DESIGNING FATIGUE-RESISTANT ENGINEERED CEMENTITIOUS COMPOSITES

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SUMMARY

Engineered cementitious composites (ECC) represent a novel type of fiber-reinforced cement-based material with ultra-high ductility. While it emerges in a wide range of infrastructures, including road surface and bridge deck, it encounters premature failure under fatigue loading. This thesis proposes a novel design approach to enhance the fatigue resistance of ECC. Specifically, the premature fatigue failure of ECC can be mitigated, and the fatigue life of ECC is greatly extended. The proposed design approach is based on the development of a new ECC fatigue theory: the fatigue-dependency of ECC, from the components on the micro-scale to the composite behavior on the macro-scale, is characterized by experiment; the fatigue-dependency is then quantified by a micromechanics-based model, which successfully predicts ECC behavior under fatigue loading. After the establishment of the new fatigue theory, two tailoring techniques, i.e. developing high flexural strength and engaging robust self-healing behavior, are practically conducted under the guidance of the theory. These two tailoring techniques successfully improve the fatigue resistance of ECC. The proposed design approach represents a multi-scale tailoring methodology against the fatigue, which can be extended to other composite materials.
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LIST OF SYMBOLS

\( a \): fiber debonding length
\( a_{\text{flexure}} \): flexural crack length
\( a_0 \): fiber debonding length before cyclic loading phase in the fatigue test
\( A \): cross-section area of ECC specimen
\( A_f \): cross-section area of fiber
\( b \): distance from tensile edge to neutral axis under flexure
\( B \): ECC beam width
\( c \): the first material constant in Paris’ Law
\( C \): the first material constant related to fatigue-induced fiber debonding rate
\( C_d \): the first material constant related to fatigue hardening degree
\( C_s \): the first material constant related to fatigue slip-hardening degree
\( d_f \): fiber diameter
\( E_f \): Young’s modulus of fiber
\( E_m \): Young’s modulus of concrete/ECC matrix
\( f \): snubbing coefficient
\( f' \): fiber strength reduction coefficient
\( G_d \): chemical bond between fiber and matrix
\( H \): ECC beam depth
\( J_b \): fiber-bridging complementary energy
\( J_{\text{tip}} \): crack tip toughness
\( k \): micro-spalling coefficient
\( k_{d} \): tensile loading stiffness of the debonding stage of single-fiber test
\( k_s \): tensile loading stiffness of the slippage stage of single-fiber test
\( K_a \): crack-tip-stress intensity factor from matrix
\( K_b \): crack-tip-stress intensity factor from fiber-bridging
\( K_m \): fracture toughness
\( K_{tip} \): crack-tip-stress intensity factor
\( L_e \): fiber embedment length
\( L_f \): fiber length
\( m \): the second material constant in Paris’ Law
\( M \): the second material constant to related fatigue-induced fiber debonding rate (throughout the thesis except in Chapter 5) / flexural moment (in Chapter 5 only)
\( M_d \): the second material constant related to fatigue hardening degree
\( M_s \): the second material constant related to fatigue slip-hardening degree
\( M_{u} \): flexural moment capacity
\( N \): number of fatigue load cycles
\( N_d \): number of applied fatigue cycles before full-debonding
\( N_f \): number of fatigue load cycles to fatigue failure
\( N_i \): number of applied fatigue cycles after full-debonding
\( P \): tensile load on single-fiber
\( P_{a} \): tensile load on single-fiber immediately before full-debonding
\( P_{b} \): tensile load on single-fiber immediately after full-debonding
\( P_{max} \): maximum load in fatigue test
\( P_{min} \): minimum load in fatigue test
\( P_{r} \): tensile load on single-fiber at the transition point under reloading
\( s \): crack spacing
\( u \): fiber displacement
\( u_0 \): fiber displacement at full-debonding

\( V_f \): fiber fraction by volume

\( w \): crack width

\( \alpha \): Cook-Gorden coefficient

\( \beta \): slip-hardening coefficient

\( \beta_d \): fatigue hardening coefficient

\( \beta_s \): fatigue slip-hardening coefficient

\( \delta_0 \): crack opening displacement at fiber-bridging strength

\( \varepsilon_c \): compressive strain

\( \varepsilon_{cp} \): compressive strength

\( \varepsilon_t \): tensile strain

\( \varepsilon_{tc} \): first cracking strain

\( \varepsilon_u \): tensile strain capacity

\( \sigma_0 \): fiber-bridging strength

\( \sigma_c \): matrix tensile cracking strength (all thesis except in Chapter 5) / compressive stress (in Chapter 5 only)

\( \sigma_{cp} \): compressive strength

\( \sigma_t \): tensile stress

\( \sigma_{tc} \): first cracking strength

\( \sigma_u \): ultimate tensile strength

\( \sigma_{fu} \): in-situ fiber strength

\( \tau \): frictional bond between fiber and matrix (hardening effect considered)

\( \tau_0 \): frictional bond between fiber and matrix

\( \phi \): fiber orientation in single-fiber test/modeling
CHAPTER 1 INTRODUCTION

1.1. Concrete: the construction material in evolution

Construction is one of the oldest industries in the history of human beings, and it still plays important parts in modern society: houses provide shelter for people’s living, highways realize fast commuting and traveling; water dams generate power for industrial production. One important feature of construction industry is that the product, e.g. buildings and infrastructure, is normally of great size and demands huge amount of raw materials.

Since the invention of Portland cement, concrete, the composite using hydrated cement to bind aggregates, has grown to the most used construction materials. In the year of 2008, over 2 billion tons of concrete were produced in the world. The number is fast growing and by the year of 2050, global concrete use is expected to reach four times that of 1990 (Scrivener 2008). The wide use of concrete reflects the advantages of this material: raw materials are widely available; manufacturing is relatively easy and economical; it can achieve excellent compressive strength; it is less susceptible to the biotic deterioration seen in timber and the corrosion seen in steel.

Despite of the advantageous properties, concrete is intrinsically brittle under tension. This weakness may cause problems regarding structural safety, durability, and sustainability. The limited tensile ductility results in low energy absorption of concrete structures under extreme loading like blast and earthquake, which makes the structure vulnerable. Concrete is also susceptible to shrinkage cracking, which makes pathways for the external aggressive agents that cause material deterioration. A good example is that in offshore reinforced concrete structures, chloride ions penetrate through cracks into concrete, accelerating the corrosion of the internal reinforcement; the accumulated rust would cause concrete spalling and ultimately loss of structural integrity. Frequent maintenance and repair are needed for concrete structures, which consumes energy and generates greenhouse gas emission. As a matter of fact, cement production contributes five percent of annual anthropogenic global CO₂ emission (Scrivener 2008).
1.1.1. Fiber reinforced concrete to overcome the brittleness

The effort to overcome the brittleness of concrete had started since it was invented, and the most effective technique is to use dispersed fibers to reinforce the concrete matrix, which technique is known as fiber-reinforced concrete (FRC). The study and application of FRC started over 40 years ago. So far, a wide range of fibers, in terms of composition (steel, natural, polymer, etc.), strength (from about 100 MPa to about 3500 MPa), geometry (straight, hook-ended, indented, twisted, etc.), and dimension (length of several millimeters to several centimeters, diameter of several micrometers to several millimeters), have been used as reinforcement (Zollo 1997, Brandt 2008). The inclusion of dispersed fibers successfully mitigates concrete brittleness by improving the post-cracking behavior. The fiber-bridging controls the crack propagation and enhances fracture toughness (i.e. energy absorption) of concrete (Gopalaratnam et al. 1991, Banthia and Sappakittipakorn 2007). The shrinkage cracking is also controlled in FRC; as a result the penetration of external agents is greatly reduced and the material durability is improved.

For a special class of FRC, i.e. high-performance fiber-reinforced cementitious composites (HPFRCC), the brittleness is even completely eliminated (Naaman 2008). For conventional FRC, the tensile load capacity starts to decrease as the first crack appearing (referred as strain-softening), and the tensile strain capacity is similar to conventional concrete (below 0.05%); for HPFRCC, the tensile load continues to increase as multiple cracks progressively generated (referred as strain-hardening), consequently the tensile strain capacity is increased by an order to around 1%. The tensile strain-hardening feature greatly enhances the fracture toughness of FRC.

Nevertheless in the early days most HPFRCC mix designs adopted very high fiber content to achieve strain-hardening. For example, SIFCON, an early version of HPFRCC, adopted steel fiber content from 4% to 15% by volume (Lankard 1984, Naaman and Homrich 1989); Cheyrezy et al. (2004) developed HPFRCC with tensile strain capacity of 0.5% by adopting polyvinyl alcohol (PVA) fiber at 4% by volume. At such high fiber content, fiber dispersion cannot reach a satisfactory state unless special
technique is used. For example, SIFCON was manufactured by pouring slurry of cement paste over a bed of steel fibers; Marikunte et al. (1999) used extrusion technology to produce HPFRCC. Due to the high fiber content and difficulty in fiber dispersion, the cost on HPFRCC was very high and the application was limited. There was a need to tailor HPFRCC for achieving strain-hardening behavior and high ductility with relatively low fiber content.

1.1.2. Engineered cementitious composites: multi-functionality based on micromechanics

During mix designing, many other factors besides fiber content, such as fiber dimension, fiber strength, matrix composition, etc., would influence the fiber-bridging in HPFRCC. Trial-and-error mix designing and material testing was conducted in attempt to understand these factors. It was soon realized that the fiber-bridging is such a complicated system, thus only after understanding all the parameters regarding fiber, matrix, and fiber-matrix interface on the micro-scale level can HPFRCC be confidently tailored. Along this way, Li and his research group from University of Michigan established a micromechanics-based model to predict the fiber-bridging behavior and design the tensile behavior of HPFRCC (Yang et al. 2008). Under the guidance of this model, the fiber content of HFPRCC was successfully reduced to below 2% by volume, while the tensile strain capacity was greatly increased to be more than 5%.

This unique group of HPFRCC is referred as engineered cementitious composites (ECC), as its mix proportion design, which is determined with the micromechanics-based model, is an engineering outcome. So far, a wide range of ECCs has been developed around the world. ECC mix designs and basic mechanical properties available in literature are summarized in Table 1.1 and 1.2 respectively (Li et al. 1995, Li et al. 2001, Kim et al. 2007, Wang and Li 2007, Yang et al. 2007, Sahmaran et al. 2009, Yang and Li 2010, Zhou et al. 2010). As shown in Table 1.1, ECC can be developed with a wide range of raw materials that are used for conventional FRC. As shown in Table 1.2, the tensile strength and ductility of ECC does not solely depend on its compressive strength and vary in a wide range. It is possible to tailor the tensile
properties and compressive properties of ECC separately, so that specific mechanical properties can be achieved for different applications.

As illustrated in Fig. 1.1, under the guidance of micromechanics-based model, more special attributes can be tailored into ECC, as additions to the basic mechanical properties (Li 2012). For example, the crack width in ECC can be controlled so that the self-healing property is introduced (Yang et al. 2009); the change of material resistivity with crack development can be used to develop self-sensing ECC for structural health-monitoring (Ranade et al. 2014); when the low-density fillers, like air bubbles (generated by air entrainment admixture), expanded perlite sand, or hollow glass bubbles, are used, lightweight ECC can be developed (Wang et al. 2003); when TiO$_2$ powders are added into the mixture, the self-cleaning functionality is added to ECC (Zhao and Yang 2014). Actually, among the ECC research community there is an obvious trend of developing new attributes to be added into this material; and in such a trend, the importance of the micromechanics-based model must be emphasized. The details of this model are reviewed in Chapter 2.

The micromechanics-based model of ECC elevates the material design of concrete, from the empirical trial-and-error for single target property, to systematic selection of raw materials for the combination of favorable attributes. The expanded spectrum of material properties, when considered on the structural design level, results in the novel design concept named Integrated Structures and Materials Design (ISMD) (Li 2007). As shown in Fig. 1.2. ISMD is a design platform for the collaboration between structural engineers and material engineers: it starts with determining the desirable material properties based on the structural performance requirements (step 1); followed by practically developing the material with the desirable properties (step 2); and finally designing and evaluating the structures adopting the developed material (step 3). The material properties and structural performance can be further optimized by repeating the three design steps. In this sense, ECC is not only another new type of concrete, but also represents a novel design approach as reflected in ISMD.
When replacing conventional concrete in reinforced structural members, such as beams, columns, walls, and joints, reinforced ECC (R/ECC) exhibits superior load-carrying capacity, deformability, and energy absorption (Fischer and Li 2002, Kesner and Billington 2002, Li and Wang 2002, Fischer and Li 2003). With the superior mechanical performance and added attributes, ECC are increasingly applied in field application (Fig. 1.3) (Li 2003). ECC was initially used for partially repairing and retrofitting as self-consolidating sprayed material (Kong et al. 2003, Kim et al. 2004). Currently, it is emerging in a few full-scale applications such as the full-scale ECC link slabs (between bridge decks) in United States (Li et al. 2005), a 27-story high-rise residential building adopting R/ECC in the coupling beams for earthquake safety (Maruta et al. 2005), the ECC/steel composite deck of Mihara Bridge in Japan (Kunieda and Rokugo 2006), and the ECC precast pavement for the planned electrified road in Singapore (Nen et al. 2014).

It is noticed that for many infrastructural applications, such as pavement and bridge deck, ECC is required to sustain repeated loads. For these infrastructures, fatigue failure is a great concern for material and structural design. As a result, good fatigue resistance is a favorable attribute that needs to be tailored into ECC. Before Section 1.3 and 1.4, which discuss the importance and method to engage robust fatigue resistance, the fatigue behavior of concrete (including FRC and ECC) is reviewed in Section 1.2.

1.2. Fatigue: nature of engineering material

The word fatigue originates from Latin and means ‘to tire’. It was commonly associated with physiological weariness, but now this terminology is also widely accepted in engineering, meaning the deterioration and damage of materials under cyclic loads. Fatigue is technically defined as ‘changes in material properties due to the repeated application of stresses or strains, which usually leads to cracking or failure’ (Suresh 1998).

The fatigue nature of engineering material was first noticed in metal, after several catastrophic failures of metal structures. For example, in 1843 the fatigue-induced
failure of locomotive front axle caused a serious railway accident near Versailles. Wohler later conducted systematic investigations, noting that the strength of steel by fatigue was appreciably lower than its static strength. His work characterized fatigue behavior with stress-life ($S$-$N$) curve. Griffith in 1921 used the energy concepts to quantitatively explain the fracture in brittle solids; based on this energy concept (Griffith 1921), Irwin in 1957 showed the stress singularity ahead of a crack could be expressed in terms of the stress intensity factor $\Delta K$ (Irwin 1957); and soon Paris suggested that the fatigue crack propagating rate (mm per cycle) is correlated to the $c \cdot (\Delta K)^m$, (where $c$ and $m$ are materials coefficients) which later is referred as the Paris’ Law (Paris and Erdogan 1963).

1.2.1. Fatigue of concrete

The majority of fatigue research reported in literature focus on metallic materials. However, there has been increasing interest in the fatigue of nonmetallic materials, such as ceramics and composites. As a group of brittle cement-based composites, fatigue of concrete also gains enormous attentions.

Many concrete structures, such as pavement, bridge deck, wind turbine, and railway sleepers, are designed to sustain fatigue load of millions of cycles. Under fatigue loads the crack in concrete would propagate, first stably and later unstably, resulting in catastrophic failure. It is necessary to characterize the fatigue resistance of concrete ($S$-$N$ curve and materials coefficients in Paris’ Law) for structural design. Also it is of great interest to extend the service life of structure by improving the fatigue resistance.

For plain concrete, the fatigue strength, like most mechanical properties, is mainly correlated to the concrete grade, marked by monotonic compressive strength. Fatigue tests have been conducted with plain concrete including different supplementary cementitious materials, different aggregate types, and different admixtures (Oh 1991, Taylor and Tait 1999, Lee and Barr 2004); despite the great variety in mix design, the results indicate that only by enhancing the concrete grade can fatigue performance be improved. It is possible to achieve higher concrete grade by using lower water-to-
cement ratio and denser aggregate compaction; however it comes with more serious brittleness, higher cost, and additional environmental impact.

Fiber reinforced concrete (FRC) can achieve noticeably longer fatigue life, compared with plain concrete of similar grade (Lee and Barr 2004). This is because the fiber-bridging reduces the stress intensity factor of near the crack tip, decelerating the crack propagation (Li and Matsumoto 1998). The extent of improvement on fatigue resistance of FRC depends upon the micromechanics factors that determine the fiber-bridging, such as geometry, dimension, modulus, strength, content of fiber (Jun and Stang 1998, Singh et al. 2005). At this moment, however, there seems limited knowledge regarding how to quantitatively determine the effect of these factors on fatigue deterioration of fiber-bridging.

1.2.2. Fatigue of ECC: a new attribute to the material

There have been several studies on the fatigue behavior of ECC on the structural member level, especially the flexural fatigue behavior. The results indicated that the fatigue life of ECC, due to the doubled modulus of rupture (MOR), could be longer than that of conventional FRC by several orders (of load cycles) (Qian 2007). Zhang and Li (2002) studied the flexural fatigue performance of ECC overlay on the top of cracked pavement; the novel material with ultra-high ductility not only greatly extended the fatigue life of the overlaid system, but also eliminated the reflective fatigue cracking.

As discussed in the preceding section and shown in Fig. 1.1, ECC is a novel group of construction material with multi-functionality: the basic mechanical properties (strain-hardening and ultra-high ductility) combined with selected attributes (self-healing, self-cleaning, self-sensing etc.) for specific application. The high fatigue resistance of ECC is expected to be added to the list as a special attribute for applications like pavement, bridge deck, or wind turbine. However, in the study of ECC fatigue behavior, it is noticed that after fatigue degradation the material would fail in a premature manner, with less crack numbers and remarkably reduced tensile ductility (Fig.1.4)
(Suthiwarpipirak et al. 2004, Qian 2007). Such observations alerts material engineers to the risk of ECC losing its strain-hardening and other relevant attributes under fatigue loads. It is a challenge for material engineers to add fatigue resistance to ECC while sustaining its basic properties.

1.3. Research motivation and objectives
In the pursuit of sustainability of infrastructures, structural durability plays an important part. As structural durability improves, less repair and maintenance activities are required. It in turns will cut the energy consumption and pollutant emission. Considering the example of road, for every time of pavement repair, fuel energy will be consumed for producing paving material (concrete or asphalt) and running the equipment on the construction site; if the repair requires road closure, additional energy will be consumed by the detouring or slowed traffic flow, and in turn more hazardous air pollutants.

Deterioration of infrastructures is a complicated process that involves different mechanisms of material degradation under mechanical load and environmental load. Among these deteriorating mechanisms, fatigue of concrete is believed to be an important one for a wide spectrum of infrastructures, like pavement, railway sleeper, offshore infrastructures, and wind turbine, which sustain repetitive loads from automobiles, trains, waves, and wind. For these infrastructures, fatigue would induce progressive crack propagation, combined with increasing penetration of external aggressive agents (CO₂, chloride ion, and sulfate), resulting in advanced structural failure.

Engineered cementitious composites (ECC) are a group of strain-hardening fiber-reinforced cementitious composites with ultra-high ductility. In recent studies, ECC showed superior fatigue resistance than plain concrete and congenital FRC. It has potential to greatly decelerate the fatigue-related distress in the aforementioned infrastructures, by replacing the traditional concrete with ECC.
Despite the great potential, some problems must be solved before practically designing fatigue-resistant ECC structures. Theoretically, the fundamental fatigue mechanism of ECC remains unclear, especially regarding the micro-scale ECC component; this knowledge gap prevents the engineers to explain or overcome the ECC ductility loss under fatigue. Practically, the relatively high expense of ECC requires longer service life of structures; there is still great room to extend the fatigue life of ECC structures by improving the materials design. The motivation of this thesis is to develop systematic method to improve the fatigue resistance of ECC. Specifically, the fatigue life of ECC is to be extended while the high ductility sustained.

1.4. Research scope and thesis organization
The research scope of this thesis is shown in Fig.1.5. The ultimate goal, as shown at the top, is to improve the fatigue resistance of ECC. As the foundation of current research, the theoretical framework of ECC fatigue is first established. On one hand, the source of fatigue-dependency of ECC in component micro-scale level is first investigated (Chapter 3). On the other hand, a fatigue-sensitive micromechanics-based model to predict ECC fatigue behavior is established based on the fatigue-dependency of ECC components (Chapter 4). After the theoretical framework is established, the fatigue-induced ECC ductility loss can be understand and explained. Under the scope of fatigue theory, ingredient tailoring is practically conducted to extend the fatigue life of ECC. Two strategies are considered: mechanically enhancing the flexural strength, i.e. MOR (Chapter 5) and chemically engaging robust self-healing (Chapter 6) are proven to be very effective. The thesis organization is based on the research scope above and described below.

In Chapter 2, literature on the state-of-the-art of relevant research topics is thoroughly reviewed. Specifically, ECC as a unique type of fiber-reinforced cementitious composites is briefly discussed in the aspect of mix design, mechanical properties, and application; the micromechanics-based model for ECC under monotonic loading is demonstrated, from the micro-scale to macro-scale; fatigue behavior and self-healing are reviewed, as material degradation mechanism and a technology to improve the
durability of cement-based construction materials, respectively. Towards the end of this chapter, knowledge gaps of this thesis are identified. Chapters 3-6 present the research work which fills the knowledge gaps identified in Chapter 2. Each chapter deals with a specific approach to achieve high fatigue resistance in ECC.

Chapters 3 and 4 attempt to establish a theoretical framework for ECC fatigue behaviors. In Chapter 3, a serial single-fiber pullout tests that characterize the fatigue-dependency of ECC components on the micro-scale is reported, where the effects of fatigue loading on fiber and interfacial degradation are identified. In Chapter 4, the characterized fatigue-dependency is integrated into the established monotonic model, from the single-fiber pullout behavior on the micro-scale, to the fiber-bridging stress-crack opening relation on the meso-scale, to the flexural fatigue behavior on the macro-scale. The new model successfully explains the ductility loss and predicts fatigue life of ECC under fatigue loading.

In Chapter 5, the effects of constitutive tensile and compressive behavior on ECC MOR are determined by a parametric study; on the basis of parametric study, ECC with high-MOR is developed with locally available raw materials; the improvement on fatigue resistance by high-MOR ECC is experimentally verified; the structural performance of precast pavement made of local high-MOR ECC is evaluated.

Chapter 6 presents the experimental work on improving the self-healing robustness of ECC and the feasibility of improving ECC fatigue performance by the self-healing approach. Currently crack width and curing condition have been identified as two important factors that affect the robustness of ECC self-healing (Yang et al. 2009), but the effects of chemical composition was rarely studied. This chapter attempts to determine the optimal content of supplementary cementitious composites, a class of raw ingredients for ECC which potentially can alter the self-healing behavior. The optimized mix design for self-healing is selected, and used to check the possibility of fatigue life extension by self-healing. The results indicate that fatigue resistance of ECC can be remarkably improved by robust self-healing.
Overall conclusions from current study are summarized in Chapter 7, and some future works worthy of further investigations are outlined.
Reference


Li, V. C., M. D. Lepech and M. Li (2005). Field demonstration of durable link slabs for jointless bridge decks based on strain-hardening cementitious composites.


