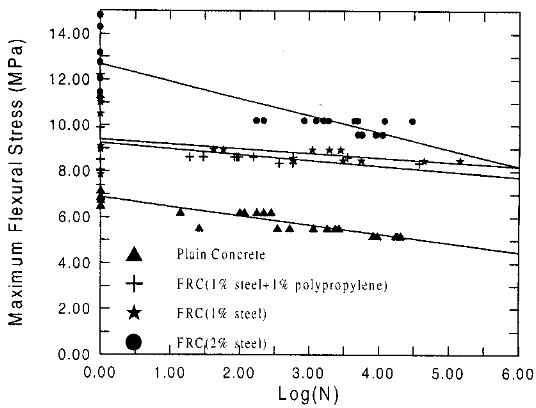


Fig. 1. Experimental maximum flexural stress and fatigue life diagrams of plain concrete and FRCs

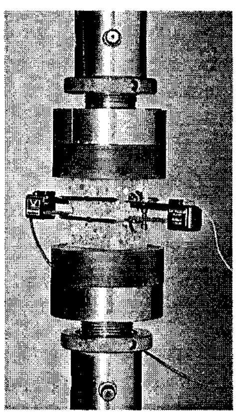
Mechanism of Fatigue Crack Propagation

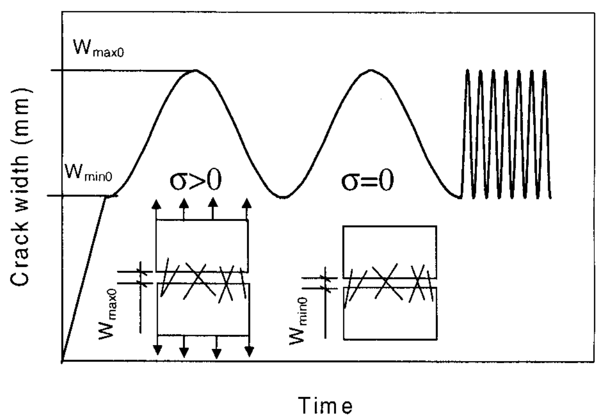
Normally, it can be said that fatigue is a process of progressive, permanent internal structural changes occurring in a material subjected to repetitive stress. The progressive fatigue damage on material constituents is responsible for fatigue life of a material. For FRC, the material phases can be broadly classified as matrix (cement paste and aggregates), fibers, as well as the interfaces of fiber/matrix and aggregate/hydrated cement paste. The fatigue loading causes these physical phases to undergo microscopic changes, such as opening and growth of bond cracks, which exist at the interface between coarse aggregate and hydrated cement paste prior to the application of load, reversed movement of fiber along the interface, fiber surface abrasion and damage of interface in repeated sliding processes. These microscopic changes in turn cause some detrimental changes in macroscopic material properties. Typically, the aggregate bridging force as well as fiber bridging force decreases with number of cycles due to the interfacial damage [14, 15] or fiber breakage due to the surface abrasion [13]. The damages on interfaces of fiber/matrix and aggregate/matrix, which are generally the weakest phase in concrete and FRCs, as well as on soft polymer fibers are likely responsible for fatigue crack initiation and growth in concrete and FRCs. The rate of fatigue crack growth in concrete and FRCs are highly dependent on the crack bridging law governing the zone behind the cement matrix crack and on the law governing the degradation of the crack bridging with the number of load cycles. Fatigue crack growth behavior, in turn, governs the fatigue life of concrete and FRC structures.

Crack Bridging Degradation

As stated above, crack bridging behavior of FRCs under cyclic loading has a significant importance for understanding and predicting fatigue crack propagation. Zhang et al. carried out an experimental study on the crack bridging behavior of FRCs under uniaxial tensile fatigue load [15]. In this study, a series of deformation controlled fatigue tensile tests with constant amplitude between maximum and minimum crack openings were carried out on two side pre-notched specimens. Two types of FRCs, reinforced with commercially available smooth and hooked steel fibers, respectively, are investigated. In this paper, only the results on straight steel fiber reinforced concrete (SSFRC) are presented. The test procedures and results are summarized as follow.