Table 1.1 Summary of ECC mix designs

<table>
<thead>
<tr>
<th>Composition</th>
<th>Content by mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>Portland cement</td>
<td>0.15-1.00 of binder&lt;sup&gt;1&lt;/sup&gt;</td>
</tr>
<tr>
<td>Water</td>
<td>0.24-0.60 of binder</td>
</tr>
<tr>
<td>Supplementary cementitious materials (SCM)</td>
<td></td>
</tr>
<tr>
<td>- Ground granulated blast furnace slag (GGBS)</td>
<td>0.00-0.85 of binder</td>
</tr>
<tr>
<td>- Coal fly ash (FA)</td>
<td></td>
</tr>
<tr>
<td>- Silica fume (SF)</td>
<td></td>
</tr>
<tr>
<td>- Limestone powder (LP)</td>
<td></td>
</tr>
<tr>
<td>Aggregates</td>
<td>0.00-1.67 of binder</td>
</tr>
<tr>
<td>- Micro-silica sands (≤ 0.20 mm)</td>
<td></td>
</tr>
<tr>
<td>- River sands (≤ 2.36 mm)</td>
<td></td>
</tr>
<tr>
<td>Fiber</td>
<td>2% by volume</td>
</tr>
<tr>
<td>- Polyvinyl alcohol (PVA) fibers</td>
<td></td>
</tr>
<tr>
<td>- Polypropylene (PP) fibers</td>
<td></td>
</tr>
<tr>
<td>- Polyethylene (PE) fibers</td>
<td></td>
</tr>
<tr>
<td>Admixture for rheology control</td>
<td></td>
</tr>
<tr>
<td>- Superplasticizer (SP), to increase the flowability</td>
<td>0.00-0.04 of binder</td>
</tr>
<tr>
<td>- Viscosity modifying admixture (VMA), to increase the viscosity</td>
<td></td>
</tr>
</tbody>
</table>

<sup>1</sup>binder means the combination of Portland cement and SCM

---

Table 1.2 Summary of ECC basic mechanical properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>First cracking strength (MPa)</td>
<td>1.50-4.64</td>
</tr>
<tr>
<td>Tensile strength (MPa)</td>
<td>1.80-5.60</td>
</tr>
<tr>
<td>Tensile strain capacity (%)</td>
<td>0.24-5.60</td>
</tr>
<tr>
<td>Compressive strength (MPa)</td>
<td>21-72</td>
</tr>
</tbody>
</table>
Fig. 1.1 Adding special attributes to ECC with the micromechanics-based model

Fig. 1.2 Integrated Structures and Materials Design (ISMD)
Fig. 1.3 Field application of ECC: (a) earth retaining wall repaired with ECC, Japan (Rokugo et al. 2005); (b) ECC link-slab, Grove Street Bridge, United States (Li et al. 2005); (c) R/ECC coupling beam, Japan (Maruta et al. 2005); (d) ECC/steel composite deck, Japan (Kunieda and Rokugo 2006).
Fig. 1.4 Crack number in ECC M45 at the failure moment reduces with flexural fatigue load level (Qian 2007)
Fig. 1.5 Research framework of the thesis
CHAPTER 2 LITERATURE REVIEW

The overall goal of the thesis is to improve the fatigue resistance of engineered cementitious composites (ECC). In the last chapter, the research scope of the thesis is illustrated: a theoretical framework about ECC fatigue is to be established first; then the two ECC tailoring strategies, i.e. high-MOR and self-healing will be implemented to extend the fatigue life of ECC. This chapter reviews relevant literatures, with focus on the micromechanics-based ECC design, fatigue durability of concrete, and the self-healing behavior of cement-based materials. At the end of this chapter, the research problems and knowledge gaps of this thesis are summarized based on the literature review.

2.1. Engineered cementitious composites

2.1.1. Generals

ECC is a class of ultra-ductile fiber-reinforced cementitious composites (Li 2003). Figure 2.1 shows the uniaxial tensile stress-strain curve of ECC M45, a standard version of ECC developed in the University of Michigan. It is observed that ECC M45, by adopting relatively small fiber content (2% or less by volume), has several unique features. First, the load capacity keeps increasing after the first matrix crack appears, which is referred as pseudo strain-hardening behavior, or strain-hardening for short; second, the tensile strain can be as high as 5%, comparable to metal, hundreds times of conventional concrete; third, the crack width of ECC, despite the strain increase, is kept around 60 μm on average, well below that of typical concrete crack (in other words the high strain capacity is the result of sequential formation of multiple cracks rather than crack widening). These three features in ECC M45, i.e. tensile strain-hardening, high ductility, and multiple fine cracks, are considered as the basic properties of this class of materials.

With these basic properties, ECC, plain or reinforced, can achieve superior structural performance. The improved structural performance by ECC is mainly reflected under severe mechanical loading. Investigations have been conducted in beams (Kanda et al.}
1998, Fukuyama et al. 1999), columns (Fischer and Li 2002), steel beam-concrete column joints (Parra-Montesinos and Wight 2000), smart frame (Fischer and Li 2003), and different types of precast structural members made of ECC. When R/ECC is used as replacement of R/C, the structural members/frames demonstrate superior seismic resistance response (Fig.2.2) and minimum demand for post-earthquake repair; ECC structural member especially demonstrates higher strength and spalling resistance in the stress-concentration zone, in the cases like pullout of anchored reinforcement (Qian and Li 2011), shear response of stud connection (Qian and Li 2006), and reflective cracking of rigid overlay (Zhang and Li 2002).

ECC also shows superior performance under severe environmental loading when compared to conventional plain or fiber-reinforced concrete. The improvement on environmental durability is mainly witnessed under the cracked state. Permeation (of fluid) through hydraulic head, sorption (of fluid) through capillary suction, and diffusion (of certain ions) due to the ion concentration gradient, are recognized as the three major mechanisms for external agents to penetrate into concrete. As a result of the intrinsic fine crack width, the external agent penetration to ECC through these mechanisms is maintained at a very low level. Lepech and Li (2009) experimentally studied the water permeation through cracked ECC and concrete, the permeability coefficient of cracked ECC ($\times 10^{-10}$ m/s, depending on the number of cracks) is smaller than cracked reinforced mortar by several orders ($\times 10^{-9}$ to $\times 10^{-4}$ m/s). Sahmanran and Li (2009) experimentally studied the sorptivity of cracked ECC, and reported that ECC with multiple cracks has sorptivity ranging from 0.032 mm/min$^{1/2}$ to 0.142 mm/min$^{1/2}$, comparable to that of uncracked conventional concrete (0.09 mm/min$^{1/2}$ to 0.17 mm/min$^{1/2}$ depending on the water to cement ratio used). Sahmaran et al. (2007) experimentally studied the chloride diffusion into ECC and mortar, and reported that the effective diffusion coefficient of deformed and cracked ECC can be smaller than that of mortar experiencing similar deformation by an order. They also studied corrosion resistance of R/ECC prism using accelerated corrosion test, and reported that
the reinforcement mass loss in R/ECC is much slower than that R/Mortar, as a result of the constrained cracking development (Sahmaran et al. 2008).

Despite the improved structural and environmental resistance, drawbacks of ECC M45 should not be overlooked, especially for specific application. Researchers attempt to tailor the mix design to develop new versions of ECC to overcome these drawbacks. One major drawback is that the mix design of ECC M45 is associated with heavy environmental impact. As coarse aggregates are not used, ECC M45 adopts high content of cement (roughly a quart of the total ECC weight) as compared with conventional concrete, which consumes more energy for cement manufacturing. Yang et al. (2007) enhanced the content of coal fly ash, a type of industrial waste, to replace more cement, which enhanced the greenness and also ductility of ECC. The manufacturing cost of ECC M45 is relatively high because it adopts expensive polyvinyl alcohol (PVA) fibers. Yang and Li (2010) used cheaper copolymer polypropylene (PP) fibers to replace PVA fibers, and successfully developed ECC with high ductility (around 3%). It is desirable to develop high early strength for ECC as it is expected to be used for precast members and fast repairing work. Li and Li (2011) successfully developed high-early-strength (HES) ECC by replacing the Type I Portland cement with finer Type III Portland cement and adding hydration-accelerating admixtures.

There is a trend to tailor ECC M45 to achieve desirable properties of ECC for specific application, besides the three examples above. However, it is possible that after altering the mix design, the basic properties of ECC, i.e. strain-hardening behavior, high ductility, and multiple-cracking behavior, will be eliminated. As a matter of fact, the basic ECC properties are achieved by a micromechanics-based designing method, and it is necessary to review the micromechanics involved in this method before successfully tailoring ECC for fatigue durability. More detailed reviews of this micromechanics-based design framework are given in the following section.
2.1.2. Micromechanics-based design framework of ECC

The scale-linking in ECC design is illustrated in Fig. 2.3. Properties of ECC on the micro-scale (mm-cm), i.e. strain-hardening, tensile ductility, and multiple fine cracks, depend on the post-cracking fiber-bridging on the meso-scale (μm-cm); fundamentally, the fiber-bridging relation, governed by the ambient tensile load-crack opening (σ-δ) curve, is determined by the properties of ECC component, i.e. fiber, matrix, and fiber-matrix interface on the micro-scale (nm-μm). Micromechanics model links the micro-scale constituent parameters to the meso-scale fiber-bridging behavior; strain-hardening criteria connect the meso-scale fiber-bridging to the macro-scale tensile strain-hardening. Once the connection is established across the three scales, material engineers would be able to design ECC properties by tailoring the micro-scale parameters. Such scale-linking not only reflects the nature of ECC, but also represents a holistic design framework based on micromechanics.

Strain-hardening criteria

The pseudo strain-hardening behavior of ECC is a result of sequential development of matrix multiple cracks. During the development these cracks, every individual crack must propagate in steady manner, i.e. the crack initiating from matrix defect, propagating and forming a flat crack across the whole section. In this condition, few fibers would rupture or be pulled out so that the fiber-bridging can sustain the ambient load to trigger the initiation of a new crack. Such crack propagation manner is referred as steady-state crack, and repeated formation of steady-state crack results in multiple-cracking and strain-hardening behavior of ECC.

The condition for steady-state crack was quantitatively analyzed by Marshal and Cox (1988) based on the J-integral method. When the fiber-bridging is characterized by σ-δ relation, the condition for steady-state crack can be expressed as the following:

\[ J_{tip} \leq \sigma_0 \delta_0 - \int_0^{\delta_0} \sigma(\delta) d\delta \equiv J'_b \]  

(Eqn. 2.1)
i.e. the fiber-bridging complementary energy $J_b'$, as defined by the right-hand-side of Eqn.2.1, must exceed the crack tip toughness $J_{tip}$, as defined as by the left-hand-side of Eqn.2.1. $J_{tip}$ is approximately equal to the matrix toughness $K_m^2/E_m$ at small fiber content, where $K_m$ is the matrix fracture toughness and $E_m$ is the matrix Young’s modulus. Such a condition is based on the understanding of crack propagation from the energy perspective; specifically, the external work, besides stretching the fiber-bridging and widening crack opening, must provide sufficient energy to propagate the crack tip normal to the direction of tensile loading Fig.2.4 schematically illustrates the energy balance during steady-state crack propagation.

Prior to steady-state crack propagation, it is necessary for a micro-crack to initiate (from a matrix flaw) at an ambient load level below the fiber-bridging capacity. This consideration represents the other condition for strain-hardening, as defined by Eqn.2.2, that the matrix tensile cracking strength $\sigma_c$ must not exceed the maximum fiber-bridging strength $\sigma_0$:

$$\sigma_c \leq \sigma_0 \quad (\text{Eqn.2.2})$$

where $\sigma_c$ is related to the pre-existing flaw size and the matrix fracture toughness $K_m$.

Eqn.2.1 and 2.2. are referred as the energy criterion and strength criterion for strain-hardening respectively. Satisfaction of both equations is necessary for ECC to achieve tensile strain-hardening behavior; otherwise tensile strain-softening behavior, which is typically seen for conventional FRC, would appear. Practically, due to the random nature of pre-existing flaw size and fiber distribution in cement composites, a large margin between $J_b'$ and $J_{tip}$, as well as $\sigma_0$ and $\sigma_c$ is preferred. Therefore the pseudo strain-hardening (PSH) performance indices (Yang and Li 2010), i.e PSH energy $(=J_b'/J_{tip})$ and PSH strength $(=\sigma_0/\sigma_c)$ have been used to quantitatively evaluate the margin and likelihood of strain-hardening:

ECC with larger PSH indices have a better chance of saturated multiple-cracking and subsequently higher strain-capacity. It has been demonstrated experimentally that PSH
energy \(>3\) and \(PSH\) strength \(> 1.45\) produce saturated multiple-cracking behavior in ECC with PVA fibers (Kanda 1998, Li et al. 2002).

**Micromechanics-based model**

The strain-hardening criteria clarifies the importance of accurately predicting the fiber-bridging constitutive law, i.e. ambient tensile stress-crack opening \((\sigma-\delta)\) relation. The fiber-bridging across a crack essentially is a multiplication of single fiber-matrix interaction. In order to establish the fiber-bridging behavior, single-fiber pullout behavior must be characterized and modeled; and the pullout of individual fibers need to be summed considering the variation in fiber distribution, fiber orientation, possibility of fiber rupture. Following this approach, researchers successfully developed a micromechanics-based analytical model to predict the monotonic \(\sigma-\delta\) relation. Experimental characterization and modeling details are reviewed here.

When a single fiber is monotonically pulled out from the matrix, crack initially occurs at the fiber-matrix interface near the surface. As the load increases, the initial crack would ‘tunnel’ from the surface into to deeper part of matrix, until fiber is fully debonded from matrix (Redon et al. 2001). As shown in Fig.2.5, the event of full-debonding (noticed by the sudden load drop from \(P_a\) to \(P_b\)) divides the fiber pullout into the preceding fiber debonding stage and the following fiber slippage stage.

Fiber debonding from matrix is essentially type II crack propagation. Two approaches, i.e. shear stress-based approach and fracture mechanics-based approach are used to explain the crack propagation (Stang et al. 1990). In shear stress-based approach, crack propagates when the shear strength of the fiber-matrix interface is exceeded; in the fracture mechanics-based approach, crack propagates if the energy released by unit area of debonded interface exceeds the interfacial toughness, or the chemical bond. For the pullout of polymer fiber like PVA, the fracture mechanics-based approach is preferred as it can better capture the load drop toward full debonding (Leung and Li 1990, Li and Stang 1997).
Based on the fracture mechanics-based approach, Lin and Li. (Lin et al. 1999) developed an analytical model for the single-fiber tensile load-fiber displacement \((P-u)\) relation. In the fiber debonding stage (Eqn.2.3a), the load \(P\) is resisted by the chemical bond \(G_d\) at the bonded interface as well as the frictional bond \(\tau_0\) at the debonded interface. After full debonding, i.e. in the fiber slippage stage, there would be no ongoing propagation so that only the frictional bond dominates pullout behavior (Eqn.2.3b). In this stage, due to the large relative displacement between fiber and matrix, the fiber surface is abraded and roughened, resulting in stronger friction. Such slippage-induced friction increase is referred as slip hardening, and slip hardening coefficient \(\beta\) is introduced into the model.

\[
P = \sqrt{\frac{\pi^2(\tau_0 u + G_d)E_fd_f^3(1+\eta)}{2}}, \quad u \leq u_0 \text{ the debonding stage} \quad (\text{Eqn.2.3a})
\]

\[
P = \frac{\pi \tau_0}{d_f}(L_e + u_0 - u)[1 + \frac{\beta(u-u_0)}{d_f}], \quad u \geq u_0 \text{ the slippage stage} \quad (\text{Eqn.2.3b})
\]

\[
u_0 = \frac{2\tau_0 L_e^2(1+\eta)}{E_fd_f} + \frac{L_e}{E_f} \frac{8G_dE_f(1+\eta)}{d_f} \quad (\text{Eqn.2.3c})
\]

where \(E_f\) is the fiber Young’s modulus; \(d_f\) is the fiber diameter; \(\eta=EfV_f/E_m(1-V_f)\) where \(E_m\) is the matrix Young’s modulus and \(V_f\) is the fiber content by volume, at a small fiber content as in ECC, \(\eta\) approaches to zero; \(L_e\) is the fiber embedment length; \(u_0\) is the fiber displacement at the full debonding moment, calculated from Eqn.2.3c.

In this analytical model, the chemical bond \(G_d\), frictional bond \(\tau_0\), and slip hardening coefficient \(\beta\) depend on the fiber and matrix properties, and must be determined from the single-fiber pullout tests as shown in Eqn.2.4.

\[
G_d = \frac{2(P_a-P_b)^2}{\pi^2E_fd_f^3} \quad (\text{Eqn.2.4a})
\]

\[
\tau_0 = \frac{P_b}{\pi d_f L_e} \quad (\text{Eqn.2.4b})
\]
\[
\beta = \frac{d_f}{L_e} \left( 1 + \frac{1}{\pi \tau_0 d_f} \frac{\Delta P}{\Delta u} \right), \quad u \to u_0 \tag{Eqn.2.4c}
\]

where \(\Delta P/\Delta u\) is the slope of the beginning part of the slippage stage.

In ECC, most fibers are oriented at an arbitrary angle \(\varphi\) relative to the crack plane. The effect of fiber orientation is considered. The misaligned fibers are subjected to additional frictional stress due to the compression normal to the fiber direction. Morton and Grove (1976) suggested Eqn.2.5 to account for the additional load resistance due to \(\varphi\):

\[
p(\varphi) = p(0) e^{f\varphi} \tag{Eqn.2.5}
\]

where \(f>0\) is referred as snubbing coefficient.

In the single-fiber pullout test, it is possible that the fiber is ruptured by the increasing load, instead of being completely pulled out. It is therefore important to determine the in-situ fiber strength. Kanda and Li (Kanda and Li 1998) experimentally investigated the in-situ fiber strength, and reported the tensile strength of embedded fiber is appreciably lower than its nominal strength. This is because the slippage would abrade and damage the fiber, reducing its load capacity. They also noticed fiber orientation \(\varphi\) could lead to more severe fiber abrasion, i.e. further reduced in-situ fiber strength. Eqn.2.6 is therefore suggested to account for this effect:

\[
\sigma_{fu}(\varphi) = \sigma_{fu}(0) e^{-f'\varphi} \tag{Eqn.2.6}
\]

where \(\sigma_f\) is the in-situ fiber strength and \(f'>0\) is referred as fiber strength reduction coefficient.

**Fiber-bridging model**

In order to determine the fiber-bridging constitutive law, i.e. the ambient tensile load-crack opening (\(\sigma-\delta\)) relation, the single-fiber pullout behavior predicted by the \(P-u\) curve must be summed, considering the variation of fiber distribution (location and orientation).
In the fiber-bridging model, the fiber embedment length $L_e$ is correlated to $z$, the distance from the fiber center to the crack surface, which varies in the range of $(0, L_f/2)$. The fiber location variation is captured by probability function $p(z)$ (Eqn.2.7) (Lin et al. 1999):

$$p(z) = \frac{2}{L_f}$$

(Eqn.2.7)

The variation of fiber orientation is captured by probability function $p(\phi)$ (Eqn.2.8) (Lin et al. 1999):

$$p(\phi) = \frac{2}{\pi}, \quad \text{for 2-D fiber distribution}$$

(Eqn.2.8a)

$$p(\phi) = \sin(\phi), \quad \text{for 3-D fiber distribution}$$

(Eqn.2.8b)

Eqn.2.8a is used for ECC specimens with thickness similar to fiber length where fiber can only be distributed in two-dimensional manner, while Eqn.2.8b is used otherwise.

Besides the variation of fiber distribution, there are other factors that influence the fiber-bridging $\sigma-\delta$ curve. Two-way fiber pullout (Wang et al. 1988), matrix micro-spalling (Kanda and Li 1998), and Cook-Gorden effect (Li et al. 1993), for example, are three mechanisms that contribute to additional $\delta$. These mechanism are considered in the recently optimized fiber-bridging model (Yang et al. 2008). Specifically, fiber displacement of both sides of the crack is considered; matrix micro-spalling coefficient $k$ and Cook-Gorden coefficient $\alpha$ are introduced.

The summation of individual fibers can result in explicit expression of $\sigma(\delta)$, with considering the fiber rupture and two-way fiber pullout during the slippage stage (Huang et al. 2015), however, fiber rupture during the debonding stage, matrix micro-spalling, and Cook-Gorden effect have not been accounted. It can also be completed with numerical method. Fig.2.6 illustrates the procedure of the numerical computation, as suggested by Yang, et al. (2008). Table 2.1 summarizes the micro-scale parameters needed as the input for this micromechanics model, where the values corresponding to the PVA fiber and the matrix mix design used for ECC M45 are given. Yang and Li
(Yang et al. 2008) experimentally measured the single crack-bridging \( \sigma-\delta \) relation to verify micromechanics-based model. As shown in Fig.2.7, current model is able to accurately capture the behavior of fiber-bridging.

2.2. Fatigue behavior of fiber-reinforced cement-based composites (FRCC)

2.2.1. Stress-fatigue life (S-N curve)

For a wide spectrum of infrastructures, which sustain repeated loads from vehicles, waves, or wind, fatigue durability must be considered in structural design, as fatigue failure is always brittle and catastrophic. In practical structural design, engineers calculate the stress in the structure and use the stress-fatigue life (S-N) curve of the material to predict the service life of the infrastructure. Therefore, it is important to experimentally characterize fatigue behavior, especially the S-N curve of concrete. The study to characterize fatigue behavior of FRC and ECC is reviewed here.


In flexural fatigue tests, the post-crack fiber-bridging leads to additional load resistance in the tensile zone, significantly enhancing the MOR of FRC specimens. As a result, when using the absolute flexural stress to characterize the flexural S-N curve, the flexural fatigue strength of FRC is always remarkably higher than the control group of plain concrete. However, when using the normalized flexural stress ratio (absolute flexural stress over MOR), the increase of normalized fatigue strength is less
significant (Fig 2.8, Lee and Barr 2004). In some studies, enhancing MOR by further increasing the fiber content even reduce the normalized fatigue strength (Johnston and Zemp 1991). It indicates that the enhancement of flexural fatigue strength in FRC is mainly a result of enhanced MOR; the effect of decelerating fatigue strength reduction is less significant.

In these studies of the flexural fatigue behavior of FRC, fibers of different types (steel fibers, polypropylene, polyvinyl alcohol, and glass), different geometry (straight, hooked), and different aspect ratios (from 47 to 100) are investigated. Despite the large number of specimens involved in these tests, controversy still remains in terms of the effect of the parameters on flexural fatigue resistance. For example, Johnston and Zemp (1991) on the basis of fatigue test over 100 specimens concluded that higher fiber aspect ratio led to higher normalized fatigue stress; while Naaman and Hammound (1998) noticed in the test that the fiber aspect ratio of 60 and 100 produced similar normalized fatigue strength. Such disagreement indicates the complicacy involved, as so many factors regarding fiber, matrix, and fiber-matrix strength could affect the fatigue durability of FRC.

ECC is a special group of FRC with ultra-high ductility. With the tensile strain-hardening property, ECC has even higher MOR compared with conventional FRC. As a result, the flexural fatigue strength (in the absolute scale) of ECC is greatly enhanced. Qian (2007) experimentally characterized the fatigue durability of two types of ECC (M45 as the standard and another group with higher ductility) and depicted S-N curves (Fig.2.9). In the comparison with plain concrete of the same grade, both types of ECC extended the fatigue life by several orders. Suthiwarapirak et al. (2004) compared the fatigue behavior of ECC reinforced with PVA fibers (M45) and PE (polyethylene) fibers in their fatigue tests; the flexural fatigue strength was further enhanced in PE-ECC. However, in both studies, ECC did not show slower fatigue strength reduction when characterized by the normalized flexural fatigue stress.
Since ECC is featured with tensile strain-hardening and the ultra-high ductility, it is necessary to check if such behavior is influenced by fatigue degradation. In both (Qian 2007) and (Suthiwarapirak, et al. 2004), an obvious trend of crack number reduction and ductility loss with the decreasing fatigue load level (or longer fatigue degradation) was observed, as shown in Fig.1.4 (PVA-ECC) and Fig.2.10 (PE-ECC). Actually, such fatigue-induced premature failure was also observed in other fiber-reinforced strain-hardening cementitious composites: Muller and Mechtcherine (2014) reported that the crack number decreases with fatigue life, but such effect is not pronounced if limited number of load cycles are applied (Jun and Mechtcherine 2010).

The premature failure represents the fatigue-dependency of ECC on the macro-scale. The reason behind is qualitatively discussed by Suthiwarapirak, et al. (2004): as the fatigue load level decreasing, the ratio of ruptured fibers to pulled-out fibers found on the fracture surface remarkably increased, indicating that on the meso-scale the failure mechanism of fiber-birding shifts from fiber pullout to fiber rupture; on the micro-scale, it is likely that the properties of fiber, matrix, and fiber-matrix interface have been altered by fatigue degradation. Nevertheless, only after such degradation is quantitatively characterized, from the micro-scale backward to the macro-scale, can the fatigue-induced premature failure be clearly explained.

2.2.2. Models for crack propagation under fatigue load

In flexural fatigue tests, the failure of specimen results from the fatigue-induced crack propagation, from the tension side to the compression side. Once the crack length $a$ reaches a critical value $a_f$, the specimen fail in a brittle manner. The fatigue life depends on the original crack length $a_0$, i.e. the crack length at the peak of the first load cycle, the critical crack length $a_f$ and the crack propagation rate $\Delta a/\Delta N$. Given the monotonic uniaxial compressive and tensile $\sigma$-$\delta$ relation, specimen dimension, and loading scheme, $a_0$ can be calculated with an analytical model developed by Maalej and Li (1994). In this model, the post-cracking tensile load capacity of FRC is considered with plane cross-section assumption and the crack length $a$ can be predicted at given monotonic flexural moment. The $a_f$ is defined as a crack length at which $K_{tip-max}$, the
crack-tip-stress intensity factor at the peak of fatigue load, is equal to $K_c$ the fracture toughness of matrix.

As for determining the crack propagation rate $\Delta a/\Delta N$, two different models have been developed for FRC: one using the fracture mechanics approach (Li and Matsumoto 1998, Matsumoto and Li 1999), the other the force equilibrium approach (Zhang et al. 1999). Both models are reviewed and feasibility of modifying them to predict the fatigue life of ECC is discussed here.