A testing method for measuring the stress-crack width relationship developed by Stang et al. [16] is used in the current tests. The test set-up and the geometry of the test specimen are shown in Fig. 2(a). The test takes place in specially designed grips, one fixed to the load cell and the other fixed to the actuator piston with standard Instron fixtures. The grips consist of a permanent part and an interchangeable steel block, which is fixed to the permanent part through 4 bolts. The specimen is glued to the blocks. The glued surfaces of the interchangeable steel blocks and the specimen are sandblasted before gluing to enhance the bond between steel and specimen. A fast curing polymer which attains 90% of its maximum strength in about 4 minutes was used. The deformation was measured using two standard Instron extensometers (type 2620-602) with 12.5 mm gauge length mounted across each of the two 9 mm deep and 3 mm width notches. The tests were performed in a 250 kN load capacity, 8500 Instron dynamic testing machine equipped for closed-loop testing. The uniaxial fatigue tensile test was conducted under





(a) (b)

Fig. 2. (a) View of the test set-up for fatigue tension and (b) Deformation-time diagram in fatigue test

displacement control with constant amplitude between maximum and minimum crack widths. The minimum crack width value was obtained by a single loading-unloading tensile test and measured at zero loads on the unloading branch. The fatigue test commenced with a ramp to the minimum crack value at a rate of 0.01 mm/second followed by a sine waveform fatigue loading in deformation control. In order to control the accuracy of the maximum crack width value, different load frequencies of 0.25 Hz in the first two cycles and 3.5 Hz for all the rest of cycles were adopted. This fatigue loading procedure is shown in Fig. 2(b). The fatigue tensile test results on the SSFRC material is shown in Fig. 3 and Fig. 4. Fig. 3 demonstrates a typical bridging stress-crack width curve (load-unload loops) during fatigue loading under deformation control. From this figure, it can be found that the secant stiffness ($\Delta\sigma/\Delta W$) of reloading branches reduces gradually with the number of load cycles, therefore the bridging stress at the maximum crack width decreases gradually. The diagrams of bridging stress at maximum crack width versus number of load cycles for a typical maximum and minimum crack widths (W_{max0} and W_{min0}) are shown in Fig. 4, where the results in the range of 1 to 10^5 cycles and 1 to 10^2 cycles are displayed respectively in the figure. Here, the average result and all the individual test results are displayed together.

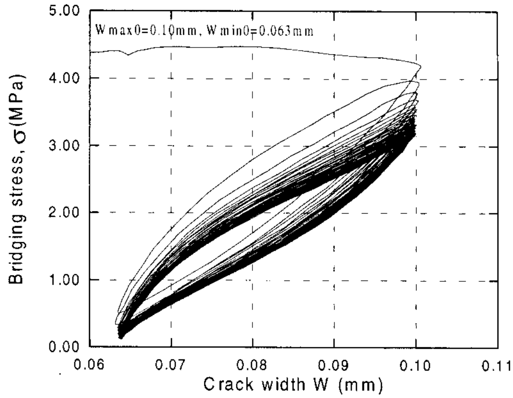


Fig. 3. Typical stress-crack width curve of a fatigue tensile test

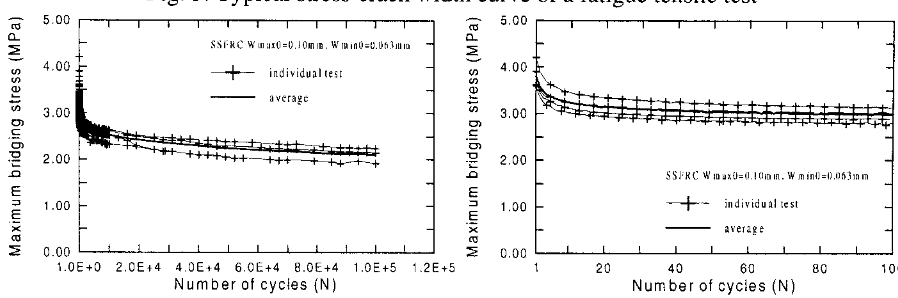


Fig. 4. Typical relation of maximum bridging stress and number of cycles

From these results, it is evident that the maximum bridging stress decreases with number of fatigue cycles for the SSFRC under deformation-controlled fatigue load. The behavior of the stress degradation in the material can be generalized as a fast dropping stage (within first ten to fifteen cycles) with a decelerated rate of stress degradation followed by a stable decreasing stage with an almost constant degradation rate within the experimental period. The bridging stress reduces 7, 15, 23, 17, 16 and 13 percent of their