In the fracture mechanics-based model, the determination of crack propagation rate $\Delta a/\Delta N$ is explicit. $\Delta a/\Delta N$ can be determined using Paris’ Law (Eqn.2.9)

$$\frac{\Delta a}{\Delta N} = c \cdot (\Delta K_{tip})^m$$

(Eqn.2.9)

where $c$ and $m$ are Paris’ constants depending the matrix property. $\Delta K_{tip}$ is the in-situ crack-tip-stress intensity factor amplitude, which is related to the stress concentration near the crack tip and can be determined using Eqn.2.10.

$$\Delta K_{tip} = \Delta K_a + \Delta K_b$$

(Eqn.2.10)

where $\Delta K_a (>0)$ is intensity factor amplitude due to externally applied load, calculated with Eqn.2.11; while $\Delta K_b (<0)$ is the stress intensity factor amplitude due to crack-bridging by fibers and/or aggregates, calculated with Eqn.2.12.

$$\Delta K_a = 2 \int_{0}^{a} G(x, a, w) \Delta \sigma_a(x) dx$$

(Eqn.2.11)

$$\Delta K_b = -2 \int_{0}^{a} G(x, a, w) \Delta \sigma_b(x) dx$$

(Eqn.2.12)

In Eqn.2.11 and 2.12, it is shown that $\Delta K_a$ and $\Delta K_b$ are essentially the tensile stress amplitude integral along the crack length $a$ (Fig.2.11), and $G(x, a, w)$ is a weight function that represents a unit force contribution to $\Delta K_a$ and $\Delta K_b$ (Cox and Marshall 1991). The tensile stress amplitude $\Delta \sigma_a$ is related to the applied external moment amplitude (Eqn.2.13); and $\Delta \sigma_b$ is the bridging stress provided by fibers and/or
aggregates, which degrades with fatigue loads. After determining \( \Delta K_{tip} \), the number of cycles to fatigue failure \( N_f \) is calculated as

\[
N_f = \int_{a_0}^{a_f} \frac{1}{C(\Delta K_{tip})^m} da
\]  

(Eqn.2.13)

In the force equilibrium-based model, the determination of crack propagation rate \( \Delta a/\Delta N \) is implicit. It is assumed that force and moment equilibrium exist at the flexural cross-section. As the crack-bridging of fibers and aggregates degrade with the fatigue cycles, the tensile force sustained by the crack-bridging decrease. In order to keep the equilibrium, the crack propagates to the compressive side so that more tensile force is established by new fiber-bridging. Iteration of crack-bridging decrease and crack propagation until \( a \) reaches \( a_f \) can determine number of cycles to fatigue failure \( N_f \).

2.2.3. Fiber-bridging degradation under fatigue load

In the aforementioned models to predict fatigue life of FRC, the determination of fiber-bridging degradation under fatigue loads is very important. In the fracture mechanics-based model, Li and Matsumoto (Li and Matsumoto 1998, Matsumoto and Li 1999) establish the fiber-bridging law (stress-crack opening curve) by summing the pullout behavior of single fibers considering the fiber distribution variation. In the single-fiber pullout model, the interfacial frictional bond \( \tau \) is the only dominant factor, and it decrease with fatigue load cycles. Such an approach to characterize fiber-bridging degradation is associated with two limitations: first in the single-fiber pullout modeling, the interfacial chemical bond and the slipping-hardening effect is not considered, as a result it can only be applied to steel, PP, and PE fibers (with negligible interfacial chemical bond), but not to PVA fibers; second, no single-fiber fatigue pullout test have been conducted to characterize the degradation of fiber-matrix interface, and the degradation rate in the single-fiber modeling is empirical. In the force equilibrium-based model, Zhang et al. (1999) experimentally measured the degradation of stress-crack opening curve in the notched cube specimen under tensile fatigue loads, and use the experimental results to characterize the fiber-bridging degradation. However, such
meso-scale test must be conducted when the mix design is changed, which makes such an approach very tedious.

2.3. Self-healing behavior of cement-based composites

Cracks, generated from restrained shrinkage, excessive loading, harsh environmental exposure, or design/construction errors, are almost inevitable to cement-based construction material like concrete. Crack propagation under fatigue load is the primary deterioration mechanism for many infrastructures; the maintenance and repair work leads to great financial cost and environmental impact. Currently, ‘self-healing concrete’ is emerging as a novel technique to realize the concept of ‘maintenance-free infrastructure’. In this section, studies on self-healing behavior of cement-based composites are reviewed, with the focus on self-healing of ECC.

2.3.1. Self-healing mechanisms

In the presence of water, cement-based composites are naturally capable of forming new substance in the crack, which not only seals the crack but also restores the mechanical performance. The new substance is referred as self-healing product. The possible sources of self-healing products are discussed in (Wu et al. 2012) and listed as following (Fig.2.12):

a. Formation of calcium carbonate (CaCO$_3$) through carbonation of free Ca$^{2+}$ leached out from matrix;

b. Loose concrete debris resulting from cracking;

c. Continued hydration of unreacted cement or cementitious materials;

d. Expansion of the hydrated cementitious matrix, such as swelling of C-S-H.

Among these four mechanisms, debris blocking and matrix expansion are only effective in sealing cracks of below 10 µm (Edvardsen 1999). As the crack width in concrete can be normally well above 10 µm, it is believed that CaCO$_3$ formation and continued hydration are the main sources of self-healing products. Characterization
techniques such as X-ray diffraction (XRD), energy-dispersive X-ray spectroscopy (EDX), thermal gravimetric analysis, and Fourier transform infrared spectroscopy (FTIR) have been applied to determine the composition of self-healing products found in the cracks (Yang et al. 2009, Kan et al. 2010, Yang et al. 2011, Kan and Shi 2012, Huang et al. 2013, Huang and Ye 2014), the results indicate that the main composition of self-healing products is CaCO$_3$, while the presence of C-S-H, the product of continued hydration is also noticed especially for healing at early-age without CO$_2$.

2.3.2. Approaches to promote self-healing

Feasibility study on self-healing concrete show that self-healing can potentially improve the durability of infrastructures in aggressive environment: it reduces the water permeability and chloride diffusion of concrete, enhancing the mechanical strength of the material (Jacobsen et al. 1996, Jacobsen and Sellevold 1996, Edvardsen 1999, Reinhardt and Jooss 2003). Unfortunately, such self-healing is often not observed in the field as the typical concrete crack could be as wide as several hundred microns, which not only demands great amount of self-healing products, but also lowers in-situ pH of crack and inhibits the formation of CaCO$_3$.

For the last 15 years, various engineering approaches have been developed in attempt to promote self-healing in concrete. These approaches are categorized into five groups, namely chemical encapsulation, bacteria additive, mineral admixtures, glass tubing, and self-controlled tight crack width (Fig.2.13) (Li and Herbert 2012).

Healing-agent encapsulation (Fig.2.13a) approach utilizes self-healing chemical agents contained in microcapsules that are uniformly dispersed in concrete matrix. When the propagating crack hits the pervasive microcapsules, the chemical agents are released to fill the crack and/or bond the crack faces. The encapsulated chemical agents must be flowable and the microcapsule must be able to sustain the high-pH environment in cement matrix. For example, Huang and Ye (2011) used sodium silicate solution in wax capsules, and the self-healing delivered good mechanical recovery; Yang et al. (2011) used silica gel microsphere to capsule the oil-phase methy methacrylate
monomer and triethylborane, greatly improving self-healing-induced compressive fatigue performance of concrete. Besides chemical agents, water itself can be stored in superabsorbent polymers (SAP) to promote internal curing, which also produce self-healing products (Mignon et al. 2015).

In the bacteria additive approach (Fig.2.13b), the formation of CaCO₃ in cracks is greatly promoted by bacteria. The bacteria spores, when encountering the crack, would resume the metabolism and creates local alkaline environment in the favor of CaCO₃ formation; they also serve as nucleation sites of CaCO₃ crystal precipitation (De Muynck et al. 2010). The bacteria selected for this approach must be protected from the high alkalinity of cement matrix and the internal compressive pressure as the microstructure continuously densifies (Jonkers et al. 2010, Van Tittelboom et al. 2010); and nutrient must be provided for bacteria growth and metabolism. Wiktor and Jonkers (Wiktor and Jonkers 2011) embedded Bacillus alkanitrilicus spores into porous expansive clay to protect it from the internal pressure, achieve long bacteria preservation; Wang et al. investigated the effective of different microcapsules, like silica-gel (Wang et al. 2012), diatomaceous earth (Wang et al. 2012), and hydrogel (Wang et al. 2014) to protect Bacillus sphaericus spores, and it is shown that when appropriate protection is provided, bacteria can induce complete sealing of cracks as wide as several hundreds of microns. Xu and Wu (2014) investigated the effect of bacteria-induced self-healing on mechanical properties and reported that the recovery of flexural strength was limited, despite robust sealing of cracks was observed.

Some specific mineral admixtures, when exposed by the crack to water, can expand and reduce the crack, promoting the self-healing behavior of concrete (Fig.2.13c). Ahn and Kishi (2010) experimentally confirm that relatively large cracks (150 µm) can be healed by adding calcium sulfoaluminate-based expansive admixtures (CSA) or swelling geo-materials (silicon dioxide, sodium aluminum silicate hydroxide, and montmorillonite clay). Sisomphon et al. (2012) experimentally investigated the effective of combined CSA and crystalline admixtures (a synthesized ternary blend of
hauyen, anhydrite, and free lime) and reported that concrete crack up to 400 µm can be completely closed.

Glass tubing (Fig. 2.13d) may be considered as a variant of chemical encapsulation as an alternative form of healing agent delivery approach. Glass tube is able to carry a relatively large amount of healing agent compared with microcapsules. Various low-viscous healing agents, both inorganic like methy methacrylate (Dry and McMillan 1996), ethyl cyanoacrylate (Li et al. 1998, Joseph et al. 2010), polyurethane with an accelerator (Van Tittelboom et al. 2011) and organic like bacteria (Wang et al. 2012), have been selected for the approach. The self-healing efficiency is verified by the recovery of mechanical properties like stiffness and strength, and a reduction of water permeation of healed concrete.

Self-controlled tight crack width (Fig. 2.13e) utilizes the intrinsic natural tendencies of CaCO₃ formation and continued hydration of cementitious materials. This approach only work if the cracks wide is small (below 100 µm), which can be realized with ECC. Yang et al. (2009) investigated the self-healing behavior of ECC M45 under different healing conditions. Under the wet-dry healing conditioning, healed ECC M45 can achieve 100% recovery of tensile stiffness, tensile strength, and water permeability. Yamamoto et al. (2010) found that there is regain in stiffness for ECC samples exposed to multiple damage and re-healing cycles, which indicates self-healing of ECC is potentially repeatable. More detailed literature review on self-healing of ECC is given in Subsection 2.3.4, before which robustness of these five self-healing approaches are evaluated in Subsection 2.3.3.

2.3.3. Robust self-healing criteria

In order to evaluate the self-healing robustness of cement-based composites, Li and Herbert raised six criteria for robust self-healing, namely shelf life, pervasiveness, quality, reliability, versatility, and repeatability, as summarized in Table 2.2 (Li and Herbert 2012). As the literature so far fails to provide adequate data to properly assess reliability, the five approaches are evaluated against the other five criteria, as
summarized in Table 2.3. It is seen that the first four approaches have limitation in at least one aspect. The chemical encapsulation approach can satisfy almost all the criteria except the shelf life remain questioned due to the chemical stability of the encapsulation. The bacteria additive approach is highly effective in crack sealing but the mechanical property recovery is minimal, therefore the quality criterion is not fully satisfied; besides, the viability of bacteria through the long service life of infrastructures remain unclear and questioned. The mineral admixtures approach is also not satisfactory for the quality criterion: very large crack can be sealed with expansive minerals but the mechanical recovery is limited. The glass tubing approach result in 100% mechanical and transport recovery; however it fails to satisfy the criterion of pervasiveness and repeatability. The self-controlled tight crack width approach based on the development of ECC, as shown in Table 2.3, shows good potential of meeting all six robustness criteria.

2.3.4. Self-healing of ECC
The self-healing property of ECC is associated with its self-controlled tight crack width, normally below 100 µm. Previous studies have shown that cracks in ECC could be healed in the presence of water (Yang et al. 2009): cracks are filled mainly by the CaCO$_3$ crystals generated by the carbonation of leached out Ca$^{2+}$, while the C-S-H by continued hydration is also found in the healed crack (Kan and Shi 2012). Self-healing of ECC has been observed in laboratory conditions with various temperature, alkalinity, and chloride concentrations as well as natural environment (Şahmaran and Li 2008, Yang et al. 2009, Li and Li 2011, Yang et al. 2011, Herbert and Li 2012, Zhu et al. 2012).

The degree of ECC healing is highly affected by the crack width. Yang et al. (2009) showed that for ECC M45 the maximum allowable crack width to engage complete mechanical recovery was around 50 µm, beyond which the mechanical recovery keeps decreasing with the increasing crack width (partial healing), until 150 µm where no noticeable mechanical recovery is observed (Fig.2.14). Curing condition is found to be an important factor that influences the healing efficiency: wet-dry cycles (one day in
water followed by one day in air) leads to mechanical and water permeability recovery than constant water curing (Yang et al. 2009, Yang et al. 2011).

2.4. Problem and knowledge gaps
The problems and the knowledge gaps within the research scope of this thesis (Fig.1.5) are summarized here based on the literature review.

Fatigue failure of fiber-reinforced cement-based composites like ECC is the result of fatigue-induced crack propagation accompanied with fiber-bridging degradation. The knowledge gap to a systematic frame for ECC fatigue lies in the understanding of fiber-bridging degradation. While a few experimental studies attempted to investigate fiber-bridging degradation on the meso-scale, holistic understanding of such degradation must be based on study in the micro-scale, i.e. single-fiber pullout behavior under fatigue loads. As a result, the fatigue-induced degradation of fiber and fiber-matrix interface must be experimentally characterized. The characterized fatigue-dependency needs to be integrated into the micromechanics-based fiber-bridging model. With this modified model, the ductility loss of fatigue-failed ECC can be understood and mitigated, and the flexural fatigue life of ECC can be predicted and extended.

In fiber-reinforced cement composites like ECC, MOR depends on the uniaxial constitutive law, i.e. tensile and compressive stress-strain (σ-δ) relation of the material. The effect of uniaxial tensile and compressive σ-δ relation on MOR must be understood first, so that the desirable material properties can determined. Then the desirable properties must be developed with locally available materials. In standard ECC, coal fly ash and fine micro-silica sand are two important ingredients that affect the tensile σ-δ relation. In Singapore, however, the access to coal fly ash and fine micro-silica sand is limited. As such, granulated ground blast-furnace slag (GGBS) and river sand are proposed as possible replacement. The effects of GGBS and river sand on ECC tensile σ-δ relation need to be experimentally investigated before developing local ECC with desirable material properties.
Self-healing capability of ECC is confirmed with preliminary studies; however, it has not yet been considered robust self-healing. While the effects of crack width and healing conditions on ECC self-healing robustness have been extensively studied, few study focus on the effect of chemical composition. As the healing products, the CaCO$_3$ and the continued hydration products are related to the cementitious binder in ECC, the content of supplementary cementitious material (GGBS for local ECC) must be studied. The effect of self-healing on mechanical property recovery has been studied, but no study investigates the feasibility of applying self-healing as a technical method to extend the fatigue life of ECC. As a result, after determining the desirable GGBS content level for self-healing, the effect of self-healing on fatigue performance of ECC needs to be experimentally verified.
References


Table 2.1 Micro-scale parameters used as model input

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Fiber parameters</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Diameter $d_f$ (µm)</td>
<td>39¹</td>
<td>provided by the fiber manufacturer</td>
</tr>
<tr>
<td>Length $L_f$ (mm)</td>
<td>12¹</td>
<td>determined by single-fiber pullout test (Kanda and Li 1998, Redon et al. 2001)</td>
</tr>
<tr>
<td>Young’s modulus $E_f$ (GPa)</td>
<td>22¹</td>
<td>suggested by Wu (2001)</td>
</tr>
<tr>
<td>In-situ strength $\sigma_{fu}$ (MPa)</td>
<td>1060²</td>
<td>measured by splitting test of ECC M45 matrix</td>
</tr>
<tr>
<td><strong>Interface parameters</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Chemical bond $G_d$ (J/m²)</td>
<td>1.08²</td>
<td></td>
</tr>
<tr>
<td>Frictional bond $\tau_0$ (MPa)</td>
<td>1.31²</td>
<td></td>
</tr>
<tr>
<td>Slip-hardening coefficient $\beta$</td>
<td>1.08²</td>
<td></td>
</tr>
<tr>
<td>Snubbing coefficient $f$</td>
<td>0.20³</td>
<td>suggested by (Wu 2001)</td>
</tr>
<tr>
<td>Strength reduction coefficient $f'$</td>
<td>0.33²</td>
<td></td>
</tr>
<tr>
<td>Cook-Gorden coefficient $\alpha$ (µm)</td>
<td>78⁴</td>
<td>determined based on the measurement of micro-spalling size</td>
</tr>
<tr>
<td><strong>Matrix</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Young’s modulus $E_m$ (GPa)</td>
<td>20⁵</td>
<td>measured by splitting test of ECC M45 matrix</td>
</tr>
<tr>
<td>Micro-spalling coefficient $k$</td>
<td>500⁶</td>
<td>suggested by (Li et al. 1993)</td>
</tr>
</tbody>
</table>
Table 2.2 The six criteria to evaluate robustness of self-healing (Li and Herbert 2012)

<table>
<thead>
<tr>
<th>Criterion</th>
<th>Definition and scope of the criterion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shelf life</td>
<td>Self-healing functionality must be valid in a shelf life of 50-100 years, which is common for infrastructures.</td>
</tr>
<tr>
<td>Pervasiveness</td>
<td>Self-healing functionality must be pervasive in the structures; cracks should be healed regardless of orientation.</td>
</tr>
<tr>
<td>Quality</td>
<td>Ideally, self-healing functionality should lead to complete recovery of both transport properties (permeability, diffusivity, etc.) and mechanical properties (strength, stiffness, potential ductility, etc.) of the structure.</td>
</tr>
<tr>
<td>Reliability</td>
<td>The recovery of transport and mechanical properties should be consistent; large coefficient of variation would suggest lack of consistency.</td>
</tr>
<tr>
<td>Versatility</td>
<td>Self-healing functionality should be valid in various environments, allowing wide range of humidity, temperature, and alkalinity.</td>
</tr>
<tr>
<td>Repeatability</td>
<td>Self-healing functionality should be repeatable as damage of infrastructures is likely to happen repeatedly.</td>
</tr>
<tr>
<td></td>
<td>Healing-agent encapsulation</td>
</tr>
<tr>
<td>----------------</td>
<td>-----------------------------</td>
</tr>
<tr>
<td><strong>Shelf life</strong></td>
<td>Remain questioned due to the chemical stability</td>
</tr>
<tr>
<td><strong>Pervasiveness</strong></td>
<td>Yes, capsules could be uniformly dispersed in concrete</td>
</tr>
<tr>
<td><strong>Quality</strong></td>
<td>Regains mechanical properties to an extent; self-sealing has not been investigated</td>
</tr>
<tr>
<td><strong>Reliability</strong></td>
<td>N.A.</td>
</tr>
<tr>
<td><strong>Versatile</strong></td>
<td>Yes, healing mechanism is independent of external environment</td>
</tr>
<tr>
<td>Repeatability</td>
<td>Healing-agent encapsulation</td>
</tr>
<tr>
<td>---------------</td>
<td>-----------------------------</td>
</tr>
<tr>
<td></td>
<td>No studies have been done with more than one healing cycle; likely not repeatable</td>
</tr>
</tbody>
</table>
Fig. 2.1 Uniaxial tensile stress-strain-crack width relationship of ECC M45 (Yang et al. 2009)
Fig.2.2 Hysteresis loops comparison of column members under fully reversed cyclic loading for: (a) R/C with stirrups and (b) R/ECC without stirrups. Higher energy absorption and more stable hysteresis loop are observed in the R/ECC members, despite the total elimination of shear stirrups (Fischer and Li 2002).
Fig. 2.3 Scale linking in ECC

![Diagram](image)

$$ J_b' = \sigma_0 \delta_0 - \int_0^{\delta_0} \sigma(\delta) d\delta $$

$$ \sigma_{ss} \delta_{ss} - \int_0^{\delta_{ss}} \sigma(\delta) d\delta = J_{\text{tip}} $$

Fig. 2.4 Typical $\sigma(\delta)$ curve for tensile strain-hardening composite; hatched area represents complimentary energy $J_b'$; shaded rea represent crack tip toughness $J_{\text{tip}}$
Fig. 2.5 Illustration of typical tensile load-displacement ($P-u$) relation in single-fiber pullout test.
Fig. 2.6 Flow chart of the numerical procedure for computing (Yang et al. 2008)
Fig. 2.7 Comparison of ECC fiber-bridging $\sigma$-$\delta$ curve obtained from uniaxial tensile test of notched specimen and that predicted from the micromechanics-based model: (a) $V_f=0.1\%$; (b) $V_f=0.5\%$ (Yang et al. 2008)
Fig. 2.8 Reviewing the normalized flexural stress-fatigue life of plain concrete and steel fiber-reinforced concrete: (a) plain concrete; (b) fiber content = 0.5% by volume; (c) fiber content = 1.0% by volume; (d) comparison of the fitting curve by regression (Lee and Barr 2004)
Fig. 2.9 Comparison of the S-N curve of ECC and conventional concrete; mix design ECC 1 (M45) has relatively lower ductility (2.5%) and higher compressive strength (46.0MPa); ECC 2 has relatively higher ductility (3.7%) and lower compressive strength (37.5MPa); Concrete 1 and 2 have similar compressive strength with ECC 1 and 2 respectively (Qian 2007)
Fig. 2.10 Cracking pattern of failed PE-ECC by flexural test: (a) monotonic load; (b) fatigue load level = 0.9; (c) fatigue load level = 0.6 (Suthiwarapirak et al. 2004)
Fig. 2.11 Illustration of the contributing component of crack-tip-stress intensity factor

(Li and Matsumoto 1998)

Fig. 2.12 Possible mechanism for self-healing mechanisms: CaCO$_3$ formation (a), blocking by debris (b); continued hydration (c); expansion of cementitious matrix (d)

(Wu et al. 2012)
Fig. 2.13 Schematic illustration of (a) chemical encapsulation; (b) bacteria additive; (c) mineral admixtures; (d) glass tubing; (e) self-controlled tight crack width (Li and Herbert 2012)
Fig. 2.13 (Continued) Schematic illustration of (a) chemical encapsulation; (b) bacteria additive; (c) mineral admixtures; (d) glass tubing; (e) self-controlled tight crack width

(Li and Herbert 2012)
Fig. 2.14 Resonant frequency (RF) ratio as a function of crack width (Yang et al. 2009)
CHAPTER 3 EXPERIMENTAL INVESTIGATION ON THE SOURCES OF
ECC FATIGUE-DEPENDENCY

Fatigue failure is always a great concern for concrete structures subject to repeated loading, such as pavement and bridge deck. Fatigue resistance of these structures depends on the structural design, construction quality, and especially the fatigue resistance of concrete. For plain concrete, the fatigue life depends on its matrix strength: fatigue crack propagation is slower in the concrete of higher grade; for fiber reinforced cementitious composites like ECC, the fatigue crack propagation is also resisted by the fiber-bridging across the crack. It is therefore important to understand the degradation of fiber-bridging under fatigue loading, in order to characterize and improve the fatigue resistance.

Macro-scale fatigue behavior of ECC have been conducted by different researchers (Suthiwarapirak et al. 2004, Qian 2007) and reviewed in Chapter 2. Despite the different versions of ECC were used in these tests, similar phenomena were observed: the specimens experienced premature failure under fatigue loading, the damage pattern of ECC failed by fatigue loading was distinct from that failed by monotonic loading. Specifically, the crack number was reduced and crack spacing was enlarged with the decreasing fatigue load level. Such premature failure on macro-scale (cm), resulting in lower ductility of ECC, suggests the necessity of exploring the fatigue degradation on meso-scale (mm) and micro-scale (μm).

On the meso-scale, the constitutive fiber-bridging behavior, i.e. tensile stress-crack opening (σ-δ) curve, is possibly changed by fatigue loading; on the micro-scale (μm), the fiber, matrix, and fiber-matrix interface properties may be fatigue-sensitive, which essentially governs fatigue dependency of σ-δ curve in the meso-scale. While some studies discussed the fatigue-induced fiber-bridging degradation on meso-scale (Matsumoto and Li 1999, Takashi 2008, Matsumoto et al. 2010), the fatigue dependency of ECC must be understood fundamentally on the micro-scale. In the established micromechanics-based model, there are 13 micro-mechanical parameters that quantify properties of fiber, matrix and fiber-matrix interface (Yang et al. 2008). It
is important to identify those dependent of fatigue loading (loading level and cycles) and to characterize their fatigue dependency.

In this chapter, a serial single-fiber pullout test was conducted to investigate the fatigue degradation of ECC on the micro-scale. Specifically, the effect of fatigue loading on in-situ fiber strength, fiber strength reduction coefficient, fiber debonding stage and fiber pull-out stage are characterized and discussed.

3.1. Effect of fatigue on (in-situ) strength of PVA fiber

3.1.1. Specimen preparation

In current section, single polyvinyl alcohol (PVA) fiber embedded in mortar matrix was tested under monotonic and/or cyclic tensile loading. The properties of PVA fibers are given in Table 3.1. The mortar matrix is made of Type I Portland cement (CEM I 52.5N), ground-granulated blast furnace slag (GGBS), and sieved fine river sands (<0.6 mm in particle size), superplasticizer, and water. Instead of coal fly ash that was used to replace Portland cement in the standard ECC M45, GGBS was used in current study as the access to fly ash was limited locally; sieved river sands were used to replace the micro silica sands in M45 for the same reason. The same GGBS and river sand were used in the ECC tailoring in Chapters 5 and 6. It has been reported that slag and river sand can produce robust strain-hardening behavior (Sahmaran, et al. 2009, Zhou, et al. 2010). The mix proportion design of the mortar matrix is given in Table 3.2.

Specimen preparation followed Katz and Li’s work (1996): long PVA fiber was cut into around 150 mm in length and cast into the fresh mortar, as shown in Fig.3.1. After one day the hardened matrix was demolded together with fiber and cut into specimens embedded with a single PVA fiber, which are referred as ‘embedded fiber’ in this section. In order to determine the in-situ fiber strength, a relatively large embedment length ($L_e$>7 mm) was used to ensure the fiber being ruptured.
3.1.2. Mechanical test

Monotonic or fatigue uniaxial tensile tests were conducted on single naked PVA fibers (without being embedded into the matrix) or embedded fiber ($L_e > 7$ mm) using an electro-dynamic universal testing machine for studying the fatigue response of naked fiber strength or in-situ strength of embedded fiber, as well as the fiber strength reduction coefficient of embedded fiber. A 10N dynamic load cell was included to provide closed-loop load control for fatigue test. The test set-up details are shown in Fig.3.2.

For the naked PVA fiber, it was glued to two metal plates mounted on the actuator and the load cell respectively; for the embedded fiber, the mortar matrix was glued to a T-plate mounted on the load cell while the fiber was glued to metal plate. For both specimens, the free length of the fiber was adjusted to about 5 mm. The testing program is summarized in Table 3.3: for each group of specimen (naked fiber, embedded fiber $0^\circ$, and embedded fiber $30^\circ$), monotonic test as well as fatigue tests of different loading level were conducted to plot the S-N curve.

In monotonic test, the loading rate is displacement-controlled at 0.03 mm/min. In fatigue test, as shown in Fig.3.3, the load was first increased to the mean of the maximum and minimum load in the ramping phase, followed with a cyclic phase where load was controlled in a sinusoidal wave. For all specimens, the minimum load $P_{min}$ was fixed at 0.20 N, while the maximum load $P_{max}$ was varied from 1.0 N to 1.4 N for naked fibers, and from 0.70 N to 0.90 N for embedded fibers.

3.1.3. Microstructure characterization

After the test, some of the fibers were observed using field-emission scanning electron microscope (JSM-7600F). The morphology of the fiber surface was analyzed to understand the mechanisms of fatigue induced fiber deterioration.
3.1.4. Results and discussion

The results of monotonic and fatigue test on single naked fibers and embedded fibers ($L_e > 7$ mm) with different orientations are illustrated in Fig.3.4, where the fitted curve shows the relation between (in-situ) fiber strength and fatigue life.

**Monotonic Test**

It is observed in Fig.3.4 that the in-site strength of embedded fiber, is remarkably lower (over 30%) than that of naked fibers. Such monotonic strength reduction due to embedment of fiber was previously reported (Kanda and Li 1998, Redon et al. 2001, Li et al. 2002). Kanda and Li (1998) and Li et al. (2002) used the hypothesis of surface abrasion of debonded fiber to explain such strength reduction, and for current study the surface abrasion is shown in Fig.3.5. As can be seen, the failure of embedded fibers is dominated by fiber rupture but located somewhere inside the matrix several hundred micron ($L_e = 7$ mm) away from the free surface, which indicates that the PVA fiber is weakened when embedded into a cement-based matrix. It can be seen that the fiber tip was sharpened, which indicates the surface layer of the fiber was damaged by the rough matrix. In contrast, the failure of naked fibers is fiber rupture located somewhere in the free length of the fiber. The tip of ruptured naked fiber was rather blunt, as shown in Fig.3.6a.

It should be noted that Redon et al. (2001) observed severe surface abrasion in fully debonded fibers due to small fiber embedment length of 0.8 mm and relatively large fiber slippage; current study complements Redon’s work by showing that surface abrasion, though not as severe as that in their study, also happened to partially debonded fibers with large embedment length.

It is also observed in Fig.3.4 that the in-situ fiber strength of $30^\circ$-oriented embedded fiber specimen is lower than that of $0^\circ$-oriented specimen due to combined bending and tension introduced to the bended fiber in which high local tensile stress is generated on the tension side of the bended fiber (Kanda and Li 1998, Yang et al. 2008).
Fatigue Test

Fig. 3.4 shows that in-situ strength of naked fiber decreases with increasing load cycles, and such decrease is shown as a bi-linear curve: slower decrease in the low-cycle range (<10,000 times) but drops faster in the high-cycle range (> 10,000 times). Trilinear fatigue load-life curve (flat-steep-flat) was well documented for polymeric materials (Suresh 1998), and the bi-linear curve for current PVA fiber is believed to reflect the first two regions (flat-steep). In the high-cycle range (region II) with low-stress level, craze of polymer formed only at the tip of microscopic crack (the high-stress zone), and cyclic loads propagated the crack by ‘damaging’ the craze. In this range, increase of stress level would greatly increase the crack propagation rate and reduce the fatigue life. In the low-cycle range (region I), however, craze formed and spread over the fiber section at the very first load cycles due to high-stress level. As a result, the curve in this range was lower than the extrapolation of the curve in region II, i.e. flatter slope (Suresh 1998, Courtney 2005).

FE-SEM images confirm different failure modes for the two regions: blunt tips are observed for fibers failed in low-cycle range (Fig. 3.6a and b), while sharp tips are observed for fibers failed in high-cycle range (Fig. 3.6c and d). Such difference may be explained as: at higher stress level, craze formation dominated, and as a matter of fact it propagates normal to the tensile direction; at lower stress level, microscopic crack propagation could result from a combination of craze and shear slip band, which may lead to inclined crack propagation and sharp fractured tip (Courtney 2005).

Fig. 3.4 shows that the load capacity of both 0° and 30°-oriented embedded fiber specimens decreases linearly with load cycles, which indicates that fatigue load cycles further reduced the in-situ strength of embedded fibers. The deterioration mechanisms of embedded fiber, however, must be different than that of the naked fiber since the load level is much below than the required stress to trigger above mentioned fatigue mechanisms in both regions.
Fig. 3.7 illustrates the failure mode of embedded fibers subject to the fatigue loads. The fiber is sharpened and damage severely which is somewhat similar to the failure mode that observed in the monotonic load (Fig. 3.5). This suggests that the fiber damage mechanisms are similar in both monotonic and fatigue loading for the embedded fibers. However, under fatigue loads, the damage must have accumulated with increasing load cycles, which result in fiber rupture at much lower load level as shown in Fig. 3.4.

Despite the similar morphology of ruptured embedded fibers subject to monotonic or fatigue, subtle differences should be noticed. It was observed that the surface of monotonically ruptured embedded fiber is smooth, while the surface of fatigue ruptured embedded fiber is rough (Fig. 3.5b and 3.7b); at higher magnification (10,000×), fatigue striations are observed on the surface of fatigue failed fiber (Fig. 3.8c) but not monotonically failed fiber (Fig. 3.5c). Such observations may be used as clue to unveil the underlying fiber damage accumulation under fatigue loads.

In Fig. 3.4, the slope of S-N curves of 0° and 30°-oriented fibers are almost the same. Specifically, Kanda et al. (1998) reported that under monotonic load, the in-situ fiber strength ratio of 30°-oriented to 0°-oriented specimen is 0.85; in current study, the ratio under fatigue loads ranges from 0.83-0.87. It indicates the fatigue loads do not alter the mechanism of further fiber strength reduction due to bended fiber.

3.2. Effects of fatigue on fiber-matrix interfacial properties

3.2.1. Specimen preparation

In current section, single polyvinyl alcohol (PVA) fiber embedded in mortar matrix was tested under monotonic and/or cyclic tensile loading. The same mix design in Table 3.2 was used for the mortar matrix. Specimen preparation followed the same procedure in the last section (Katz and Li 1996): long PVA fiber was cut cast into the fresh mortar. After one day the hardened matrix was demolded together with fiber and cut into specimens embedded with a single PVA fiber, which are referred as ‘pullout specimen’ in this section. In order to investigate the effect of fatigue loading on fiber-
matrix interfacial properties, a relatively small embedment length ($L_e \sim 1.5$ mm) was used to obtain relatively complete pullout-curve.

3.2.2. Mechanical test

Monotonic or fatigue uniaxial tensile tests were conducted on pullout specimens using an electro-dynamic universal testing machine for studying the fatigue response of the fiber. A 10N dynamic load cell was included to provide closed-loop load control for fatigue test. The test set-up details are shown in Fig.3.2. The pullout specimen was glued to a T-plate mounted on the load cell and the fiber was glued to metal plate. The free length of the fiber was adjusted to about 2.5 mm.

A serial test composed of fatigue preloading and monotonic reloading was conducted to pullout specimens ($0^\circ$, $L_e \sim 1.5$ mm) to investigate the effect of fatigue loading on fiber-matrix interfacial properties. The fatigue preloading consists of a ramping phase and a cyclic phase, the detail of which is illustrated in Fig.3.3. Followed by the monotonic reloading which is displacement controlled at 0.03 mm/min. The fatigue preloading is meant to introduce fatigue deterioration at different fatigue load level and cycle to the pullout specimen while the monotonic reloading is used to characterize and determine the fiber/matrix interface properties after the fatigue. In additional to the testing groups, a control group of pullout specimens were also included. For the control group, no fatigue preloading was applied before the monotonic loading, so that the effect of fatigue preloading can be examined by comparing the testing groups and the control group.

The single-fiber testing program is summarized in Table 3.4. For the control group the single fiber is monotonically pulled out: crack initially occurs at the fiber-matrix interface near the surface; as the load increases, the initial crack would ‘tunnel’ from the surface into the deep part, until the fiber is fully debonded from matrix. The event of full-debonding divides the fiber pullout into the preceding debonding stage and the following slippage stage, which have distinct fiber-matrix interface properties (illustrated in Fig.2.5, refer to Chapter 2 for the details). As a result, at relatively low
fatigue load level \((P_{\text{max}}=0.35-0.40 \text{ N})\), fiber may not experience complete debonding even at the end of the cyclic phase. The results therefore can be used to understand the effect of fatigue in the debonding stage of single fiber pullout. At the intermedium load level \((P_{\text{max}}=0.40-0.50 \text{ N})\), the complete debonding may occur during the cyclic phase, the result of which can be used to determine the fiber debonding rate under fatigue loading, i.e. tunneling crack propagation during fatigue. At relatively high fatigue load level \((P_{\text{max}}=0.50-0.70 \text{ N})\) when \(P_{\text{max}}\) exceeds the debonding force \(P_a\), the fiber would fully debond from the matrix during the ramping stage, therefore the results can be used to understand the effect of fatigue in the slippage stage of single fiber pullout.

3.2.3. Results and discussion
Fatigue-induced fiber debonding

It is of great interest to answer if the crack would propagate under fatigue loads. If this assumption is true, it is necessary to determine when the full-debonding occurred so that investigating the effects of fatigue loads in the debonding and the slippage stages separately becomes possible. For control group, the fiber displacement in the debonding stage never exceeded 60 \(\mu\text{m}\); while the fiber displacement in the slippage stage could be as large as several millimeters. It suggests that the effect of fatigue loading after full-debonding on fiber-matrix interface can be very different from that before full-debonding, and that separate investigations for both stages are necessary.

In the displacement-controlled monotonic single fiber pullout tests, the complete fiber debonding from matrix is accompanied with a sudden load drop. In the current load-controlled fatigue single fiber pullout tests, a displacement leap may indicate full-debonding. As a result, the displacement history in fatigue preloading tests was carefully analyzed.

Fig.3.9 shows the displacement history in the ramping phase and cyclic phase at different load levels. It is shown that in the cyclic phase, the fiber displacement increased with load cycles, indicating degradation of the fiber-matrix interface. At relatively low fatigue load level \((P_{\text{max}}=0.35-0.40\text{N})\), the fatigue-induced debonding
crack propagation was very slow, and the full-debonding did not take place even at the end of the fatigue preloading test as shown in Fig.3.9a. At the intermedium fatigue load level (Fig.3.9b, max. load from 0.40 to 0.50N), the displacement leap, which represents full-debonding of the fiber, happens in the cyclic phase, indicating the fatigue loads indeed propagate the fiber debonding crack and eventually lead to full-debonding. At relatively high load level ($P_{\text{max}}>$0.50-0.70N), the full-debonding of fiber, shown by the displacement leap, took place during the ramping phase of the fatigue preloading as shown in Fig.3.9c.

Fig.3.10a shows the relation between fatigue-induced crack propagation $\Delta a$ and number of cycles $N$ to full-debonding under three load levels ($P_{\text{max}}$=0.40N, 0.45N, and 0.50N), and three linear lines are drawn to fit the data points, respective. $\Delta a$ is calculated based on Eqn.3.1.

$$\Delta a = L_e - a_0 \quad \text{(Eqn.3.1)}$$

where $L_e$ is the fiber embedment length, and $a_0$ is the length of crack propagation at the peak of the first load cycle. Given the specific load level applied to pullout specimen, $a_0$ can be calculated. The details for calculating $a_0$ can be found in (Lin et al. 1999).

Fig.3.10a shows that $\Delta a$ increases with the $N$, i.e. fatigue-induced fiber debonding. In Fig.3.10b shows that the debonding rate, i.e. the slope of the fitting $\Delta a$-$N$ curve, enhances with the increasing load level, which shows a trend of power function and follows the form of Paris’ Law as Eqn.3.2 (Pook and Frost 1973).

$$\Delta a/N = C \cdot P_{\text{max}}^M \quad \text{(Eqn.3.2)}$$

**Effect of fatigue on the fiber-matrix interface (debonding stage)**

At relatively low fatigue load level ($P_{\text{max}}$=0.35-0.40N) the full-debonding did not take place even at the end of the fatigue preloading test. The effect of fatigue loads on the fiber-matrix interfacial properties of the debonding stage is presented here.
Fig. 3.11 shows typical single-fiber pullout behavior of fatigue-degraded specimen (red) and the control specimen (blue). As full-debonding of PVA fiber from matrix was not reached in the fatigue preloading test, the sudden load drop from $P_a$ to $P_b$ could still be observed in Fig. 3.11.

The effects of number of load cycles and load level in fatigue tests on $P_a$, $P_b$, $P_a-P_b$, and debonding stiffness $k_d$, i.e. slope of the $P-u$ curve in the debonding stage, are shown in Table 3.6. It can be seen that $P_a$, $P_b$, and $k_d$ increase with load cycles and load level, while $P_a-P_b$ remains constant. Before the moment of full-debonding, the external load $P_a$ is resisted by the chemical bond ($G_d$, J/m$^2$) at the undebonded fiber-matrix interface, as well as the frictional bond of the debonding stage ($\tau_d$, MPa) between the debonded fiber and matrix. After full-debondding, the load $P_b$ is resisted by frictional bond only (Redon et al. 2001). $G_d$ and $\tau_d$ can be calculated with the following equations (Chapter 2 for more details):

$$G_d = \frac{2(P_a-P_b)^2}{\pi^2 E f d_f^2}$$  \hspace{1cm} (Eqn.3.3)

$$\tau_d = \frac{P_b}{\pi d_f L_e}$$  \hspace{1cm} (Eqn.3.4)

The effects of number of load cycles $N$ and load level $P_{max}$ in fatigue tests on $G_d$ and $\tau_d$ are shown in Fig. 3.12. The chemical bond is barely affected by fatigue loads. The frictional bond actually get stronger with increasing number of load cycles, which phenomenon is referred as fatigue-induced friction enhancement in debonding stage, i.e. fatigue hardening. The fatigue hardening effect gets stronger at higher load level.

Fig. 3.13 shows the increase of tensile stiffness during the fatigue test. The tensile stiffness value, due to the dynamic effect from the high-frequency fatigue loading, is much higher than monotonic tensile stiffness. It shows that at the beginning (before 10,000 cycles), the tensile stiffness increases with load cycle; but afterwards such increase is mitigated and the trend enters into a flat stage. It indicates that there could be two combating mechanisms that influence the nature of fatigue hardening: first the
debonded fiber surface is abraded and roughened by the reciprocated movement during the fatigue loading (Fig.3.14), resulting increasing interfacial frictional bond; second, the PVA fiber itself is softened by the fatigue loads, as shown in Fig.3.15. For the beginning period, the fiber surface roughening is dominant; but the effect fiber softening becomes stronger and offsets the fatigue hardening effect.

In Fig.3.11, it is also notice that there is residual displacement after the fatigue tests. Before full-debonding, the displacement is mainly contributed by the stretching of the debonded portion of the fiber. It is possible that friction between the roughened fiber surface and the matrix prevents the stretched fiber to relax to its original status.

Effect of fatigue loading on the fiber-matrix interface (slippage stage)

At relatively high load level ($P_{\text{max}} > 0.50-0.70\, \text{N}$), the full-debonding of fiber from matrix took place during the ramping phase of the fatigue preloading. The effect of fatigue loads on the fiber-matrix interfacial properties of the slippage stage is presented here.

Fig.3.16 shows typical single fiber pullout behavior of fatigue degraded specimen (red) and the control specimen (blue). As the fiber is fully debonded during the ramping stage, it shows no load drop before fiber rupture in the red curve, distinct from the specimens without full-debonding (Fig.3.11). The curve rises in a bi-linear trend: steeper before the transition point ($P_t$), followed by with a flatter slope. Probably there are two different mechanisms dominating the interfacial frictional bond between the fiber and matrix: before the transition point there seems to be significant jamming effect due to the loose debris from the debonding stage (Fig.3.14b), which preventing large fiber displacement; once the jamming is overcome at a sufficiently high load level ($P_t$), there will be large fiber displacement (or fiber slippage) and the frictional bond is then governed by the fiber surface roughness only.

The effects of number of load cycles and load level in fatigue tests on the transition load $P_t$ and slippage stiffness $k_{s1}$ and $k_{s2}$, i.e. slope of the $P-u$ curve before and after the transition, are shown in Table 3.7. It can be seen that $P_t$ is always close to the
maximum load experienced in the fatigue preloading, despite the different load cycles. \( k_{s1} \), the slope before the transition point, seems not affected by the fatigue load cycles and fatigue load level; \( k_{s2} \) the slope after the transition point, increases with fatigue load cycles and fatigue load level. Frictional bond in the slippage stage \( \tau_s \) is calculated with the following equation:

\[
\tau_s = \frac{k_{s2}}{\pi(t_\epsilon \beta - d_f)}
\]  
(Eqn.3.5)

The change of \( \tau_s \) with fatigue load cycle and load level is shown in Fig.3.17. The increase of \( k_{s2} \) indicates that the slip-hardening phenomenon during the slippage stage is fatigue sensitive. This fatigue-induced friction enhancement in slippage stage, i.e. *fatigue slip-hardening*, increases with fatigue load cycles and fatigue load level. This probably can be explained by the hypothesis that before the transition point the \( P-u \) behavior is dominated by the jamming of loose debris, which is insensitive to fatigue loads.

### 3.3. Effects of fiber surface treatment on fiber strength and fiber/matrix interface properties under fatigue

Experimental study has been conducted to understand the effects of fatigue degradation on ECC on the micro-scale level in the last section. Specifically, single-PVA-fiber pullout test were conducted to investigate the fatigue-induced changes in PVA fiber properties and fiber-matrix interfacial properties. The test results indicate that the in-situ strength of embedded PVA fiber is remarkably reduced by the fatigue loads; the tensile stiffness of fiber pullout \( P-u \) curve is increased by the fatigue hardening and fatigue slip-hardening effects. These changes would produce negative impact to fiber-bridging properties on the meso-scale, subsequently influencing the ECC ductility on the macro-scale. Specifically, reduced in-situ fiber strength would lead to lower fiber-bridging strength \( \sigma_0 \) (Fig.2.4), fatigue hardening and fatigue slip-hardening would lead to stiffness increase of the fiber-bridging \( \sigma-\delta \) curve (Lin et al. 1999); these changes would decrease the pseudo strain-hardening indices *PSH energy* \( (= J_b / J_{tip} \) ) and *PSH strength* \( (= \sigma_0/\sigma_c \) ), reducing the ECC ductility.
The aforementioned in-situ fiber strength reduction and interfacial hardening are believed to be related to the fiber abrasion by the reciprocated movement during the fatigue load. Oil-treatment on fiber surface can effectively reduce the interfacial bond between the fiber and matrix. Li et al. (2002) investigated the effect of surface oil-coating content on PVA fiber-matrix interfacial properties and concluded that the chemical bond, frictional bond, and slip-hardening coefficient decrease with increasing oil content. It is possible that the reduced interfacial bond can mitigate the fiber abrasion and slow down the fatigue degradation. It is therefore important to investigate the effect of oil-treatment on the in-situ fiber strength and interfacial hardening under fatigue loads.

3.3.1. Specimen preparation
In current section, PVA fiber with surface oil-coating was used in the experimental study. The properties of these fibers are the same with Table 3.1 except the oil had been coated on the surface, which makes up 1.2% mass of the fiber. The fiber surface oil-treatment is completed by the manufacture. The mortar adopts the identical mix design as shown in Table 3.2, with the same raw ingredient source.

The pullout specimen preparation follows the same procedures described in Fig.3.1 except the fiber embedment length was increased as the interfacial bond remarkably reduced by oil-coating. The embedment length \( L_e \) ranged from 1.5 mm to 20 mm for different testing purpose, the details of which will be illustrated together with the mechanical testing program later.

3.3.2. Mechanical test
For PVA fiber without oil coating, the interfacial bond is strong and the fiber slippage is limited; for the oil-coated PVA fiber, the interfacial bond is much reduced and very large fiber slippage can occur before the fiber rupture. Such large slippage leads to very severe fiber surface abrasion and even fibril delamination (Redon et al. 2001), which may greatly affect the in-situ strength of embedded fiber. Therefore for oil-coated fiber, it is important to characterize the relationship between in-situ fiber strength and fiber-
matrix slippage before investigating the effect of fatigue load on in-situ fiber strength. As a result, a serial monotonic fiber pullout tests were conducted to fulfill the purpose. The fiber embedment length was ranged from 1.5 mm to 20 mm. The test set-up is shown in Fig.3.2. The loading was displacement-controlled at 0.03 mm/min.

In order to investigate the effects of fatigue loading on the in-situ strength of oil-coated PVA fibers, similar monotonic and fatigue uniaxial tests program (to that for non-oil fibers) were conducted (Table 3.5). The embedment length $L_e$ of all specimens were kept above 15 mm to ensure there is no full-debonding before fiber rupture, so that the fiber strength reduction by large slippage was eliminated. The test set-up is shown in Fig.3.2. The monotonic test was displacement-controlled at 0.03 mm/min; the fatigue test was load-controlled as shown in Fig.3.3.

A serial test composed of fatigue preloading and monotonic reloading was conducted to pullout specimens with oil-coated PVA fibers, similar to that for non-oil-coated PVA fibers. The details are given in Table 3.6.

3.3.3. Results and discussions

Effect of fiber slippage on in-situ fiber strength

The monotonic in-situ fiber strength was measured for embedded fiber specimen of varying fiber embedment length $L_e$. Fig.3.18 shows the relationship between in-situ fiber strength and fiber embedment length. As can be seen in Fig.3.18a, the in-situ fiber strength increases with increased fiber embedment length until a certain level ($L_e \sim 12$ mm), beyond which the impact of increased embedment length on in-situ fiber strength is very limited. Detailed study shows that thin embedded fiber experiences full-debonding (blue diamond) characterized by a sudden load drop (Fig.3.19) in the single fiber pullout curve while the thick embedded fiber specimens generally show no complete debonding before sudden fiber rupture as shown in Fig.3.19.