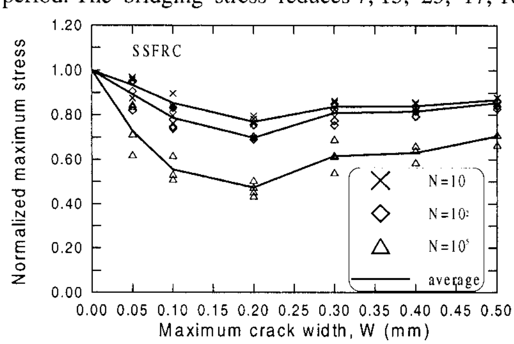


Fig. 5. Relations of normalized maximum bridging stress and maximum crack width of SSFRC, showing the results after 10, 10^2 and 10^5 cycles respectively.

original values when the maximum pre-cracked values are 0.05 mm, 0.10 mm to 0.50 mm respectively after 10 cycles (see [15] for details). This indicates that the rate of bridging decay is affected by the maximum crack width, as shown in Fig. 5 here showing the relations between maximum stress, which is normalized with the stress at first cycle, and maximum crack width after 10, 10^2 and 10^5 cycles respectively. According to Fig. 5, the largest stress degradation in SSFRC occurs at a certain point of the maximum crack width between 0.1 to 0.3 mm. Beyond this maximum crack width, the stress decay diminishes. The largest reduction on crack bridging stress can be more than 50 percent of its original value after 10^5 cycles. From the experimental results described above, we can conclude that the bridging fibers and aggregates in cement-based composites suffer from fatigue damage, exhibiting bridging stress degradation with number of fatigue cycles.

FATIGUE LIFE PREDICTION OF FRC BEAMS UNDER FLEXURAL LOAD

Fatigue strength approach based on experiments requires time-consuming test data collection and processing for a broad range of design cases which, in principle, is not applicable to other design cases. Therefore, a mechanism based fatigue model that is capable of both predicting the fatigue life for a given FRC structure and designing an FRC material for a given fatigue life is needed. Recently, fatigue models based on crack bridging degradation have been developed [17]. The model is able predicting fatigue crack propagation and further predict fatigue life of FRC structures. The model can be summarized as follow.

The fatigue crack growth process in concrete or FRC materials can be broadly divided into two stages: the crack initiation period and the development period. Now considering a simply supported rectangular beam loaded in bending fatigue load with a constant amplitude between maximum and minimum moment M_{max} and M_{min}. When $M_{max}<M_{fc}$, where M_{fc} is the first crack moment, the fatigue life of beam can be given by:

$$N_t = N_{ci} + N_{cg} \tag{1}$$

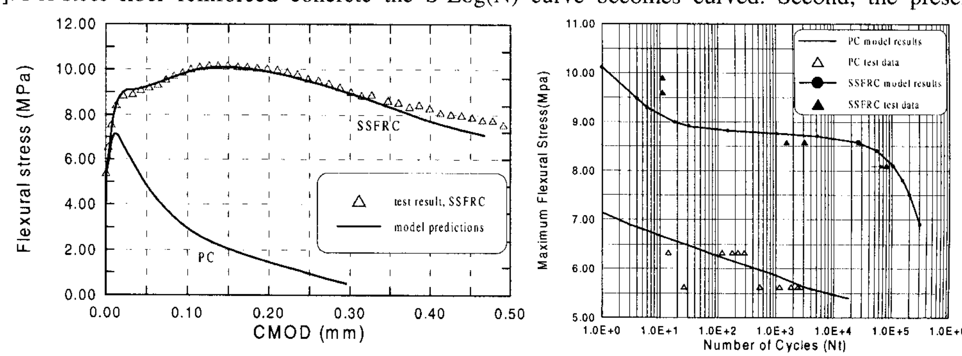
When $M_{max} \geq M_{fc}$, the fatigue life is:

$$N_t = N_{cg} \tag{2}$$

where N_t is the total fatigue life, N_{ci} and N_{cg} are the fatigue life component for the crack initiation and growth respectively. The first term, N_{ci}, is dependent on the microcracking in material which is highly influenced by the microstructure of concrete matrix, such as water/cement ratio, aggregate properties as well as pore structure, size distribution and content. The second term, N_{cg}, is strongly dependent on the bridging performance within the fracture zone under fatigue loading.