Fiber slippage of embedded fiber specimens was determined based on displacement between full-debonding and fiber rupture in the single fiber pullout curve as shown in
Fig. 3.19. For specimen that fiber rupture before complete debonding (i.e. thick pullout specimen), the slippage of fiber is assumed to be zero. As can be seen in Fig. 3.18b, fiber slippage reduces with increased embedment length for fully debonded samples. Fig. 3.18c further illustrates relationship between in-situ fiber strength and fiber slippage. As can be seen, in-situ fiber strength decreases with increased fiber slippage. The large relative fiber slippage from the surrounding matrix not only abrades fiber surface but also cause severe fiber delamination (Redon et al. 2001).

Fig. 3.20 compares the morphology of ruptured fibers with and without undergo full-debonding and slippage. It can be seen that if the fiber was ruptured after its full-debonding and large slippage, as shown in Fig. 3.20a, the fiber experienced severe surface abrasion. At higher magnification, fibrils in the fiber seemed to be broken progressively, which indicates the fibril delamination. However, if the fiber was ruptured before full-debonding as shown in Fig. 3.20b, the fiber tip appears to the blunt and intact; the cement paste covers the fiber, indicating there was no fiber delamination before it ruptured.

**Effect of oil-treatment on fatigue-induced fiber strength reduction**

Fig. 3.21 compares the normalized in-situ strength-fatigue life curve of embedded fiber with and without surface oil-treatment. The normalized in-situ fiber strength is defined as the ratio of fatigue fiber strength over monotonic fiber strength. For the oil-coated group, the fiber embedment length of all specimens was kept above 15 mm to prevent the influence of fiber slippage on fiber strength. It can be seen that surface oil-coating effectively reduces the fatigue strength reduction with increased load cycles. This may attributed to that oil-coating treatment reduces the interfacial bond between the fiber and matrix and the fiber abrasion under fatigue load is mitigated as a result.

**Effect of oil-treatment on fiber/matrix interfacial properties subject to fatigue**

Fatigue hardening and fatigue slip-hardening refer to the phenomena that the fiber-matrix interfacial frictional bond, in the debonding stage and slippage stage
respectively, is increased by fatigue load. The degree of these interfacial hardening effects depends on the load cycles as well as load level: the frictional bond gets stronger with increasing fatigue load cycles and load level. In the previous tests with non-oil-coated specimen, interfacial hardening is reflected by enhancing chemical bond $G_d$ and frictional bond $\tau_d$ (debonding stage) and $\tau_s$ (slippage stage) with load cycle and load level. In current tests with oil-coated specimens, similar fatigue hardening effect is observed and compared with non-oil-coated specimen.

Fig.3.22 and 3.23 compares oil-coated and non-oil-coated pullout specimens in terms of the change of chemical frictional bond $G_d$, frictional bond $\tau_d$ and $\tau_s$ with the increasing load cycle $N$ and load level $P_{max}$ experienced in fatigue preloading, respective. It can be seen that oil-treatment reduces the interfacial chemical bond and frictional bond, for both monotonic and fatigue loading scenarios. The effects of fatigue loading on oil-coated specimens are similar to that observed with non-oil-coated specimens: the fatigue loading did not change the chemical bond; the fiber/matrix friction increases with $N$ and $P_{max}$, indicating the fatigue hardening and fatigue slip-hardening effects still exist. However, the increasing rates of $\tau_d$ and $\tau_s$ of oil-coated specimens are much reduced when compared with that of non-oil-coated specimens. It indicates that surface oil-treatment can mitigate the effect of fatigue hardening. It could be attributed to the lower interfacial chemical bond introduced by the oil-treatment, which may generate smoother debonded interface and mitigated fiber abrasion during fatigue loading.

3.4. Conclusions

In this chapter, experiments were carried out in the component micro-scale level to trace the sources of fatigue-dependency in ECC. Specifically, fiber and fiber-matrix interface properties subject to fatigue are experimentally investigated. Monotonic and fatigue single-fiber tests were conducted to two type of specimens, i.e. embedded fibers and pullout specimens, to investigate the effects of fatigue load level $P_{max}$ and load cycle $N$ on the (in-situ) fiber strength and fiber-matrix interface properties, respectively. Effects of fiber surface treatment on fatigue-dependency of fiber and interface
properties were studied as well (Section 3.3). The conclusions based on the experiment from Section 3.1 are drawn as:

- In-situ strength of embedded fibers is remarkably lower than that of naked fibers due to the fiber surface abrasion by the surrounding matrix. Fatigue loading would further reduce the in-situ fiber strength by aggravating the fiber surface abrasion and lowering the nominal fiber strength (naked fiber test). SEM images reveal distinctive failure modes of the naked fibers and the embedded fibers subject to monotonic as well as the fatigue loads. However, fatigue loading does not aggravate the in-situ fiber strength reduction due to the orientation.

- Fatigue strength of naked fiber decreases bi-linearly with load cycles, while that of embedded fiber decreases linearly with load cycles. The severity of fiber degradation enhances with increasing load cycles.

The conclusions based on the experiment from Section 3.2 are drawn as:

- Propagation of the fiber-matrix interfacial crack with fatigue cycles, i.e. *fatigue-induced fiber debonding*, is discovered from the post-fatigue single-fiber pullout tests. The fiber-matrix interface debonds under constant fatigue load that is lower than the monotonic debonding strength. The fiber debonding rate depends on the fatigue load level and follows Paris’ Law.

- In the post-fatigue single-fiber pullout test, the fiber-matrix friction in the debonding is enhanced by the fatigue loading, even if the loading level is too low to cause full-debonding. Such phenomenon is referred as *fatigue hardening*, and the degree of fatigue hardening depends on the load level and number of fatigue cycles experienced.

- In the post-fatigue single-fiber pullout test, the fiber-matrix friction in the slippage stage is also enhanced, by the relatively high-level fatigue loading. Such phenomenon is referred as *fatigue slip-hardening*. The degree of fatigue
slip-hardening also depends on the load level and number of fatigue cycles experienced.

The conclusions based on the experiment from Section 3.3 are drawn as:

- The monotonic in-situ strength of embedded fiber is subject to the fiber embedment length. At relatively large embedment length, there would be no fiber full-debonding from the surrounding matrix and the in-situ strength remains constant despite of the varying embedment length; at relative small embedment length, the embedded would completely debond and slip from the matrix, as the embedment length decrease, the slippage would increase and resulting in lower in-situ fiber strength.

- The fatigue-induced in-situ fiber strength reduced is also observed for oil-coated embedded fibers. The fiber strength decreases with the fatigue load cycles experienced. However, the strength reduction rate is much decreased compared with non-oil-coated embedded fiber, as the surface oil-coating decreases the fiber-matrix interfacial bond and mitigate the fiber abrasion under fatigue loads.

- Both fatigue hardening and fatigue slip-hardening are observed for pullout specimen with oil-coated fibers. The rate of frictional bond increase, however, is decreased due to the surface oil-treatment.
References


Table 3.1 Properties of PVA fibers

<table>
<thead>
<tr>
<th>Diameter</th>
<th>Nominal tensile strength</th>
<th>Density</th>
<th>Surface oil coating by mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>39 μm</td>
<td>1,600 MPa</td>
<td>1,300 kg/m³</td>
<td>0.0%</td>
</tr>
</tbody>
</table>

Table 3.2 Mix proportions of mortar matrix

<table>
<thead>
<tr>
<th>Cement</th>
<th>GGBS</th>
<th>Sand</th>
<th>Water</th>
<th>Superplasticizer</th>
<th>Water/binder (cement+GGBS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>551 kg/m³</td>
<td>827 kg/m³</td>
<td>276 kg/m³</td>
<td>414 kg/m³</td>
<td>2.2 L/m³</td>
<td>0.30</td>
</tr>
</tbody>
</table>

Table 3.3 Testing program for fiber strength under fatigue loading

<table>
<thead>
<tr>
<th>Type of specimen</th>
<th>Monotonic loading (displacement control)</th>
<th>Fatigue loading (load control)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( P_{\text{min}} ) (N)</td>
</tr>
<tr>
<td>Naked fiber</td>
<td>0.03 mm/min</td>
<td>0.2</td>
</tr>
<tr>
<td>Embedded fiber ((L_e&gt;7 \text{ mm, } 0^\circ))</td>
<td></td>
<td>0.2</td>
</tr>
<tr>
<td>Embedded fiber ((L_e&gt;7 \text{ mm, } 30^\circ))</td>
<td></td>
<td>0.2</td>
</tr>
</tbody>
</table>

Table 3.4 Testing program for interfacial degradation under fatigue loading \((L_e \sim 1.5 \text{ mm})\)

<table>
<thead>
<tr>
<th>Targeting problem</th>
<th>Testing procedure</th>
<th>Fatigue preloading (load control)</th>
<th>Monotonic reloading (displacement control)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( P_{\text{min}} ) (N)</td>
<td>( P_{\text{max}} ) (N)</td>
</tr>
<tr>
<td>Control group</td>
<td>N.A.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Effect of fatigue loading on debonding stage</td>
<td>0.2</td>
<td>0.35-0.40</td>
<td>10k-500k</td>
</tr>
<tr>
<td>Fatigue-induced fiber debonding</td>
<td>0.2</td>
<td>0.40-0.50</td>
<td>10k-500k</td>
</tr>
<tr>
<td>Effect of fatigue loading on slippage stage</td>
<td>0.2</td>
<td>0.50-0.70</td>
<td>1k-100k</td>
</tr>
</tbody>
</table>
Table 3.5 Testing program for oil-coated fiber in-situ strength under fatigue loading

<table>
<thead>
<tr>
<th>Type of specimen</th>
<th>Monotonic loading (displacement control)</th>
<th>Fatigue loading (load control)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Embedded fiber ($L_e&gt;15 \text{ mm, } 0^\circ$)</td>
<td>0.03 mm/min</td>
<td>$P_{\text{min}}$ (N)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.2</td>
</tr>
</tbody>
</table>

Table 3.6 Effects of load cycle $N$ and load level $P_{\text{max}}$ in fatigue test on $P_a$, $P_b$, $P_a - P_b$, and the debonding stiffness $k_d$ in the debonding stage

<table>
<thead>
<tr>
<th></th>
<th>$N=0$</th>
<th>$N=10,000$</th>
<th>$N=100,000$</th>
<th>$N=500,000$</th>
<th>$N=10,000$</th>
<th>$P_{\text{max}}=0.35N$</th>
<th>$P_{\text{max}}=0.50N$</th>
<th>$P_{\text{max}}=0.70N$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_a$ (N)</td>
<td>0.48 ± 0.12</td>
<td>0.51 ± 0.05</td>
<td>0.57 ± 0.03</td>
<td>0.59 ± 0.04</td>
<td>0.53 ± 0.06</td>
<td>0.48 ± 0.12</td>
<td>0.51 ± 0.05</td>
<td>0.57 ± 0.03</td>
</tr>
<tr>
<td>$P_b$ (N)</td>
<td>0.35 ± 0.12</td>
<td>0.38 ± 0.04</td>
<td>0.43 ± 0.04</td>
<td>0.44 ± 0.12</td>
<td>0.41 ± 0.06</td>
<td>0.35 ± 0.12</td>
<td>0.38 ± 0.04</td>
<td>0.43 ± 0.04</td>
</tr>
<tr>
<td>$P_a-P_b$ (N)</td>
<td>0.13 ± 0.03</td>
<td>0.13 ± 0.03</td>
<td>0.13 ± 0.03</td>
<td>0.15 ± 0.05</td>
<td>0.12 ± 0.06</td>
<td>0.13 ± 0.03</td>
<td>0.13 ± 0.03</td>
<td>0.13 ± 0.03</td>
</tr>
<tr>
<td>$k_d$ (N/mm)</td>
<td>5.31 ± 0.89</td>
<td>8.06 ± 0.99</td>
<td>8.81 ± 0.91</td>
<td>10.74 ± 0.48</td>
<td>9.21 ± 1.48</td>
<td>5.31 ± 0.89</td>
<td>8.06 ± 0.99</td>
<td>8.81 ± 0.91</td>
</tr>
</tbody>
</table>

Table 3.7 Effects of load cycles $N$ and load level $P_{\text{max}}$ in fatigue test on the transition load ($P_t$) and the slippage stiffness ($k_{s1}$ and $k_{s2}$) in the slippage stage

<table>
<thead>
<tr>
<th></th>
<th>$N=1,000$ $P_{\text{max}}=0.60N$</th>
<th>$N=10,000$ $P_{\text{max}}=0.60N$</th>
<th>$N=100,000$ $P_{\text{max}}=0.60N$</th>
<th>$N=10,000$ $P_{\text{max}}=0.50N$</th>
<th>$N=10,000$ $P_{\text{max}}=0.70N$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_t$ (N)</td>
<td>0.59 ± 0.02</td>
<td>0.64 ± 0.02</td>
<td>0.66 ± 0.02</td>
<td>0.54 ± 0.02</td>
<td>0.74 ± 0.03</td>
</tr>
<tr>
<td>$k_{s1}$ (N/mm)</td>
<td>6.27 ± 0.95</td>
<td>8.98 ± 1.31</td>
<td>9.26 ± 1.47</td>
<td>7.99 ± 0.57</td>
<td>8.24 ± 0.59</td>
</tr>
<tr>
<td>$k_{s2}$ (N/mm)</td>
<td>2.14 ± 0.94</td>
<td>2.39 ± 0.89</td>
<td>2.92 ± 0.76</td>
<td>1.82 ± 0.48</td>
<td>3.66 ± 0.47</td>
</tr>
</tbody>
</table>
Table 3.8 Testing program for interfacial degradation (oil-coated fiber) under fatigue loading

<table>
<thead>
<tr>
<th>Targeting problem</th>
<th>Testing procedure</th>
<th>Fatigue preloading (load control)</th>
<th>Monotonic reloading (displacement control)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$P_{\text{min}}$ (N)</td>
<td>$P_{\text{max}}$ (N)</td>
</tr>
<tr>
<td>Control group</td>
<td>N.A.</td>
<td>0.2</td>
<td>0.30-0.40</td>
</tr>
<tr>
<td>Effect of fatigue loading on debonding stage (Le ~2.5 mm)</td>
<td></td>
<td>0.2</td>
<td>0.60-0.70</td>
</tr>
<tr>
<td>Effect of fatigue loading on slippage stage (Le ~1.5 mm)</td>
<td></td>
<td>0.2</td>
<td>0.30-0.40</td>
</tr>
</tbody>
</table>
Fig. 3.1 Illustration of the specimen preparation.

Fig. 3.2 Test set-up for monotonic and fatigue tensile tests
Fig. 3.3 The loading scheme of fatigue test

Fig. 3.4 PVA fiber (in-situ) strength vs. fatigue life
Fig. 3.5 FE-SEM images of monotonically ruptured embedded fiber (embedded fiber with $L_e > 7$ mm): (a) 200x; (b) 2,000x; (c) 10,000x
Fig.3.6 FE-SEM images of monotonically ruptured naked fiber (a) and fatigue ruptured naked fiber (b, 151 cycles; c, 102,040 cycles; d, 305,070 cycles)
Fig. 3.7 FE-SEM images of fatigue ruptured embedded fiber (embedded fiber with $L_e > 7$ mm): (a) 200×; (b) 1,000×
Fig. 3.8 FE-SEM (JOEL JSM-7600F) images showing the existence of striation on fatigue ruptured embedded fibers (embedded fiber with $L_e > 7 \text{ mm}$): (a) 100×; (b) 2,000×; (c) 10,000×
Fig. 3.9 Three types of displacement history in fatigue test: (a) no complete debonding of fiber in the cyclic phase, \(P_{\text{max}}=0.35\text{N}, N=10\text{k}\); (b) complete debonding of fiber in the cyclic phase, \(P_{\text{max}}=0.50\text{N}, N=10\text{k}\); (c) complete debonding of fiber in the ramping phase, \(P_{\text{max}}=0.70\text{N}, N=10\text{k}\).
Fig. 3.10 (a) The relation between the length of crack propagation $\Delta a$ and number of cycles $N$ to full-debonding; (b) the relation between fatigue-induced fiber debonding rate $(\Delta a/N)$ and the maximum load in fatigue tests $P_{\text{max}}$. 

$$(\Delta a/N) = 1095 \times P_{\text{max}}^{13.54}$$
Fig. 3.11 Typical tensile load-fiber displacement ($P$-$u$) relation of control specimen and specimen that has experienced relatively low-level fatigue loads ($\leq 0.40$N).
Fig. 3.12 Effects of number of fatigue cycle $N$ and fatigue load level $P_{\text{max}}$ on chemical bond $G_d$ and frictional bond $\tau_d$ of debonding stage
Fig. 3.13 Change of tensile stiffness in the cyclic phase of the fatigue test (maximum load=0.35 N). Due to the dynamic effect the stiffness value is much higher than that in monotonic test.
Fig. 3.14 (a) The surface of intact naked fiber; (b) the surface of embedded fiber experienced fatigue loads, which shows abrasion and loose debris attached to the fiber surface.
Fig. 3.15 Naked fiber softening as fatigue load cycle increasing

Fig. 3.16 Typical tensile load-fiber displacement ($P-u$) relation of control specimen and specimen that has experienced relatively high-level fatigue loads ($\geq 0.50$ N).
Fig. 3.17 Effects of number of fatigue cycle $N$ and fatigue load level $P_{max}$ on frictional bond of slippage stage $\tau_s$
Fig. 3.18 Relationships between in-situ fiber strength, fiber slippage, and fiber embedment length
Fig. 3.19 Tensile load-fiber displacement relation of embedded fiber specimen of large $(L_e > 12 \text{ mm})$ or small $(L_e < 12 \text{ mm})$ fiber embedment length
Fig. 3.20 Ruptured fiber tip morphology: (a and b) fiber underwent complete debonding followed by fiber slippage; and (c) sample ruptured without complete debonding and fiber slippage.
Fig. 3.21 Normalized in-situ strength vs. fatigue life of embedded PVA fiber
Fig. 3.22 Change of chemical bond $G_d$, frictional bond of debonding stage $\tau_d$ and slippage stage $\tau_s$, with fatigue load level $N$ in the non-oil-coated and oil-coated specimens.
Fig. 3.23 Change of chemical bond $G_d$, frictional bond of debonding stage $\tau_d$ and slippage stage $\tau_s$, with fatigue load level $P_{\text{max}}$ in the non-oil-coated and oil-coated specimens.
CHAPTER 4 MICROMECHANICS-BASED FATIGUE FIBER BRIDGING MODEL

Chapter 3 experimentally studied the fatigue-dependency of the ECC component, such as fiber and fiber-matrix interfacial bond. The results reveal that fatigue load would significantly alter the single-fiber pullout behavior: 1) fatigue loading would reduce the in-situ fiber strength; 2) fatigue loading would cause debonding between fiber and matrix and allow propagation of interface tunneling crack, or the fatigue-induced fiber debonding; 3) the fatigue loading would induce stronger interfacial bond between the fiber and matrix, i.e. the fatigue hardening effect and fatigue slip-hardening effect.

These changes of micro-scale properties of ECC, as observed in the single-fiber pullout test, are expected to introduce changes of meso-scale properties of ECC, i.e. the relationship between fiber-bridging stress and crack opening displacement ($\sigma-\delta$ curve). It is already well known that the ultra-high ductility of ECC, which results from its multiple-cracking behavior, depend on the two pseudo strain-hardening (PSH) indices as defined in Eqn.4.1 and 4.2 (Yang et al. 2008).

$$PSH\ energy = \frac{J_p}{J_{tip}}$$  \hspace{1cm} (Eqn.4.1)

$$PSH\ strength = \frac{\sigma_0}{\sigma_c}$$  \hspace{1cm} (Eqn.4.2)

where $J_{tip}$ and $\sigma_c$ are the crack tip toughness and matrix tensile crack strength respectively. In order to achieve saturated multiple cracking and robust strain-hardening behavior, $PSH\ energy$ and $PSH\ strength$ are required to be larger than 3 and 1.2 respectively (Yang and Li 2010). Under fatigue loading, both $PSH\ energy$ and $PSH\ strength$ of ECC may decrease and results in unsaturated multiple cracking. In order to determine the effect of fatigue loading (fatigue load level and number of fatigue load cycle) on strain-hardening behavior and tensile ductility of ECC on the macro-scale, it is important to quantitatively calculate the post-fatigue $PSH\ energy$ and $PSH\ strength$, by creating a fiber-bridging $\sigma-\delta$ model that considers the effect of fatigue load.
In the study of the strain-rate effect on ECC, Yang et al. (2008) added the strain-rate dependency, which are characterized by the single-fiber pullout test with different loading rate (Yang and Li 2006), to the existing static micromechanics-based model of ECC (Lin et al. 1999, Yang et al. 2008). With this modified model, the effects of high loading rate on ECC fiber-bridging behavior are quantitatively determined, under the guidance of which Yang et al. (2012) successfully made ECC with high impact resistance by tailoring the ingredients and mix proportion design. Therefore, modifying the micromechanics-based model with the characterization test results could be a very plausible approach to determine the post-fatigue fiber-bridging of ECC.

This chapter presents the work of adding the fatigue-dependency into the established micromechanics-based monotonic ECC model. Together with Chapter 3, it forms the theoretical base to tailor ECC for higher fatigue resistance (Chapter 5 and 6). Specifically, Section 4.1 reports the work to model the post-fatigue single-fiber pullout behavior on the micro-scale, the work to form post-fatigue fiber-bridging σ-δ curve on the meso-scale, and the work to form a crack propagation model for ECC under flexural fatigue loading to predict the fatigue life; Section 4.2 shows the parametric study with the new model on the effect of fatigue loading on fiber-bridging of ECC and predicts the ECC fatigue life under different load levels.

4.1. Micromechanics-based fatigue model of ECC

4.1.1. Micro-scale: fatigue single-fiber pullout model

Fiber debonding stage

Fatigue-induced fiber debonding, i.e. fatigue-induced fiber debonding, and the fatigue-induced friction enhancement in debonding stage, i.e. fatigue hardening, which were observed in the debonding stage of single-fiber pullout, should be characterized and considered in the new model. The approaches to include these two effects are presented separately.
In the monotonic model, the process of fiber debonding from matrix is captured by a fracture-based approach: the debonding crack propagates only when the surface energy $G$, or the energy released by the creation of new crack surface of unit area, is equal to the chemical bond between fiber and matrix $G_d$. Besides the resistance from the propagating crack, the friction $\tau_0$ at the deboned surface also contributes to the tensile load $P$. When interface is debonding, based on fracture criterion the relation between $P$ and debonding crack length $a$ is given as Eqn.4.3:

$$ P = \pi \tau_0 a (1 + \eta)d_f + \sqrt{\pi^2 G_d E_f (1 + \eta)d_f^3 / 2} \quad \text{(Eqn.4.3)}$$

where $d_f$ is the fiber diameter, $E_f$ is the Young’s modulus of the fiber, $\eta = E_f V_f / E_m (1 - V_f)$, where $E_m$ is the Young’s of the matrix, and $V_f$ is the fiber fraction by volume. Based on the equilibrium of the fiber-matrix system, the correlation between $P$, $a$ and fiber displacement $u$ is obtained as Eqn.4.4:

$$ P = \pi E_f d_f^2 u / 4a + \pi \tau_0 a (1 + \eta)d_f / 2 \quad \text{(Eqn.4.4)}$$

By combining Eqn.4.3 and 4.4 and eliminating $a$, the monotonic $P-u$ relation is obtained as Eqn.4.5:

$$ P = \sqrt{\pi^2 (\tau_0 u + G_d) E_f d_f^3 (1 + \eta) / 2} \quad \text{(Eqn.4.5)}$$

As shown by the experimental results, the fiber-matrix interface debonds under fatigue load with constant loading level. Therefore, in current model the debonding crack length is composed of $a_0$, the crack length at the peak of the first load cycle and $\Delta a$, the crack propagation induced by fatigue loads (Eqn.4.6),

$$ a = a_0 + \Delta a (N, P_{max}) \quad \text{(Eqn.4.6)}$$

where $a_0$ can be calculated with Eqn.4.3 by substituting $P$ with $P_{max}$, the fatigue load level; $\Delta a$ as stated in Chapter 3, is governed by Eqn.4.7

$$ \Delta a = N_d \cdot C \cdot P_{max}^M \quad \text{(Eqn.4.7)}$$
In addition to fatigue debonding, \( \tau_d \) the frictional bond in the debonding stage is enhanced by fatigue loading. Such fatigue hardening effect can be captured by introducing the fatigue hardening coefficients \( \beta_d \), and substituting \( \tau_0 \) with \( \tau_d = \tau_0 (1 + \beta_d) \). The coefficients \( \beta_d \) is calculated as Eqn.4.8.

\[
\beta_d = \beta_d(N_d, P_{\text{max}}) \tag{Eqn.4.8}
\]

where \( N_d \) are the fatigue cycles applied before the fiber fully debonded from the matrix. Fig.3.12 indicates that both \( \beta_d \) increases with \( N \) and \( P_{\text{max}} \), which relationships must be quantitatively determined before calculating the effect of fatigue hardening.