The present model focuses on the fatigue life prediction on N_{cg}, i.e. the case of maximum load M_{max} is larger than the first crack load M_{fc}. Based on the above discussions, some basic assumptions for fatigue modelling on N_{cg} can be stated: (1) after a dominant fatigue crack is created, the bridging behavior within the fracture zone governs the rate of fatigue crack advancement; (2) the stress at the crack tip remains constant and is equal to the material tensile strength; (3) material properties outside the fracture zone are unchanged during fatigue loading. It is further assumed that concrete and FRC materials essentially show a linear response in tension up to peak load. After peak one discrete crack is formed. And the discrete crack formation is described by the crack bridging law (or stress-crack width relationship) under both monotonic and cyclic loading. Thus the following material parameters are fundamental in the constitutive relations of concrete and FRC in fatigue tension: the Young's modulus E, the tensile strength σ_t and the cyclic stress-crack width (σ-W,N) relationship of both aggregate bridging and fiber bridging. In compression the behavior of concrete and FRC materials is assumed to be linear elastic and the Young's modulus in compression is the same as in tension. With the above assumptions, a semi-analytical method for predicting fatigue behavior of unreinforced concrete and FRC beams under bending load had been developed. In the model, the cyclic bridging law (or cyclic stress-crack width relationship) was incorporated in integration form which can easily be replaced by other bridging models for different kinds of FRC materials with different fiber types, volume concentration and matrix properties. The complete theoretical curves, in terms of fatigue crack length or crack mouth opening displacement (CMOD) with number of cycles diagrams, as well as the classical S-N curves are obtained and compared with experimental results. The details of the model derivation can be found in the paper by Zhang et al [17].

In the numerical simulation, a specific fatigue loading procedure with M_{min} equal to zero corresponding to that the condition of fatigue tension and bending tests is assumed. The geometry of the specimen in the numerical model is the same as that used in the fatigue bending tests. A fit based cyclic bridging laws for different types of FRC (including concrete) based on the experimental results introduced in the previous section are used in the simulation. The detailed expressions of the monotonic and cyclic crack bridging models as well as related material parameters used in the model can be found elsewhere [17]. In order to compare the results between monotonic loading and fatigue loading, the monotonic bending behavior is simulated first in terms of the load-CMOD relation. Fig. 6(a) shows the predicted monotonic flexural stress-CMOD curves of plain concrete (PC) and SSFRC respectively, together with experimental results for SSFRC. On inspecting the numerical results for the load-CMOD diagrams of the two types of concrete beams under three point bending, several features can be distinguished: (1) load level I: The flexure stress increases linearly with deformation up to tensile strength of the materials, 5.2, 5.4 MPa for PC and SSFRC respectively. In this stage, material behavior obeys elastic constitutive relations and no fictitious crack is formed, therefore CMOD is equal to zero. (2) load level II: The flexural stress increases up to 7.1 and 9.1 MPa for PC and SSFRC. In this period, the deformation increases a little more than proportionally with respect to the stress. A fictitious crack develops in the middle of beam and grows with the load increasing; (3) load level III: The flexural stress increases up to the maximum values, the flexural modulus of beam, about 10 MPa for SSFRC. At this stage, the deformation increases much more than proportionally with respect to the stress. Fatigue behavior is commonly represented by stress-fatigue life curves, normally referred to as S-N curves. In the case of fatigue in bending, S refers to the maximum flexural stress according to classical elastic theory. Fig. 6(b) shows the predicted S-N curves for these two types of concrete, where the fatigue life is presented in the form of logarithm. Some test results are shown together with the theoretical results. It can be seen that model predictions agree well with the test results. First, the S-Log(N) curve of plain concrete is almost linear which agrees with a number of experiments [8-10]. For steel fiber reinforced concrete the S-Log(N) curve becomes curved. Second, the present