In the monotonic model, the frictional bond \( \tau_0 \) can be calculated as \( (P_b/\pi dfLe) \) with the experimental results from the monotonic single-fiber pullout test (Redon et al. 2001). As the fatigue hardening coefficient \( \beta_d \) is introduced in current model, the frictional bond can be rewritten as Eqn.4.9

\[
\tau_0 (1 + \beta_d) = \frac{P_b}{\pi df Le} \tag{Eqn.4.9}
\]

where \( P_b \) is the force resistance immediately after full-debonding, which can be measured in the monotonic reloading test.

Fig.4.1 shows the relationship between \( \beta_d, N_d, \) and \( P_{\text{max}} \) from the previous test (refer to Fig.3.12, fatigue test with non-oil-coated PVA fibers). It can be seen that the fatigue hardening coefficient increases with the fatigue load cycles \( N_d \) in a logarithmic manner, and the increasing rate \( \beta_d/\log N_d \) is correlated with the power of \( P_{\text{max}} \). As a result, Eqn.4.8 can be rewritten as

\[
\beta_d = \begin{cases} C_d \cdot P_{\text{max}} N_d \cdot \log N_d, & N_d \geq 1 \\ 0, & N_d < 1 \end{cases} \tag{Eqn.4.10}
\]

**Fiber slippage stage**

Fatigue-induced friction enhancement was also observed in the slippage stage of single-fiber pullout test, i.e. *fatigue slip-hardening*, as shown in Chapter 3.2. This effect
can be captured by introducing a fatigue slip-hardening coefficient $\beta_s$, and substituting $\tau_0$ with $\tau = \tau_0 (1 + \beta_s)$. The coefficient $\beta_s$ is calculated as Eqn.4.11.

$$\beta_s = \beta_s(N_s, P_{max}) \quad \text{(Eqn.4.11)}$$

where $N_s$ are the fatigue cycles applied after fiber fully debonded from the matrix. Fig.3.17 indicates that $\beta_s$ increases with $N$ and $P_{max}$, which relationships must be quantitatively determined before calculating the effect of fatigue hardening.

In the model of monotonic single-fiber pullout, the tensile load $P$ in the slippage stage is equal to $\tau_0 \pi d_f [L_e + S(L_e \beta / d_f - 1)]$, where $\beta$ is the slip-hardening coefficient under monotonic load, $S$ is the fiber slippage displacement and the second-order term $S^2$ is very small and hence are ignored (Redon et al. 2001). In current research, the force resistance, when ignoring the second-order term of slippage, can be rewritten as

$$P = \tau_0 (1 + \beta_d + \beta_s) \pi d_f [L_e + S \left( \frac{L_e \beta}{d_f} - 1 \right)] \quad \text{(Eqn.4.12)}$$

And based on the derivation of Eqn.4.12, $\beta_s$ can be calculated as

$$\beta_s = \frac{\Delta P/\Delta S|_{\Delta S\to 0}}{\tau_0 \pi (L_e \beta / d_f)} - 1 - \beta_d \quad \text{(Eqn.4.13)}$$

where $\Delta P/\Delta S|_{\Delta S\to 0}$ can be obtained from the monotonic reloading test.

Fig.4.2 shows the relationship between $\beta_s$, $N_s$, and $P_{max}$ the previous test (refer to Fig.3.17 fatigue test with non-oil-coated PVA fibers), where $\beta_s$ is calculated with Eqn.4.13. Here the effect of $N_d$ is ignored as before the cyclic loading phase, the fiber has been fully debonded from the matrix. It can be seen that the fatigue slip-hardening coefficient increases with the fatigue load cycles $N_s$ in a logarithmic manner, and the increasing rate $\beta_d / \log N_s$ is correlated with the power of $P_{max}$. As a result, Eqn.4.8 can be rewritten as

$$\beta_s = \begin{cases} C_s \cdot P_{max}^{M_s} \cdot \log N_s, & N_s \geq 1 \\ 0, & N_s < 1 \end{cases} \quad \text{(Eqn.4.14)}$$
Overall post-fatigue single-fiber pullout behavior

By introducing the debonded crack length \( a \) and the hardening coefficients \( \beta_d \) and \( \beta_s \), the load-deflection relationship after fatigue loading, i.e. \( P(u, N_d, N_s, P_{\text{max}}) \) can be expressed as Eqn.4.15.

\[
P = \begin{cases} 
\min \left\{ \frac{\pi E_f d_f^2 u}{4a} + \pi \tau_0 (1 + \beta_d) a (1 + \eta) d_f / 2 }{ \sqrt{\pi^2 [\tau_0 (1 + \beta_d) u + G_d] E_f d_f^3 (1 + \eta)/2} }, & u < u_0 \\
\pi d_f \tau_0 (1 + \beta_d + \beta_s) (L_e + u_0 - u) \left[ 1 + \frac{\beta(u-u_0)}{d_f} \right], & u \geq u_0
\end{cases}
\]

(Eqn.4.15a)

\[
P = \pi d_f \tau_0 (1 + \beta_d + \beta_s) (L_e + u_0 - u) \left[ 1 + \frac{\beta(u-u_0)}{d_f} \right], & u \geq u_0
\]

(Eqn.4.15b)

\[
a = \min \left\{ a_0 + \Delta a (N_d, P_{\text{max}}) \right\} / L_e
\]

(Eqn.4.15c)

\[
a_0 = \frac{p_{\text{max}} - \sqrt{\pi^2 G_d E_f (1 + \eta) d_f^3 / 2}}{\pi \tau_0 d_f (1 + \eta)}
\]

(Eqn.4.15d)

\[
u_0 = \frac{2 \tau_0 L_e^2 (1 + \eta)}{E_f d_f} + \frac{L_e}{E_f} \sqrt{\frac{8 G_d E_f (1 + \eta)}{d_f}}
\]

(Eqn.4.15e)

where \( \Delta a, \beta_d, \) and \( \beta_s \) are calculated with Eqns. 4.7, 4.11, and 4.14 respectively.

The \( P-u \) curve of single-fiber pullout after fatigue are calculated based on the modified model. In the analytical model, the \( N_d \) and \( N_s \) are calculated as Eqn.4.16 and 4.17.

\[
N_d = \begin{cases} 
N, & N < N_{d0} \\
N_{d0}, & N > N_{d0}
\end{cases}
\]

(Eqn.4.16a)

\[
N_s = N - N_d
\]

(Eqn.4.16b)

where \( N_{d0} \) is the number of cycle needed to fully debond the fiber from matrix and calculated as Eqn.4.17.

\[
N_{d0} = \begin{cases} 
1, & a_0(P_{\text{max}}) > L_e \\
(L_e - a_0)/(C \cdot P_{\text{max}}^{\alpha}), & a_0(P_{\text{max}}) < L_e
\end{cases}
\]

(Eqn.4.17)
4.1.2. Meso-scale: fatigue fiber-bridging model

On the micro-scale, the post-fatigue load-fiber displacement behavior of an individual embedded fiber has been characterized and modeled. On the meso-scale, once the crack occurs the fiber-bridging are sustained by plenty of individual fibers, which are being pulled out from the matrix; as a result, the post-fatigue fiber-bridging of ECC, i.e. the relation between ambient tensile stress and crack opening displacement ($\sigma$-\(\delta\)), can be obtained by summing the load carried by the individual fibers under certain fiber displacement.

In single-fiber pullout model, the fiber embedment length \(L_e\) is predetermined and the fiber orientation \(\varphi\) (the angle between fiber alignment and tensile loading) is 0°; in the fiber-bridging model, the randomness of fiber distribution must be considered by varying \(L_e\) and \(\varphi\). It has been discovered that misaligned fiber (\(\varphi \neq 0\)) is subject to additional load due to the Euler friction pulley effect at the fiber exit point, and new \(P(\varphi)\) can be calculated as suggested by (Morton and Groves 1976):

\[
P(\varphi) = P(\varphi = 0) \cdot e^{f\varphi}
\]

(Eqn.4.18)

where \(f>0\) is referred as snubbing coefficient.

When accounting the force resistance by individual fibers, fiber rupture must be considered by introducing the in-situ fiber strength, \(\sigma_{fu}\). It has been reported that the in-situ fiber strength is a function of fiber orientation (Yang et al. 2008); and it is also affected by the fatigue load cycles, as reported in Chapter 3. As a result, it can be calculated as:

\[
\sigma_{fu}(\varphi,N) = \sigma_{fu}(\varphi = 0,N = 1) \cdot e^{-f''\varphi} \cdot [1 - f''' \cdot \log(N)]
\]

(Eqn.4.19)

\(f''>0\) and \(f'''>0\) are referred as fiber strength factor due to fiber orientation and fatigue load, respectively.
The randomness of $L_e$ and $\phi$ can be accounted by adopting probability density functions $p(z)$ and $p(\phi)$, where $z$ is the distance from fiber geometrical center to the crack surface, and $L_e=L_f/2-z$. (Yang et al. 2008) suggested that $p(z)$ and $p(\phi)$ can be calculated as:

\[
p(z) = \frac{2}{L_f}, \quad 0 < z < L_f/2 \tag{Eqn.4.20}
\]

\[
p(\phi) = \begin{cases} 
\frac{2}{\pi}, & 0 < \phi < 2/\pi, \quad 2D \text{ fiber distribution} \\
\sin(\phi), & 0 < \phi < 2/\pi, \quad 3D \text{ fiber distribution}
\end{cases} \tag{Eqn.4.21}
\]

where the 2D fiber distribution is applied to the ECC specimen whose smallest dimension is comparable to the fiber length; and the 3D fiber distribution is applied to the ECC specimen whose smallest dimension is much larger than the fiber length.

For a given cross section area of specimen $A$, the number of fibers $N_f$ for within a specific range of $L_e$ ($z_{e1}$ to $z_{e2}$) and $\phi$ ($\phi_1$ to $\phi_2$) can be calculated as:

\[
N_f = \frac{A}{A_f} \cdot V_f \cdot \int_{\phi_1}^{\phi_2} \int_{z_{e1}}^{z_{e2}} \cos\phi \cdot p(\phi) p(z) dz d\phi \tag{Eqn.4.22}
\]

where $A_f$ is the cross section area of an individual fiber and $V_f$ is the fiber content fraction by volume. The total number of fibers crossing the crack can be calculated with Eqn.4.22, when adopting $z_1=0$, $z_2=L_f/2$, $\phi_1=0$, and $\phi_2=\pi/2$.

With the aforementioned equations (Eqn.4.7, 4.10, 4.14-4.22), the fiber-bridging constitutive law after fatigue loading, i.e. $\sigma(\delta, N, \sigma_{\text{max}})$ where $N$ is the number of load cycles, $\sigma_{\text{max}}$ is the ambient fatigue tensile stress, can be computed with a numerical procedure illustrated in Fig.4.3.

4.1.3. Macro-scale: composite flexural fatigue model

On the meso-scale, a fatigue-dependent fiber-bridging model has been developed for ECC, and the effect of fatigue load level and load cycles, i.e. $\sigma(\delta, N, \sigma_{\text{max}})$, can be characterized. On the macro-scale, this model is used to predict ECC flexural fatigue behavior, such as flexural crack propagation, and flexural fatigue life.
Li and Matsumoto (1998) developed a micromechanics-based model to predict the flexural crack propagation of FRC under fatigue loading. This model considers the degradation of fiber-matrix interfacial bond and successfully captures the increase of flexural crack length $a_{\text{flexure}}$ with increasing load cycles $N$. However, this model does not consider the chemical bond between fiber and matrix; as a result it can only be applied to FRC in which the chemical bond can be ignored such as steel fiber or hydrophobic fiber (PE). Here Li and Matsumoto’s model is modified by adopting the aforementioned fiber-bridging degradation model.

In the flexural fatigue test, crack growth of concrete is observed to obey Paris’ Law (Baluch et al. 1989, Perdikaris and Calomino 1989, Bazant and Schell 1993), which gives the relation between the crack growth rate and the crack-tip-intensity factor amplitude:

$$\frac{da_{\text{flexure}}}{dN} = c \cdot \Delta K_{\text{tip}}^m$$ (Eqn.4.23)

where $a_{\text{flexure}}$ is the crack length, $N$ is the number of fatigue cycles, $c$ and $m$ are the Paris constants depending on the matrix properties, $\Delta K_{\text{tip}}$ is the stress intensity factor amplitude at the crack tip. Hence the fatigue life $N_{\text{failure}}$ can be computed as:

$$N_{\text{failure}} = \int_{a_{\text{flexureo}}}^{a_{\text{flexuref}}} \left( \frac{1}{c \cdot \Delta K_{\text{tip}}^m} \right) da_{\text{flexure}}$$ (Eqn.4.24)

where $a_{\text{flexureo}}$ and $a_{\text{flexuref}}$ are the initial and failure crack length, respectively.

The stress intensity factor amplitude $\Delta K_{\text{tip}}$ is related to the stress distribution along the crack and can be calculated as:

$$\Delta K_{\text{tip}} = \Delta K_a + \Delta K_b$$ (Eqn.4.25)

where $\Delta K_a$ and $\Delta K_b$ are the stress intensity factor amplitude due to external loading and fiber-bridging respectively (Fig.4.4). For a beam with depth of $H$ under flexural loading, $\Delta K_a$ and $\Delta K_b$ are calculated as an integral of stress intensity over the crack length:
\[ \Delta K_a = 2 \int_0^{a_{flexure}} G(x, a, H) \cdot \frac{6 \Delta M_B}{B H^2} \cdot \left(1 - \frac{2x}{H}\right) dx \]  
\text{(Eqn.4.26)}

\[ \Delta K_b = -2 \int_0^{a_{flexure}} G(x, a, H) \cdot \sigma(\delta, N, \sigma_{max}) \, dx \]  
\text{(Eqn.4.27)}

where \( x \) is the measured from the tension face of the beam; \( B \) is the beam width; \( G(x, a, H) \) is a weight function that represents a unit force contribution to the stress intensity factor, and the details of \( G(x, a, H) \) can be found in (Cox and Marshall 1991, Tada et al. 2000); \( \sigma(\delta, N, \sigma_{max}) \) is the fiber-bridging stress considering fatigue degradation, where \( \delta \) and \( \sigma_{max} \) are function of \( x \) and can be expressed as \( \delta(x) \) and \( \sigma_{max}(x) \) respectively.

Within the cracked length, the axial elongation of beam is mainly contributed by the extension of crack width \( \delta \). As a result, the relation \( \delta(x) \) can be calculated as

\[ \delta(x) = s \cdot \varepsilon(x) \]  
\text{(Eqn.4.28)}

where \( s \) is the crack spacing and the tensile strain distribution \( \varepsilon(x) \) can be obtained based on the plane section assumption, as shown in Fig.4.5. The \( \varepsilon(x) \) is calculated as

\[ \varepsilon(x) = \frac{\varepsilon_{tc}(b_0-x)}{(a_{flexure}o-x)} \]  
\text{(Eqn.4.29)}

where \( \varepsilon_{tc} \) is the matrix cracking strain and \( b_0 \) is the distance from neutral axis to tension face at the first peak load of fatigue test. The \( \sigma_{max}(x) \) can be rewritten as

\[ \sigma_{max}(x) = \sigma(\delta(x))_{monotonic} \]  
\text{(Eqn.4.30)}

The crack width and the fiber-bridging stress in the original Eqns. 4.28 to 4.30 are assumed to remain unchanged under fatigue load cycles. This was intent to simplify the computation in Li et al.’s work. With the development of fatigue sensitive fiber-bridging model, the flexural fatigue propagation and flexural fatigue life of ECC can be computed by combining Eqns. 4.23 to 4.30. This is realized by a numerical procedure, the flow chart of which is illustrated in Fig.4.6.
4.2. Causes of premature failure of ECC subject to fatigue

Qian (2007) reported that the number of cracks to the fatigue failure of ECC M45 significantly reduces with the load cycles; similar observation was also reported by Suthiwarapirak et al. (2004). In summary, ECC exhibits premature failure and strain-hardening loss subject to fatigue.

By introducing the fatigue-induced fiber strength reduction, fatigue debonding, fatigue hardening, and fatigue slip-hardenig on the micro-scale, the effect of fatigue load on meso-scale fiber-bridging are now captured into the fatigue fiber-bridging model. Table 4.1 listed all the input parameters needed for computing the post-fatigue fiber-bridging behavior. The value of these parameters for single PVA fiber with and without oil coating, based on the testing results in Chapter 3, is also given. Fig.4.7 and Fig.4.8 shows the post-fatigue $\sigma(\delta)$ curve of ECC, which are computed with the modified model. It can be seen that the slope of the curve (increasing part) become steeper as $N$ increasing, reflecting the hardening effects observed in the single-fiber test; the load capacity of fiber bridging $\sigma_0$ gradually decrease as $N$ increasing, mainly reflecting the fiber rupture due to the fiber strength reduction.

With the $\sigma(\delta)$ curve obtained from the modeling results, $J_{b'}$ and $\sigma_0$ after fatigue loading can be calculated. Assuming $J_{tip}$=40N/m based on the suggestion of (Bažant and Becq-Giraudon 2002) for Grade 70 concrete, and $\sigma_c$=4MPa (See Chapter 3), the PSH indices are calculated and shown in Fig.4.9. It can be seen that the PSH energy for oil-coated fibers are much higher than that for non-oil-coated fibers; and both PSH energy and PSH strength decreases with load cycles. Subsequently the multiple-crack and strain-hardening behavior may be mitigated and even eliminated. This explains from a micromechanics point of view why ECC exhibits lower ductility subject to fatigue as observed in literature.

4.3. Validation of the composite flexural fatigue model

Table 4.2 listed all the input parameters needed for modeling the flexural fatigue behavior of ECC. The values of these parameters were selected based on the mix
design of GGBS60 (reported in Chapter 5), and the values in Table 4.1 (oil-coated fibers) were selected for parameters of the fatigue fiber-bridging model. The flexural fatigue life of ECC are computed at different loading level and compared with the experimental results, as shown in Fig.4.10.

It can be seen that the model can successfully predict the flexural fatigue strength decrease with fatigue cycles. The source of margin between model-predicted fatigue life from the experimental results may be attributed to the following cause: the slip-hardening coefficient $\beta$ is underestimated as fiber displacement in single-fiber pullout test is overestimated by including additional displacement by the set-up, which in turns underestimates the fatigue resistance due to fiber-bridging and results in faster degradation.

4.4. Conclusions

This chapter introduces the fatigue-dependency of ECC component on the micro-scale, i.e. the fatigue-induced fiber debonding, fatigue hardening, fatigue slip-hardening, and fatigue-induced fiber strength reduction observed from the single-fiber pullout test, into the established micromechanics-based monotonic ECC model developed by Li et al. By capturing the fatigue-dependency in the micro-scale, the modified model is able to compute the fiber-bridging constitutive law $\sigma(\delta)$ of ECC after fatigue degradation. The new fatigue-dependent fiber-bridging model successfully explains the cause of premature fatigue failure of ECC: the modeling results suggest that the pseudo-strain hardening coefficient $PSH$ energy and $PSH$ strength are greatly reduced by the fiber and interface degradation, resulting unsaturated multiple-cracking and reduced tensile ductility of ECC. The new model can be used to guide component tailoring and ingredients selection for fatigue-resistant ECC.

The fatigue dependent fiber-bridging $\sigma(\delta)$ relation is engaged into the flexural fatigue crack propagation model for normal FRC. The new model successfully predicts the flexural fatigue life of ECC GGBS under different loading level (relevant experiments will be reported in Chapter 5).
This chapter establishes a multi-scale model that predicts the ECC mechanical behavior under fatigue loading. This model, which represents a novel fatigue design approach, can be extended to a wide range of fatigue-sensitive engineering composites. It will not only capture the fatigue failure mode, but also predict the fatigue life of these materials.
Reference


### Table 4.1 Parameters as input to the fiber-bridging fatigue model

<table>
<thead>
<tr>
<th>Input parameters</th>
<th>Notation</th>
<th>Non-oil-coated PVA fiber</th>
<th>Oil-coated PVA fiber</th>
</tr>
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<tbody>
<tr>
<td><strong>Monotonic micro-scale parameters</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fiber Young’s modulus</td>
<td>$E_f$(GPa)$^1$</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td>Fiber diameter</td>
<td>$d_f$(mm)$^1$</td>
<td>39</td>
<td>39</td>
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<tr>
<td>Fiber fraction</td>
<td>$V_f$(%)</td>
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<td>2</td>
</tr>
<tr>
<td>Matrix Young’s modulus</td>
<td>$E_m$(GPa)$^2$</td>
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<td>20</td>
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<td>Chemical bond</td>
<td>$G_d$(J/m$^2$)</td>
<td>2.68</td>
<td>1.77</td>
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<tr>
<td>Frictional bond</td>
<td>$\tau_0$(MPa)</td>
<td>1.89</td>
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<td>Slip-hardening coefficient</td>
<td>$\beta$</td>
<td>0.19</td>
<td>0.21</td>
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<tr>
<td>Snubbing coefficient</td>
<td>$f^2$</td>
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<tr>
<td>Fiber strength reduction coefficient (orientation)</td>
<td>$f'^2$</td>
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<td><strong>Fatigue-dependent micro-scale parameters</strong></td>
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<td>Fatigue-debonding coefficients</td>
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<td>$M$</td>
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<td>Interfacial hardening coefficients</td>
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<td>$M_s$</td>
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<td>Fiber strength reduction coefficient (fatigue)</td>
<td>$f''$</td>
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<td>0.0536</td>
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</table>

All values in this table are measured in current study except $^1$ measured provided by the fiber manufacturer and $^2$ parameter value recommended by (Yang et al. 2008)
Table 4.2 Parameters as input to ECC flexural fatigue model

<table>
<thead>
<tr>
<th>Input parameters</th>
<th>Notation</th>
<th>Mix design GGBS60</th>
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</thead>
<tbody>
<tr>
<td>Beam depth</td>
<td>$H$ (mm)</td>
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<tr>
<td>Beam width</td>
<td>$B$ (mm)</td>
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<tr>
<td>Crack spacing</td>
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<td>Matrix Young’s modulus</td>
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</tr>
<tr>
<td>Matrix cracking strain</td>
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<tr>
<td>Paris’ constants$^2$</td>
<td>$c\left(m^{0.5}/\text{cycle\cdot MPa}\right)$</td>
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<tr>
<td></td>
<td>$m$</td>
<td>14</td>
</tr>
</tbody>
</table>

$^1$measured from flexural test result
$^2$ parameter value recommended by (Baluch et al. 1989)
Fig. 4.1 The relationship between fatigue hardening coefficient $\beta_d$ and fatigue cycle $N_d$ and fatigue load level $P_{\text{max}}$: 
$\beta_d = C_d \cdot P_{\text{max}}^{M_d} \cdot \log N$

(a) 

(b)
Fig. 4.2 The relationship between fatigue slip-hardening coefficient $\beta_s$ and fatigue cycle $N_s$ and fatigue load level $P_{\text{max}}$: 

$$\beta_s = C_d \cdot P_{\text{max}}^{M_d} \cdot \log N_s$$
Fig. 4.3 Flow chart of the numerical procedure for computing ECC fiber-bridging law $\sigma$ 

$$(\delta, N, \sigma_{\text{max}})$$
Fig. 4.4 Illustration of the components in stress intensity factor amplitude $\Delta K_{tip}$

Fig. 4.5 Tensile strain distribution along ECC beam under flexure, based on plane section assumption
Fig. 4.6 Flow chart of the numerical procedure for computing flexural fatigue life of ECC
Fig. 4.7 Modeling the post-fatigue $\sigma(\delta)$ of non-oil-coated PVA fiber ECC: (a) effect of load cycles $N$; (b) effect of load level $\sigma_{\text{max}}$. 
Fig. 4.8 Modeling the post-fatigue $\sigma(\delta)$ of oil-coated PVA fiber ECC: (a) effect of load cycles $N$; (b) effect of load level $\sigma_{\text{max}}$. 
Fig. 4.9 Effect of fatigue load on pseudo strain-hardening (PSH) index: (a) $PSH$ energy; 
(b) $PSH$ strength
Fig. 4.10 Comparison between ECC flexural fatigue life obtained from experiment and that from modeling results
CHAPTER 5 DEVELOPING HIGH-MOR ECC FOR FATIGUE RESISTANCE

For many concrete structures like bridge deck, pavement, and railway sleeper, flexural fatigue resistance must be considered in structural design. High concrete flexural strength, i.e. modulus of rupture (MOR), is important to extend the service life of these structures. Therefore, it is desirable to develop high-MOR feature to improve the fatigue resistance of ECC. In this chapter, high-MOR ECC with high fatigue resistance is developed under the guidance the ECC fatigue theory developed in Chapter 3 and 4; the new version of ECC is applied in designing precast pavement slab subject to flexural fatigue, as a demonstration of extending the infrastructure service life with high-MOR ECC. The material development and structural design are conducted under the guidance of Integrated Structures and Materials Design (ISMD) method (Li 2007).