(a) (b)

Fig. 6. (a) Monotonic flexure stress versus CMOD curves for plain concrete and SSFRC beams and (b) relation of maximum flexure stress with fatigue life, shown together with experimental data

model predicts that the steel fibers can significantly improve the bending fatigue performance of concrete structures, which has been demonstrated by many researchers [7-10]. For steel fiber concrete beams, with maximum flexural stress between 9.00 to 10.00 MPa (Level III), the fatigue life is very short, within 1 to 30 cycles. The reason for this short fatigue life is a combination of a large initial crack length and significant bridging degradation due to large crack openings. With the maximum flexural stress between 5.4 to 9.00 MPa (Level II), the fatigue life increases notably with decreasing maximum flexural stress. The longer fatigue life is a product of both the shorter initial crack length and the smaller crack openings. When the maximum flexural stress is lower than 5.4 MPa (Level I), no dominant macro fatigue crack occurs after first cycle. However, fatigue crack initiation will not be treated in the present study.

INTRODUCING DUCTILE STRIP FOR DURABILITY ENHANCEMENT OF CONCRETE BRIDGE DECKS

General Introduction

As pointed out in the Introduction section, fatigue cracking in concrete bridge decks appears to be preceded by the formation of cracking due to concrete shrinkage in the transverse direction. Thus deterioration due to fatigue can be curtailed if shrinkage cracks in concrete is minimized or even eliminated. In the present study, attempts at localizing shrinkage induced deformation into designated strips, where an engineered fiber reinforced cementitious composite (ECC) material with strain-hardening and high strain capacity (up to 5%) is used, were carried out. As a result, while microcrack damage exists in the ECC strip, the concrete remains intact. This concept has recently been demonstrated by simulating the shrinkage in concrete under restrain condition as tensile load acting on a specially designed specimen. Experimental results show that it is possible to achieve the targeted deformation mode with certain design on the ECC/concrete interfaces. Due to the special material properties of ECC, the strain energy produced by shrinkage (under restrain condition) of hardened cement and temperature changes can be released by the high strain ability of ECC material so that cracking in plain concrete can be avoided. Thus the fatigue durability of concrete slabs can be improved, resulting in a longer service life. The proposed design concept may be implemented by placing ECC as periodic special joints or "ductile strip" between stretches of concrete slabs. By replacing standard joints with ductile strips, common deterioration problems associated with joints may be also eliminated. The current concept of introducing ductile strip in concrete bridge decks will be summarized as follows. The details on this work can be found in the paper by Zhang, et al [18].

Design of Concrete Slab with Ductile Strips

Assume that a concrete bar is composed of two kinds of materials, ductile ECC material and plain concrete with length l_I and l_{II} respectively. The bar has the same cross section along the length. Further assume that the two materials are perfectly joined together without failure at the interface under tensile load. The general dimension of the bar and the corresponding stress-strain relationship under tensile load of individual materials are shown in Fig. 7, where the concrete tensile strength is higher than that of the ECC material. Under uniaxial tension, the overall strain capacity of the bar, ε_c, is reached when the load reaches the tensile strength of the ECC material. Hence ε_c is

$$\varepsilon_c = \varepsilon_I\left(\frac{l_I}{l}\right) + \varepsilon_{II}\left(\frac{l_{II}}{l}\right) \tag{3}$$

where ε_I is the strain capacity of ductile material and ε_{II} is the strain value of plain concrete corresponding to the tensile strength of ECC material. l is the total length of the bar. Therefore, the composite strain capacity, ε_c, (strain at peak stress in curve labeled I-II in Fig. 7(b)), is a function of ε_I, ε_{II} and l_I or l_{II}. For given material properties, ε_c is influenced only by the individual element length l_I or l_{II}. Fig. 8 demonstrates the overall strain capacity, ε_c as a function of l_I with different given strain capacity of ECC material, ε_I. It clearly shows that for a given ductile strip width, the higher the strain capacity of ductile ECC material, the higher the overall strain capacity of the composite bar. In addition, a high composite strain capacity can also be obtained through adjusting the length of the ductile strip. With a reasonable combination of plain concrete and ductile ECC strips, it is possible to achieve a prescribed strain capacity requirement, which may be twenty or thirty times the strain

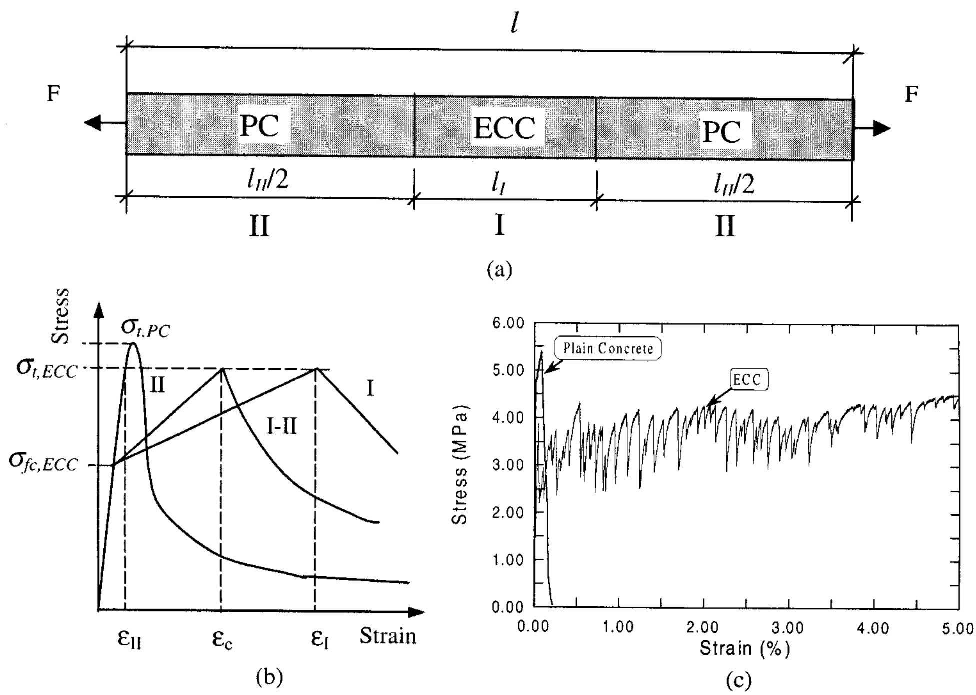


Fig. 7. (a) A PC/ECC/PC tensile bar, (b) schematic stress-strain behavior of ECC and concrete under uniaxial tensile load and (c) experimental obtained stress-strain curves of plain concrete and ECC

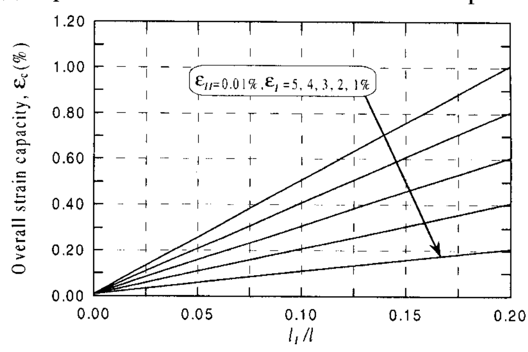
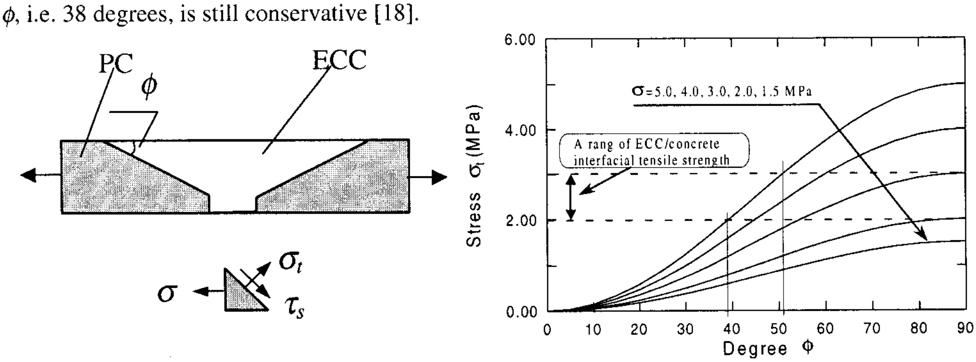