The following three sections present the three standard steps defined in ISMD (Fig. 1.2), i.e. determining the target materials properties to achieve high MOR and flexural fatigue resistance (Step 1); developing ECC to achieve the determined target material properties with locally available ingredients (Step 2); and designing and evaluating structural performance of precast pavement slab using local ECC (Step 3).

5.1. ISMD Step 1: Determining the target material properties

In road engineering, hot-mixed asphalt (HMA) and Portland cement concrete are the two most used paving materials. Pavements made of these two materials are referred as flexible pavement and rigid pavement respectively. Generally, Portland cement concrete has higher resilient modulus (30-50 GPa, Neville 2011) than that of HMA (about 3.5 GPa at 21°C, Roberts et al. 1996); compared with HMA, Portland cement is less sensitive to environmental impact like the varying temperature or humidity. As a result, rigid pavements usually have longer designed service life (20-40 years) than that of flexible pavement (10-20 years, Fwa 2005). Despite the advantage of rigid pavement in durability, the majority of pavements in Singapore are still flexible pavements. The preference on flexible pavements mainly results from the ease of maintenance: HMA is easier to demolish; after paving the period for HMA hardening can be as short as 12
hours, while it takes no less than two weeks for concrete curing before road opening (Fwa 2005).

Precast rigid pavement slab is proposed as a solution to the problems associated with cast-in-place rigid pavement: it can be removed as slab units without difficult demolition; it can be fast installed in several hours, minimizing the road closure period. Novel techniques for manufacturing and installing precast slab have been developed around the world (Chang et al. 2004); the accelerated field test shows that precast pavements can achieve similar service life as the cast-in-place pavements at around 200 million equivalent single axle loads (EASL) (Kohler et al. 2009). However, some drawbacks still exist in precast slab: reinforcement must be installed in the slab to resist the self-weight loading during handling and transportation, prestressing of the reinforcement bars is needed if the slab dimension is large; the complicated construction process associated with prestressed reinforcement causes high cost in both labor and equipment.

ECC, as illustrated in Chapter 2, is a unique group of fiber-reinforced cementitious composites. The strain-hardening behavior and high ductility of ECC make it a promising replacement for conventional concrete in precast pavement slab. The high toughness and damage tolerance of ECC may be able to eliminate the requirement of reinforcement in precast slab.

In the current design of ECC precast slab, long service life is determined to be the target structural performance. Normally pavement deterioration is associated with the repeated flexural stress from traffics, and the service life is determined by the flexural resistance of the paving material. Fig.5.1 compares the flexural stress-fatigue life (S-N) relation of conventional concrete and ECC M45 (Qian 2007). It shows that at similar flexural stress level, ECC M45 can sustains fatigue cycles several orders higher than conventional concrete does. The high flexural fatigue strength of ECC results from its doubled MOR (11MPa) than that of conventional concrete (5 MPa), despite that the slope of S-N curves are similar. In the development of local ECC, it was expected that
the MOR can be further enhanced while the fatigue strength degradation rate, slope of the S-N curve, can be maintained.

5.1.1. Parametric study for high MOR

For strain-hardening cementitious composites (SHCC) like ECC, MOR is determined by uniaxial constitutive law of the material, i.e. tensile and compressive stress-strain relation. Maalej and Li (Maalej and Li 1994, Maalej and Li 1994) developed an analytical model to calculate the MOR of SHCC based on the tensile and compressive stress-strain relation. This model was adopted for parametric study here because it considers the tensile strain-hardening and ultra-high ductility of ECC; and it simplifies the tensile and compressive $\sigma$-$\varepsilon$ relation to bi-linear curves, as shown in Fig.5.2. It can be seen that the uniaxial $\sigma$-$\varepsilon$ relation can be described by six controlling variables, i.e. first cracking strength ($\sigma_{tc}$), first cracking strain ($\varepsilon_{tc}$), tensile strength ($\sigma_{tu}$), tensile ductility ($\varepsilon_{tu}$), compressive strength ($\sigma_{cp}$) and compressive strain capacity ($\varepsilon_{cp}$).

In current parametric study, tensile strength $\sigma_{tu}$ (=4.0, 5.0, and 6.0MPa), tensile ductility $\varepsilon_{tu}$ (=0.1%, 0.5%, 1%, 2%, 3%, 4%, and 5%), and compressive strength $\sigma_{cp}$ (=40MPa, 50MPa, and 60MPa) were chosen as independent variables, while the value of the other three terms were assumed as following:

$$\sigma_{tc} = 0.8\sigma_{tu} \quad \text{(Eqn.5.1)}$$

$$\varepsilon_{tc} = \frac{\sigma_{tc}}{E} = \frac{\sigma_{tc}}{(2\sigma_{cp}/\varepsilon_{cp})} \quad \text{(Eqn.5.2)}$$

$$\varepsilon_{cp} = 0.45\% \quad \text{(Eqn.5.3)}$$

where $E$ is the young’s modulus of the material for both tension and compression in the first linear stage. The relations assumed in Eqn.5.1-5.3 were intended for the simplicity in calculation, and they are reasonable given the previous research on ECC (Li et al. 2001, Wang and Li 2007, Yang et al. 2007).

Given all the six parameters the stress distribution along the depth can be determined under the plane section assumption, and the force and moment equilibrium can be
expressed in equations. The maximum moment $M_u$ is calculated by solving the equations when the strain on the tensile bottom reaching $\varepsilon_{tu}$ (tensile failure), or the strain on the compressive top reaching $\varepsilon_{cp}$, (compressive failure). MOR can be calculated as:

$$MOR = \frac{6M_u}{BH^2}$$

(Eqn.5.4)

Where $B$ and $H$ are beam depth and width respectively. The details of the MOR calculation are given in Appendix A.

5.1.2. Parametric study results

Three independent variables create 63 different combinations of $\sigma_{tu}$, $\varepsilon_{tu}$, and $\sigma_{cp}$. The calculated $M_u$ and MOR for all the combinations are given in Appendix B. The effect of $\sigma_{tu}$, $\varepsilon_{tu}$, and $\sigma_{cp}$ on MOR are shown in Fig.5.3. It can be seen that some combinations do not achieve relatively high MOR over 11 MPa. The MOR of ECC increases almost linearly with tensile strength $\sigma_{tu}$, in spite of the tensile ductility $\varepsilon_{tu}$ and compressive strength $\sigma_{cp}$. It indicates that high tensile strength is a key factor to enhance MOR of ECC.

Fig.5.4 shows the effect of increasing tensile ductility on MOR. Despite the difference in tensile and compressive strength, all the curves in Fig.5.4 show the similar trend: in the low ductility range below 1%, MOR increases rapidly; then it enters into a plateau in the medium ductility range from 1% to the transition line that separate the ‘tensile failure zone’ and ‘compressive failure zone’, where the MOR increase is much slower; finally it moves beyond the transition line and enters the ‘compressive failure zone’.

It is interesting to know that if the tensile ductility of ECC is very high, failure will first occur at the compressive top rather at the tensile bottom of the ECC beam. Fig.5.5 illustrates the stress distribution along the depth of conventional concrete beam and ECC beam. It shows that at large curvature the stress at the compressive top of ECC beam could be very high. In Fig.5.4, it is noticed that the transition line shifts to the
right with the increase of compressive, which indicates that sufficient compressive strength is also important to maintain high MOR.

The results of parametric study show that high tensile strength, moderate tensile ductility (about 1-3%), and sufficient compressive strength are determined to be the target material properties as in the ISMD.

5.2. ISMD Step 2: Developing local ECC with the target material properties

5.2.1. Materials and methodology

This section presents the development of local ECC to achieve the target material properties determined in the previous section. ECC M45 satisfies the requirements on material property, but the raw ingredients used, such as coal fly ash and silica sand, are strictly controlled in Singapore as they are classified as hazardous substance. In current study, ground granulated blast-furnace slag (GGBS) and river sands, which are available in Singapore, were used as the replacement for coal fly ash and silica sand respectively. Type I ordinary Portland cement (OPC CEM I 42.5), GGBS provided by Engro Co. Ltd (Elementary composition as in Table 5.1), river sands sieved to below 600 μm in particle size (as shown in Fig.5.6), and PVA fibers from Kuraray Co. Ltd (properties as in Table 5.2) were used to manufacture local ECC. The parametric studies in Chapter 4 suggested that oil-treatment on the surface of PVA fiber is important to decelerate the degradation of ECC components (fiber and fiber-matrix interface) and sustain the multiple-cracking behavior under fatigue loading. Therefore, the PVA fibers used in this study are coated with oil of 1.2% by weight. Superplasticizer (SP) was also included to ensure the workability of ECC. Changing the content of supplementary cementitious materials (SCM) like GGBS can greatly alter the mechanical properties of ECC (Kim et al. 2007, Qian et al. 2009, Qian et al. 2010, Zhou et al. 2010). In current study, three different mix designs, which vary in the OPC replacement ratio by GGBS, were investigated. The details are shown in Table 5.3.

The solid materials, i.e. cement, GGBS, river sands were first mixed with a three-gear Planetary (TM) mixer for two minutes. Water and super-plasticizer were slowly added
into the mixture at low mixing speed within one minute, followed by medium mixing speed for two minutes to assure the viscosity of the mixture. PVA fibers were then slowly added into the mixture at low mixing speed within one minute, followed by medium mixing speed for three minutes. The fresh mixture was cast into moulds with different dimensions for specific experimental purposes. For monotonic uniaxial tensile tests, compressive tests, four-point flexural tests, and fatigue four-point flexural tests (GGBS60 only), the dog-bone specimens (as recommended by (Kim et al. 2007)), cube specimens (50×50×50 mm), coupon specimens (300×75×12 mm, depth=12 mm), and prism specimens (280×70×50 mm, depth=50 mm) were used respectively. The specimens were demolded after 24 hours and cured in lab (20 °C) until testing date (age of 28 days).

A serial monotonic test, including uniaxial tensile test, compressive test, and four-point flexural test, were conducted to characterize local ECC’s mechanical properties. Displacement-controlled uniaxial tensile tests were conducted on dog-bone specimens with an Instron 5569 UTM, where two LVDTs were used to determine the extension of the middle uniform part of specimens. The loading rate of tensile test was 0.04mm/min. Load-controlled compressive tests were used to determine the compressive strength, where the loading rate was 0.25 MPa/s. Displacement-controlled four-point flexural tests were conducted on coupon specimens with the same Instron UTM, where the three spans were all 80 mm. The loading rate of flexural test was 0.7mm/min.

Among the three mix designs, GGBS60 with highest MOR in monotonic flexural test, was selected to verify local ECC’s resistance against fatigue loads. In order to determine the full S-N curve of GGBS, both monotonic and flexural fatigue tests were included. Prisms instead of coupon were used for these tests to control the magnitude of deflection, so that it would not exceed the testing machine’s capacity in fatigue tests. The monotonic tests were displacement-controlled and conducted in the same scheme as coupon specimens except for a lower loading rate (0.2 mm/min). The fatigue tests were load-controlled and the loading scheme is shown in Fig.5.7: the minimum load was 0.20 MOR that was determined from monotonic tests, while the maximum load
ranging from 0.85 MOR to 0.95 MOR; before the start of sine-wave loading at 8 Hz, the load was increased monotonically to the medium level (average of maximum and minimum load) within two minutes. The tests were stopped once the deflection reached 10 mm, at which the specimens would certainly fail according to the results of monotonic tests. The cycle numbers at failure were recorded for drawing the S-N curve.

5.2.2. Mechanical test results

Monotonic test results of GGBS0, 30, and 60 are summarized in Table 5.4. The values in the table are the average and standard deviation of at least three specimens. Equivalent flexural stress-deflection curves are shown in Fig.5.8, where the equivalent stress \( \sigma \) was calculated with Eqn.5.5.

\[
\sigma = \frac{6M}{bh^2} \quad \text{(Eqn.5.5)}
\]

where \( M \), \( b \), and \( h \) are the moment at the middle span, width and thickness of the coupon specimen, respectively.

It is shown in Table 5.4 and Fig.5.8 that the content of GGBS had appreciable impact on mechanical properties of ECC including MOR. The tensile strength and compressive strength increase with the inclusion of higher dosage of GGBS. This may be attributed to that the GGBS in matrix would consume the Ca(OH)\(_2\), the weakest part of cement matrix, forming stronger C-S-H structures (Neville 2011). GGBS60 exhibited the highest MOR at 12.90 MPa, resulting from the highest tensile strength at 4.88 MPa, moderate tensile ductility at 1.26%, and sufficient compressive strength at 70.8 MPa.

With the highest MOR, GGBS60 was chosen for flexural fatigue tests. Fig.5.9 shows the flexural S-N curve of conventional concrete (Oh 1991), ECC M45 (Qian 2007), and GGBS 60 (local ECC). It is clearly shown that at the same fatigue life, the fatigue strength of ECC M45 is about twice that of conventional concrete, and local ECC further enhances the fatigue strength. Local ECC satisfies the target fatigue performance addressed in Step 1 of ISMD, i.e. MOR higher than 10 MPa and similar
fatigue strength decreasing rate with ECC M45. During fatigue load, multiple cracks gradually developed and the fiber-bridging sustained the load. Due to the fiber-bridging degradation with load repetition, it eventually lost its load capacity and broke into half. It can be seen in Fig.5.9 that the improved fatigue strength of local ECC not only resulted from aforementioned higher MOR, but also slower fiber-bridging degradation as shown by the flatter slope of the S-N curve. The fatigue-induced fiber-bridging degradation is thoroughly investigated and discussed in Chapter 5 and 6.

5.3. ISMD Step 3: designing and evaluating the structural performance
This section presents the work on designing precast pavement slab with locally developed ECC (GGBS60) and evaluating its structural performance. In current study, the slab thickness is set as the design parameter. The maximum flexural stress in the ECC slab under vehicle loading and self-weight loading is determined with finite element program ANSYS. The structural safety during pavement service and slab installation is evaluated based on the numerical calculation.

5.3.1. Structural performance under vehicle loading
Finite element program ANSYS was adopted to calculate the flexural stress (i.e. the tensile stress at bottom) in the ECC slab. ANSYS provides the ‘Cast Iron Plasticity’ material module which enable separately defining the tensile and compressive $\sigma$-$\varepsilon$ curve of ECC. Though the module originally designed for cast iron which has different yielding strength under tensile and compressive load, it is also suitable for representing material behavior of ECC. Table 5.5 shows the local ECC (GGBS60) properties as input to the material module, where the tensile behavior is defined as a triple-linear curve, the compressive a bi-linear curve. Fig.5.10 compares the equivalent flexural stress-deflection relation obtained from experimental test and numerical calculation. It can be seen that flexural behavior of local ECC can be accurately predicted by using this material module.

Fig.5.11 illustrates the modeling details of ECC pavement slab (5.0×3.6 m) and the granite base beneath the slab. The thickness of ECC slab was varied from 50 mm to
250 mm. The modulus of granite base and soil subgrade was 250 MPa and 50 MPa respectively. Two loading scenarios, extreme loading at the middle point of the long edge of the slab and fatigue loading at the slab center, were considered in current design. As suggested by PCA guideline (Fwa 2005), a vertical load of 65kN and 40kN, evenly distributed in a circle comparable to typical tyre contact area, were applied for these two loading cases respectively.

The variation of maximum flexural stress under extreme and fatigue loading cases with slab thickness is given in Fig.5.12 together with the flexural S-N curve of local ECC (GGBS60). It can be seen that the maximum flexural stress in the ECC slab increases with the decreasing slab thickness. For the extreme loading case, the flexural stress is well below the ECC MOR (~13MPa) even at slab thickness of 50 mm.

Fatigue failure is the limiting factor for the service life of rigid pavement. In current design, it is expected that the local ECC with high fatigue resistance can extend the pavement service life from 20 years to 40 years. In Singapore, the average daily traffic on a heavy-duty expressway is typically 50,000-60,000 vehicles, which leads to average ESAL (equivalent single axle load = 80 kN as suggested by AASHTO) of 4.5-7.5 million per year. ESAL to ECC and concrete fatigue failure would be 360 million and 120 million respectively. For ECC slab, at slab thickness of 50 mm, the maximum flexural stress is only about 4.5 MPa even at 50 mm, while the flexural fatigue strength for 40-year service life is about 9.5 MPa. For normal concrete, the slab thickness required for 20-year fatigue life is about 150 mm.

In addition to flexural stress in the slab, it is necessary to limit the maximum vertical compressive stress in the base/subgrade below the load capacity. Fig.5.13 illustrates the relation between the maximum vertical compressive stress in base/subgrade and slab thickness under the extreme loading scenario. It can be seen that 50 mm ECC slab will introduce very high vertical compressive stress into the granite base and soil subgrade. It is necessary to increase the slab thickness to above 92 mm to make sure the
granite/soil strength shall not be exceeded. The high vertical stress may be attributed to the lower stiffness of ECC under cracked state, which transfer more concentrated load to the base/subgrade.

The structural performance evaluation suggests that replacing normal concrete pavement with ECC slab can greatly extend the fatigue life of the pavement. As a result, fatigue failure is no longer the factor limiting factor to design thinner pavement; instead, the high vertical stress transferred by ECC slab to base/subgrade is the new limiting factor.

5.3.2. Slab safety under self-weight loading
For precast slab, the flexural stress due to self-weight of the slab during installation is a concern for the slab design. For the slab dimensions used in current study (5.0×3.6), four lifting points (one meter from both long and short side) are adopted. The flexural stress during lifting by self-weight is computed with FEM and shown in Fig.5.14. While the stress on the top is much higher than that at the bottom, the flexural stress is much lower than the first cracking strength of ECC, despite the slab thickness. It can be seen that the maximum flexural stress introduced by self-weight is much lower than the first cracking strength of ECC (~3.5MPa), ensuring the safety during slab installation.

5.4. Conclusions
In this chapter, under the guidance of ECC fatigue theory developed in Chapter 3 and 4, ECC mix design is tailored to achieve high flexural strength (MOR) and fatigue resistance for the application of precast pavement slab. The material tailoring is guided by the integrated structures and materials design (ISMD) approach: first the target constitutive material properties to engage high MOR are determined; second a new version of ECC (GGBS60) is practically developed under the guidance of fatigue-theory, to satisfy the requirement of target properties; finally the structural performance of precast pavement slab made with the new version of ECC is evaluated. Following conclusions are drawn based on the study:
- High tensile strength, medium tensile ductility, and sufficient compressive strength is determined as the material properties that produce high MOR of ECC. In order to achieve high MOR, the tensile ductility of ECC should be higher than 1%.
- Increasing GGBS content (within the range of 0% to 60% cement replacement) enhances the tensile strength and tensile ductility of ECC, as a result; GGBS60 produces the highest MOR (~13 MPa) and flexural fatigue strength.
- In the pavement design, fatigue failure is no longer the limiting factor to minimize the pavement slab thickness when replacing normal concrete with high-MOR ECC. The slab thickness can be reduced from 150 mm to 92 mm, and the service life of pavement can be reduced from 20 years to 40 years.
References


Table 5.1 Elementary composition of GGBS in current study

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
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<tr>
<td>CaO</td>
<td>39.43%</td>
</tr>
<tr>
<td>SiO₂</td>
<td>31.19%</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>13.41%</td>
</tr>
<tr>
<td>MgO</td>
<td>9.32%</td>
</tr>
<tr>
<td>SO₃</td>
<td>4.25%</td>
</tr>
<tr>
<td>Others</td>
<td>2.40%</td>
</tr>
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</table>

Table 5.2 Properties of PVA fibers in current study

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<table>
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<tr>
<td>Length</td>
<td>12mm</td>
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<tr>
<td>Diameter</td>
<td>39μm</td>
</tr>
<tr>
<td>Nominal tensile strength</td>
<td>1,600MPa</td>
</tr>
<tr>
<td>Density</td>
<td>1,300kg/m³</td>
</tr>
<tr>
<td>Surface oil coating by weight</td>
<td>1.2%</td>
</tr>
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</table>

Table 5.3 Proportions of mix design in current study

<table>
<thead>
<tr>
<th>Test No.</th>
<th>OPC (kg/m³)</th>
<th>GGBS (kg/m³)</th>
<th>Sand (kg/m³)</th>
<th>Water (kg/m³)</th>
<th>PVA fiber (kg/m³)</th>
<th>SP (L/m³)</th>
<th>GGBS/binder ratio</th>
<th>W/b ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>GGBS0</td>
<td>1411</td>
<td>0</td>
<td>282</td>
<td>423</td>
<td>26</td>
<td>3.6</td>
<td>0%</td>
<td>0.30</td>
</tr>
<tr>
<td>GGBS30</td>
<td>976</td>
<td>418</td>
<td>279</td>
<td>418</td>
<td>26</td>
<td>3.0</td>
<td>30%</td>
<td>0.30</td>
</tr>
<tr>
<td>GGBS60</td>
<td>551</td>
<td>827</td>
<td>276</td>
<td>414</td>
<td>26</td>
<td>2.4</td>
<td>60%</td>
<td>0.30</td>
</tr>
</tbody>
</table>
Table 5.4 Summary of monotonic test results

<table>
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<tr>
<th>Test No.</th>
<th>Tension strength (MPa)</th>
<th>Tensile ductility (%)</th>
<th>Compression strength (MPa)</th>
<th>MOR (MPa)</th>
<th>Deflection at failure (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GGBS0</td>
<td>4.25±0.48</td>
<td>1.12±0.36</td>
<td>63.1±3.8</td>
<td>10.77±0.98</td>
<td>7.57±2.51</td>
</tr>
<tr>
<td>GGBS30</td>
<td>4.40±0.19</td>
<td>1.05±0.07</td>
<td>66.3±3.7</td>
<td>10.56±1.80</td>
<td>9.58±3.31</td>
</tr>
<tr>
<td>GGBS60</td>
<td>4.88±0.62</td>
<td>1.26±0.28</td>
<td>70.8±1.8</td>
<td>12.90±0.66</td>
<td>18.93±1.19</td>
</tr>
</tbody>
</table>

Table 5.5 Local ECC properties as input for FEM analysis

<table>
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<tr>
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<th>End point of the first slope</th>
<th>End point of the second slope</th>
<th>Slope of the third slope</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ε (%)</td>
<td>σ (MPa)</td>
<td>ε (%)</td>
</tr>
<tr>
<td>Tension</td>
<td>0.0125%</td>
<td>2.50</td>
<td>0.25%</td>
</tr>
<tr>
<td>Compression</td>
<td>0.2%</td>
<td>40</td>
<td>0.25%</td>
</tr>
</tbody>
</table>
Fig. 5.1. Flexural S-N curve of concrete and ECC

Fig. 5.2. The by-linear tensile (upper) and compressive (lower) stress-strain curves assumed in Maalej and Li’s analytical model
Fig. 5.3 ECC MOR as function of $\sigma_{tu}$, $e_{tu}$, and $\sigma_{cp}$.
Fig. 5.4 ECC MOR as function of tensile ductility $\varepsilon_{tu}$
Fig. 5.5 Sectional stress distribution of conventional concrete beam (a) and ECC beam (b) approaching flexural failure.

Fig. 5.6 Particle size distribution of river sand in current study.
Fig. 5.7 Loading scheme of fatigue test in current study
Fig. 5.8 Equivalent flexural stress-deflection curves in monotonic flexural tests
Fig. 5.9 Fatigue flexural test results: S-N curve of local ECC and its comparison with conventional concrete, and ECC M45.
Fig. 5.10 Flexural behavior of ECC: experimental results vs. FEM results (in courtesy of Nguyen)
Fig. 5.11 Illustration of the ECC pavement FEM modeling (in courtesy of Nguyen)

Fig. 5.12 Fatigue life-flexural stress-slab thickness relation of local ECC (GGBS60) and normal concrete (in courtesy of Nguyen)
Fig. 5.13 The relation between the maximum vertical compressive stress in pavement base/subgrade and slab thickness (in courtesy of Nguyen)

Fig. 5.14 The maximum flexural stress due to the self-weight of ECC slab during lifting (in courtesy of Nguyen)
CHAPTER 6 ENGAGING ROBUST SELF-HEALING IN ECC FOR FATIGUE RESISTANCE

ECC is a group of cementitious materials with potential to heal the crack without human interference. Robust self-healing may extend the fatigue life of the material, by preventing the crack propagation under fatigue loading. In this chapter, two research tasks are completed to study the feasibility of applying the self-healing behavior to improve the fatigue resistance of ECC: in Section 6.1, the effect of slag content, crack width, and alkalinity (as an important group of ingredient for ECC mix design) on self-healing is holistically studied to realize the optimal mix design and environmental conditions for self-healing; in Section 6.2, the improvement of fatigue resistance through self-healing is experimentally verified with the optimal ECC mix design and environmental conditioning.