Fig. 8. Overall strain capacity of ECC-concrete composite bar as a function of ECC strip length, l_I

capacity of plain concrete without lost of load carrying capacity. In this case, cracking can be avoided within the plain concrete section when the structure is subject to tensile stress, such as shrinkage stress. For localization of deformation into the ECC strip, sufficient strength difference between ECC and concrete must be guaranteed, i.e. $\sigma_{fc,ECC}<\sigma_{t,ECC}<\sigma_{t,PC}$, as shown schematically in Fig. 7(b). The experimentally determined tensile stress-strain curves of the designed ECC and concrete, which satisfy the above requirements, are shown in Fig. 7(c). In addition, ECC material and plain concrete can be considered as two kinds of cementitious materials with different properties. It is necessary to design the ECC/concrete interface to ensure that damage under tensile stresses occurs inside the ductile strip instead of at the interface area. If first cracking occurs at or close to the ECC/concrete interface, the multiple cracking phenomenon cannot be developed due to the "fiber-end" effect at the interface area, i.e. the fiber bridging is weak at the interface area. Therefore, the present interface design principally overcomes the fiber-end effect along the ECC/concrete interface. As an example, a geometrical method to enhance the ECC/concrete interface will be given below. Fig. 9(a) demonstrates the idea of the ECC/concrete interface geometric design. From stress element analysis at interface, the normal and shear stresses at interface, σ_t and τ_s are given by

$$\sigma_t = \sigma \sin^2(\phi)$$
$$\tau_s = \frac{1}{2}\sigma \sin(2\phi) \tag{4}$$

where σ is the overall tensile stress acting on the slab. ϕ is the angle between ductile material/concrete interface and horizontal line. From Eqn (4), we can see that the normal stress σ_t is a function of ϕ for a given stress level σ. Fig. 9(b) shows the relationship between the interfacial normal stress and the angle ϕ under some typical stress level. σ_t is reduced significantly by lowering the angle ϕ. This indicates that it is possible to prevent interfacial failure with a reasonable interfacial angle for a given interfacial tensile strength. For example, for hot joining, the general interfacial tensile strength should be equal to the minimum value of the tensile strength of concrete and the first crack strength of ECC material. Therefore, this interfacial tensile strength at least can achieve 2-3 MPa after 28 days curing. In this case, as ϕ is selected to be less than 38 degrees interfacial failure can still be prevented even as 5 MPa overall stress is acting on the structure since the interfacial tensile stress is less than 2 MPa. On the other hand, the tensile strength of ECC is normally less than 5 MPa. Therefore, the above selection of ϕ, i.e. 38 degrees, is still conservative [18].

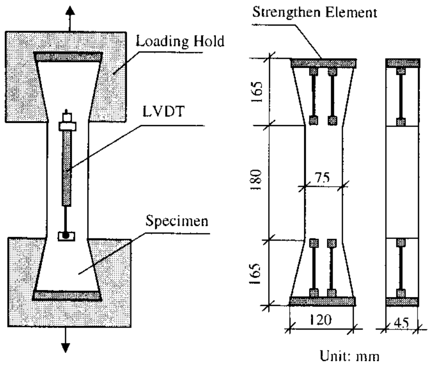


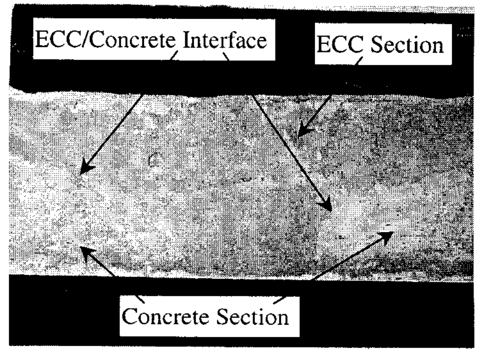
(a) (b)

Fig. 9. (a) ECC/concrete interface geometric design and (b) influence of interfacial angle ϕ on the normal stress at ECC/concrete interface

Experimental Simulation and Results

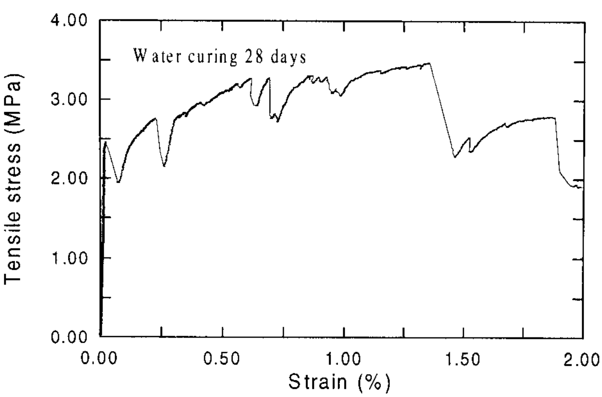
In this section, the validity of the concept will be experimentally verified through uniaxial tensile test on ECC-concrete composite specimen. In the present work, ϕ=30 degree was adopted. The size of specimen used in the tensile tests is shown in Fig. 10(a). In order to cast the concrete sections and the ECC strip at the same time and to ensure an inclined angle of 30 degrees for the ECC/concrete interface, a special casting device was developed. The details on the specimen casting device and procedures can be found elsewhere [18]. The overview of the specimen after casting is shown in Fig. 10(b). The above tensile specimens were cured in water at 23^{0}C and tested at 28 days after casting. The tensile test results on the specimens with ECC strip are shown in Fig. 11(a) in terms of tensile stress versus strain diagrams. As the specimen was loaded in tension, first cracking occurred in the ECC element at around 2.5 MPa followed by multiple cracking due to sufficient difference between the first crack strength of ECC, ultimate tensile strength of ECC and tensile strength of concrete. Deformation was successfully localized into the ECC strip instead of in the concrete section. The width of the microcracks in the ECC strip is less than 0.1-0.2 mm. Furthermore, in the present dimension of the specimen, the strain attained 1.4% at peak load (3.5 MPa). A view of the specimen after cracking in ECC section is shown in Fig. 11(b).

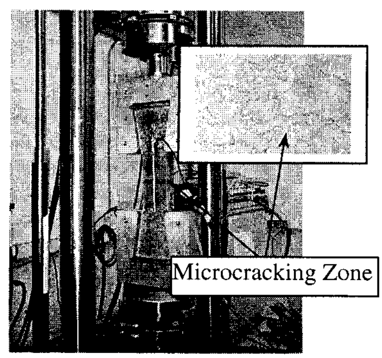




(a) (b)

Fig. 10. (a) Test set-up and geometry of specimen and (b) view of the specimen after casting, the ECC has a slightly darker color than the concrete





(a) (b)

Fig. 11. (a) Tensile stress strain curve of the ECC-concrete composed specimen and (b) a view of cracking in ECC strip, the close-up view of the microcracking zone is shown in insert

CONCLUSIONS

This paper reviews the recent results on the studies of durability enhancements of concrete bridge deck. Fatigue performance of cementitious material can be significantly improved by adding fibers. With the same maximum load level, the fatigue life can be prolonged by several orders of magnitude depending on the fiber types and content. The fatigue crack growth rate in fiber reinforced cementitious composite is found to be governed by the crack bridging degradation behavior of the composite in the fracture zone. A composite slab containing both plain concrete and ECC strips, with proper design at the ECC/concrete interfaces, and careful selection of material properties (i.e. to assure that the tensile strength of concrete higher than that of ECC material), it is possible to localize the tensile deformation into the ECC strip instead of cracking in the concrete section. Due to the strain-hardening performance of the ECC material with high strain capacity (up to 5%), the overall strain capacity and the integrity as well as the fatigue durability of the composite slab can be significantly improved. This concept has been validated with a simple laboratory experimental simulation. Further experiments with larger scale specimens are needed in order to apply the design concept explored in the present work in more realistic field situations.

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