6.1. Optimizing chemical composition of ECC for robust self-healing

Infrastructures contribute to public transportation, energy harvesting, and commercial activities of modern society. They are vital to the well-being of a nation and the life quality of its citizens. Concrete infrastructures like concrete bridges and pavement, were designed to serve the public for decades, however, many of them fail to provide good service as a result of the fast deterioration. In most developed countries, concrete maintenance and rehabilitation cost about 50% of the outlay on infrastructures (Li 2004).

Deterioration of concrete infrastructure, such as corrosion of reinforcing steel, is associated with cracks. Reinforced concrete members could crack under structural loading, but more often due to constrained shrinkage/thermal deformations, which are practically inevitable (Wittmann 2002). Cracks not only reduce the load capacity of concrete members, but also create pathways for aggressive ions to penetrate in, which accelerates concrete deterioration. It is highly desirable to engage self-healing in concrete.
ECC is a group of fiber-reinforced strain-hardening cementitious composites with self-controlled crack width below 100 μm (Li, Wang et al. 2001). In the presence of water, the fine cracks in ECC can seal without any external intervention; the healing products generated in the cracks are composed of mainly CaCO₃, through the carbonation of free Ca²⁺, together with some continued hydration products (Yang et al. 2009, Kan et al. 2010, Yang et al. 2011). The mechanical properties of ECC, i.e. strength and stiffness, can be restored and the healing degree is highly affected by crack width. Yang et al. (2009) showed that the maximum allowable crack width to engage complete mechanical recovery was around 50 μm for ECC M45. They also reported that crack beyond 150 μm did not show any healing. Partial recovery was observed for cracks between 50 to 150 μm.

Besides crack width, chemical composition also affects the healing degree of ECC. Supplementary cementitious materials (SCM), such as fly ash, silica fume and/or ground granulated blast slag (GGBS), are an important group of ECC ingredients. High content of SCM can improve the ductility and greenness of ECC. However, the effects of SCM content on self-healing are seldom studied.

In this study, the effects of GGBS content on ECC self-healing were studied. Self-healing was experimentally determined for a wide range of GGBS content (0% to 60% cement replacement), crack width (0 to 300 μm) and environmental alkalinity (water and NaOH solution, pH = 13). In Chapter 5 GGCBS was used to engage high MOR in ECC for longer fatigue life; the result indicates that all of the three levels can produce relatively high MOR. The experimental work in this section can be treated as a further tailoring task to engage robust self-healing for even longer fatigue life. As a result, the same GGBS content levels were used in this section. High-pH conditioning was also involved to check the feasibility of enhancing self-healing capability by alkali-activation of slag (Neville 2011). The research outcome sheds light on designing robust self-healing behavior into ECC, which is a necessary step toward applying self-healing to improve fatigue resistance.
6.1.1. **Experimental program**

ECC specimens with different slag content were prepared for self-healing experiment. The specimens were pre-loaded to form a single crack with pre-determined crack width up to 300 μm, followed by conditioning the pre-cracked specimens in different environmental exposures. Self-healing performance was evaluated based on crack width reduction, resonant frequency recovery, and microstructure analysis. Detailed experimental program is given in following sections.

**ECC specimen preparation**

Type I Portland cement (CEM I 52.5 N), GGBS, fine aggregates, and polyvinyl-alcohol (PVA) fibers, superplasticizer (SP) and tap water were used as ingredients for ECC specimen preparation. The GGBS (particle size less than 100μm) was produced by quenching and drying the molten iron slag from normal blast-furnace. The elementary composition of GGBS is shown in Table 6.1. The fine aggregates (particle size less than 600μm, Fig. 3.6) were obtained by sieving normal river sand. The PVA fibers are of 12 mm in length, 39 μm in diameter, and 1,600MPa in the nominal tensile strength.

Three levels of slag content, i.e. 0%, 30%, and 60% cement replacement, were studied as shown in Table 6.2. The same mix designs were used in Chapter 5. The water-to-binder (cement and GGBS) ratio was fixed at 0.30, while the GGBS-to-binder ratio ranged from 0% to 60%. Since the crack width in ECC is self-controlled, lower fiber dosage was adopted in mixes 4 to 6 in order to obtain larger crack width beyond 100μm.

Cement, GGBS, and fine aggregates were dry mixed first with a three-gear planetary mixer for two minutes. Water and SP were slowly added into the mixture at low mixing speed within one minute, followed by medium mixing speed for another two minutes to achieve the required rheology of the fresh mortar. PVA fibers were then slowly added into the mixture at low mixing speed within one minute, followed by medium mixing speed for three minutes to ensure good fiber dispersion.
The fresh mixture was cast into molds of dog-bone specimens (Fig.6.1a) for uniaxial tensile tests (Fig.6.1b) and self-healing tests, and cube specimens (50x50x50 mm for compressive tests. The specimens were de-molded after one-day and were cured in laboratory air (20°C) until the pre-determined age (28 days for the mechanical tests and 40 days for the self-healing tests). Uniaxial compressive and tensile tests were conducted for ECC specimens and the test results are given in Table 5.4 in Chapter 5.

**Self-healing tests**

At the age of 40 days, dog-bone specimens were pre-cracked using the same set-up (Fig.6.1b) as uniaxial tensile test. A single crack was introduced on the specimen by tensile load. At 2% fiber content, i.e. GGBS0 to 60, the crack width of ECC is self-controlled and it would not be possible to extend crack width beyond 100μm without generating a second crack. As an alternative, GGBS0a to 60a with lower fiber content were used to produce larger single crack with crack width up to 300μm.

Two different wet/dry cycles were used as conditioning regimes to engage self-healing. The first type of wet/dry cycle consisted of submerging the specimens in water (20°C) for one day, followed by exposing the specimens in air (20°C) for another day, which was suggested by Yang et al. (2009). The second type of wet/dry cycle consisted of one day in NaOH solution (pH=13, 20°C) followed by another day in air (20°C). This conditioning regime was intent to encourage slag hydration through alkali activation, which may promote the formation of C-S-H as self-healing products. Saturated Ca(OH)₂ solution, which is often used to activate slag, was not used here, because it introduce additional Ca²⁺ that contributes to the formation of CaCO₃ as self-healing products.

Crack width reduction on the specimen surface was measured as a direct assessment of self-healing. Images of all pre-cracked specimens before and after conditioning were taken at magnification of 210× by Nikon DS-Fi2 high resolution camera. High-magnification images helped to monitor changes of crack width. For each specimen, three to six locations were measured and the surface crack width was reported.
Resonant frequency (RF) can be used to determine the stiffness and to characterize damage in concrete. In this study, recovery of transverse resonant frequency (TRF) of single-cracked dog-bone ECC specimen was adopted to characterize the stiffness recovery as an indicator of self-healing. Similar method was also used in previous tests (Yang et al. 2009, Yang et al. 2011). TRF of all specimens were measured before self-healing conditioning and at the end of every conditioning cycle. Specifically, the TRF measurement adopted the impact method suggested by ASTM C215 (Standard Test Method for Fundamental Transverse, Longitudinal, and Torsional Resonant Frequency of Concrete Specimens). A resonant frequency meter was used to detect and to record the TRF. The normalized TRF, i.e. the ratio of TRF of pre-cracked specimen to TRF of uncracked specimens (average of three) with the same conditioning regime, was calculated and reported. For each combination of slag content and healing conditionings, three uncracked virgin specimens were included as control group for normalization because TRF also increased after conditioning due to further hydration of matrix, the effect of which must be excluded from true healing-induced TRF recovery.

Characterization of healing products was carried out in addition to the crack width and TRF measurement. After conditioning, the morphology of healing products in cracks was observed with field emission scanning electron microscope (FE-SEM, JSM-7600F). Energy-dispersive X-ray spectroscopy (EDX, Oxford X-Max 80mm²) was used to determine the chemical composition of self-healing products. All the SEM/EDX samples were obtained from the internal part of specimen.

6.1.2. Experimental results

Crack width reduction

It was observed that crack width reduced with conditioning cycles as crystal-like substance grew in the crack. In current study, the crack width reduction in pre-cracked ECC specimens is measured and used as a direct assessment of self-healing at different slag content under the two conditioning regimes.
Fig. 6.2 and 6.3 show the surface crack width of GGBS0/0a, GGBS30/30a, and GGBS60/60a specimens before and after 14 water/dry and NaOH/dry conditioning cycles, respectively. A 45-degree reference line representing zero-healing (blue dash) was plotted. Data points below the reference line indicate reduced crack width after healing. A linear trend line (red solid) was plotted to fit these data points. As can be seen, for all the three groups, surface crack width was reduced under water/dry or NaOH/dry conditioning cycles. While crack width reduction on the surface was measured in all specimens with crack width up to around 300 μm, only very fine cracks (<30μm) could possibly be completely sealed.

In order to quantitatively compare the degree of self-healing of different ECC groups (GGBS0, 30, and 60) and under different conditioning regimes, crack width reduction ratio, as defined by Eqn.6.1, is calculated. In Eqn.6.1, \( w_0 \) and \( w \) represent the crack width before and after conditioning, respectively, and \( w \) is determined based on the fitted curve (red solid line) in Fig. 6.2 and 6.3.

\[
\text{Crack width reduction ratio (\%) = } \frac{(w_0 - w)}{w_0}
\]  
(Eqn.6.1)

Fig. 6.4 shows the crack width reduction ratio of pre-crack ECC specimens at different slag content, crack width and conditioning environment. It can be seen that slag content greatly affected the reduction of crack width for both conditioning regimes. Specifically, GGBS0 showed the least crack width reduction while GGBS30 exhibited the highest potential to engage self-healing. In addition, specimens exposed to the water/dry conditioning regime showed more pronounced crack width reduction as compared to the NaOH/dry conditioning regime. For all slag contents under both conditioning regimes, smaller cracks exhibited higher crack width reduction ratio.

Resonant frequency recovery

Fig. 6.5 and 6.6 show the normalized TRF of pre-cracked ECC specimens before (blue cross) and after (red circle) 14 water/dry or NaOH/dry conditioning cycles, respectively. As can be seen, pre-cracking greatly reduced resonant frequency of ECC specimens
(blue cross) and the reduction amplified with increasing crack width, indicating more damage. After conditioning, the resonant frequency recovered and in some cases even to its original level. In this study, considering the variation of specimen quality, normalized TRF recovered to 97% was accounted as complete recovery.

As can be seen in Fig.6.5, GGBS30 showed the highest potential to engage self-healing in terms of TRF recovery as compared to GGBS0 and GGBS60. It was observed that under water/dry conditioning environment, GGBS30 allowed complete recovery of TRF as long as the crack width is less than around 90μm while the maximum allowable crack width for GGBS60 and GGBS0 were around 60μm and 40μm, respectively. In Fig.6.6, under NaOH/dry conditioning, GGBS30 again showed the most significant TRF recovery with a maximum allowable crack width around 80μm while that for GGBS60 and GGBS0 were both around 40μm. Comparing Fig.6.5 and 6.6, it is noticed that specimens exposed to the water/dry conditioning regime showed more pronounced resonant frequency recovery as compared to the NaOH/dry conditioning regime.

SEM and EDX

Fig.6.7a shows the SEM image of a healed crack and Fig.6.7b and 6.7c illustrates the typical morphology of the healing products. It can be seen that healing products, distinct from the matrix, grew in the crack and almost fill the 100μm gap. The healing products are composed of irregularly precipitated crystal-like particles. While the large particles (≥ 10μm, Fig.6.7a) fill the major portion of the gap, smaller particles (≤ 10μm, Fig.6.7b) can be found in the void between the large particles and crack surface. In Fig.6.7c, it is seen that the crystal-like particles grow from very small precipitates, several hundreds of nanometers in size. It is also observed that healing products precipitate on the PVA fiber as shown in Fig.6.8. Fiber-bridging might very likely facilitate the precipitation of healing products and promotes self-healing in ECC.

Chemical composition of self-healing products was determined with EDX for ECC samples at different slag content under conditioning regime. At least three locations were analyzed for each group and the average values are reported in Table 6.3. As can
be seen, the self-healing products are mainly composed of calcium, silicon, and carbon. It is also noticed that self-healing products in samples subjected to the water/dry conditioning cycles has much higher calcium-to-silicon (Ca/Si) ratio than that in samples subjected to the NaOH/dry conditioning cycles. The Ca/Si ratio of C-S-H is known to be around 2. For example, Huang et al. (2011, 2013, and 2014) reported that the C-S-H found in healed cracks has a Ca/Si ratio of 2 to 3 for Type I Portland cement system and a lower Ca/Si ratio of 0.9 to 1.7 for slag-blended cement system (66% cement replacement). It is therefore believed that CaCO₃ from carbonation is the dominant healing products in samples subjected to the water/dry conditioning regime while a mixture of CaCO₃ and C-S-H from continued hydration represents the major healing products in samples subjected to the NaOH/dry conditioning regime.

6.1.3. Discussion

Preceding sections show that slag content greatly affects the self-healing of ECC. Compared with GGBS0 and GGBS60, GGBS30 exhibits the most crack width reduction (Fig.6.4) and resonant frequency recovery (Fig.6.5 and 6.6). The precipitation of CaCO₃ highly depends on the amount of free Ca²⁺ leached out from matrix in the wet conditioning. Leachability of calcium ion is related to the concentration of the Ca²⁺ in the pore solution in matrix, where Ca(OH)₂ is dissolved (Neville 2011). The concentration of Ca²⁺ in the pore solution depends on two important factors: 1) the amount of Ca(OH)₂ available in matrix and 2) the alkalinity of the pore solution. Previous study on hydration of slag-blended cement indicated that replacing cement with slag consumes Ca(OH)₂ and lowers the pH of pore solution (Chen 2006). While GGBS30 may have lower Ca(OH)₂ content in matrix as compared with GGBS0, lower pH in the pore solution of GGBS30 greatly promotes the dissolution of Ca(OH)₂ (Mindess et al. 2003), resulting in higher overall free Ca²⁺ concentration in the pore solution of GGBS30. At even higher GGBS content, i.e. GGBS60, less Ca(OH)₂ is produced from cement hydration and much of the Ca(OH)₂ is consumed by slag (Kolani et al. 2012). Therefore, less free Ca²⁺ is available in the pore solution of GGBS60 compared with GGBS30.
As discussed above, CaCO$_3$ is the dominant healing products for samples subjected to the water/dry cycle while a mixture of CaCO$_3$ and C-S-H represents the major healing products for samples subjected to the NaOH/dry cycle. NaOH/dry conditioning indeed promotes slag hydration as more C-S-H was identified in the healing products as shown in Table 6.3. However, the general observation concluded that specimens subjected to the water/dry cycles show more pronounced self-healing than specimens subjected to the NaOH/dry cycles, in terms of both crack width reduction and resonant frequency recovery as shown in Fig.6.4 to 6.6. This implies that NaOH/dry conditioning regime, while encouraging slag hydration, may suppress the formation of CaCO$_3$ precipitates in cracks as healing products. This may be attributed to the fact that the insolubility of CaCO$_3$ peaks at pH of 9.8 (Edvardsen 1999) and therefore the increased alkalinity under NaOH conditioning (pH=13) suppresses CaCO$_3$ precipitation in cracks. In addition, CaCO$_3$ precipitation through carbonation is more effective than the formation of C-S-H through alkali-activated slag hydration to reduce crack width and to regain resonant frequency.

Based on the discussion above, self-healing in ECC depends not only on the crack width but also on the chemical composition of the matrix and conditioning environment. The required crack width to engage robust self-healing can be different for different chemical composition of the matrix. Fig.6.9 illustrates this concept by plotting the allowable crack width to engage complete healing at different slag content in ECC specimens. The three data points in Fig.6.9 were the allowable crack widths to engage complete resonant frequency recovery for samples subjected to the water/dry conditioning cycles (Fig.6.5). The fitted line represents the envelope to engage complete healing in which the allowable crack width is a function of slag content. As can be seen, the allowable crack width to engage complete resonant frequency recovery increases with increasing slag content and peaks at about 90 µm at 35% of cement being replaced by slag. The addition of slag up to 35% to 40% enables the healing of larger crack. After that, the allowable crack width decreases when more cement is replaced by slag. Materials in the area below the envelope shall exhibit high potential
for complete healing. Materials above the envelope, on the other hand, shall exhibit partial healing and even no healing if materials locate far away from the curve. Qian et al. (2009) investigated self-healing behavior of ECC with about 55% cement replaced by slag and concluded that only cracks below 60μm in width can achieve complete healing. This observation further confirms the validity of Fig.6.9.

Design of self-healing in ECC should consider both the physical conditions (crack width) and the chemical environment (chemical composition of matrix). Fig.6.9 therefore can be used as a design chart for self-healing ECC subjected to the water/dry conditioning cycles. To engage complete healing, the crack width in ECC should be tailored to be less than the allowable crack width at a given slag content and conditioning environment. This approach indeed provides insights for ingredients selection and component tailoring to engage robust self-healing in ECC.

**6.2. Effect of self-healing on fatigue resistance of ECC**

In practical application like pavement, ECC may experience unexpected extreme loading, which may cause multiple cracks in the matrix. The presence of cracks reduces the overall stiffness, as fiber-bridging across the crack is weaker than the intact matrix. Yang et al. (2009) experimentally confirms that the self-healing of ECC can effectively recover the tensile stiffness loss, in some cases even to 100% of control specimens. Qian et al. (2009, 2010) observed appreciable recovery of flexural stiffness in cracked ECC that experienced wet-dry conditioning. Such healing-induced stiffness recovery indicates that the fiber-bridging in ECC are strengthened during the healing conditioning, very likely due to the formation of healing products.

In the study of FRC fatigue, it has been known that fiber-bridging across a crack can decelerate the crack propagation in the concrete matrix: the stronger fiber-bridging, the smaller rate of crack propagation (Li and Matsumoto 1998). It is therefore expected that self-healing may extend the fatigue life of ECC, as it strengthens the fiber-bridging and may decelerate the crack propagation.
This section presents the work on the effect of self-healing on the flexural fatigue life of ECC. The optimal cement replacement ratio by GGBS of 30%, as advised by the testing results in the last section, is chosen for current study.

6.2.1. Experimental program

ECC specimen preparation

Type I Portland cement (CEM I 52.5 N), GGBS, fine aggregates, and polyvinyl-alcohol (PVA) fibers, superplasticizer (SP), rubber powders (RP), and tap water were used as ingredients for ECC specimen preparation. The Portland cement, GGBS (chemical composition given in Table 6.1), fine aggregates (particle size distribution given in Fig. 3.6), and PVA fibers are the same used in the previous section. The rubber powders are obtained by cutting, shredding, and grinding the waste tyre; the density of the rubber powders is 1100 kg/m$^3$, and the particle size distribution is shown in Fig.6.10.

The mix proportion design is given in Table 6.4. The previous study indicates that GGBS30 has the best chemical composition and high potential to engage self-healing, so GGBS-to-binder ratio of 30% was chosen here. However, GGBS30 exhibits lower ductility than GGBS60 as shown in the previous Chapter 5, which indicates the PSH indices of GGBS30 is smaller than that of GGBS60. One way to increase the PSH indices is to reduce the matrix toughness and cracking strength by increasing the water-to-binder ratio and to introduce artificial flaws such as rubber powders used in current mix design. The reduction of matrix toughness and cracking strength can also lead to the reduce of crack width which favors self-healing.

All the solid ingredients except for PVA fibers were dry mixed first with a three-gear planetary mixer for two minutes. Water and SP were slowly added into the mixture at low mixing speed within one minute, followed by medium mixing speed for another two minutes to achieve the required rheology of the fresh mortar. PVA fibers were then slowly added into the mixture at low mixing speed within one minute, followed by medium mixing speed for three minutes to ensure good fiber dispersion. The mixture with dispersed PVA fibers were then cast into small beam molds (280×70×50 mm).
The specimens, after demolding at one day, were cured in the laboratory condition (air, 20°C) for at least 28 days before the fatigue test.

**Fatigue test with self-healed ECC**

Four-point flexural fatigue test was used to evaluate the fatigue resistance of ECC. In order to determine the effect of self-healing, fatigue test was stopped after certain number of cycles, followed by wet-dry conditioning for healing and fatigue testing until the failure of specimens. Fig.6.11 illustrates the testing program. For all fatigue tests (cyclic loading), the flexural load level was fixed at 0.80 of ECC MOR. For Group 1, cyclic loading was applied to the specimens to failure so that the fatigue life of the specimens was determined. For Group 2, the specimens were conditioned for 14 wet-dry cycles and subject to cyclic loading until failure. This is to exclude the effect of ECC matrix hardening under water curing, which may also contributes to fatigue life extension. For Groups 3-5, the specimens were first loaded under fatigue of 387, 1,700, and 5,100 cycles, respectively, followed by 14 wet-dry cycles to engage healing. After which, the healed samples were loaded under fatigue to failure. For Group 6, the specimens experience two times of fatigue pre-cracking and healing before they were cyclically loaded to failure to investigate the repeatability of ECC self-healing. For each group, at least five specimens were included. The details of fatigue loading program and self-healing conditioning are given below.

All the flexural tests in this section were conducted with MTS Landmark servo-hydraulic testing machine. The MOR of ECC was determined by the displacement-controlled (0.2 mm/min) monotonic four-point flexural test (span length: 80-80-80mm) beforehand. For current mix design the MOR was measured as 7.33±0.76 MPa due to the inclusion of rubber powders. The fatigue tests were load-controlled: loading frequency equals to 5 Hz with a maximum load of 0.80 MOR and a minimum load of 0.20 MOR. A relatively high load level was chosen to limit the testing time. The real-time deflection during the fatigue tests was also recorded. For the self-healing sessions, every cycle of wet-dry conditioning consisted of submerging the specimens in water.
(20°C) for one day, followed by exposing the specimens in air (20°C) for another day, which was suggested by Yang et al (2009).

Besides fatigue life, crack width reduction was also measured to evaluate the self-healing degree of fatigue-cracked ECC. Images of cracks before and after conditioning were taken at magnification with Nikon DS-Fi2 high resolution camera. The crack width was measured at high-magnification image to monitor changes of crack width. Crack numbers on each specimen were also recorded after every fatigue loading session.

6.2.2. Experimental results

Effect of self-healing on fatigue life

Fig.6.12 illustrates the flexural fatigue life distribution of ECC specimens from Group 1 to Group 6. Each vertical bar in Fig.6.12 represents 14 wet-dry cycles for self-healing. It can be seen that the fatigue life of ECC varies a lot given different loading and conditioning history. The comparison between Group 1 and Group 2 demonstrates the extension of ECC fatigue life by water curing. The specimens in Group 1 sustained averagely 6,863 load cycles, while those from Group 2 extended the fatigue life to averagely 25,272 cycles. The extension of fatigue life can be attributed to that the continued hydration. The matrix toughness increased with hydration, and perhaps the fiber-matrix interface was also strengthened (will be further discussed later). These component-level changes control the crack development under fatigue load. The comparison of Group 3 to 5 against Group 2 shows the feasibility of extending fatigue life of ECC by self-healing also the effects of fatigue damage level (in current study i.e. the healing timing) on self-healing. Group 3, where the specimens were conditioned in 14 wet-dry cycles after 387 fatigue load cycles, sustained averagely 49,649 cycles, nearly the double of Group 2. However, the fatigue life of ECC was gradually shortened as the fatigue damage level was enhanced, as shown by the fatigue life of Groups 4 (averagely 35,004 cycles) and 5 (averagely 12,138 cycles), which were conditioned after 1,700 cycles and 5,100 cycles, respectively. The comparison of
Group 3 and Group 6 shows the repeatability of extending ECC fatigue life via self-healing. Slight fatigue life extension of Group 6 (averagely 50,348 cycles), as compared with Group 3 (averagely 49,649 cycles), was observed. In this case, the effect of second healing session is limited, which may be attributed to the fact that the second healing session was applied too late (after 5,100 cycles).

Fatigue of ECC is a process of material degradation. In the load-controlled flexural fatigue test, the material degradation is reflected by the continuous increase of beam deflection. Fig.6.13 shows the typical deflection (at the loading point) increase with fatigue load cycles of self-healed ECC beams. The deflection increased dramatically at the beginning due to the load ramping and dynamic effect; then it gradually entered into a relatively stable stage, where the deflection-load cycle curve shows a linear trend; approaching fatigue failure, the deflection increase became very fast until the beam rupture. Instead of the dramatic deflection increase at the beginning and approaching the end of the fatigue test, the stable stage better represent the fatigue resistance of the material. As a result, the deflection increasing rate (the slope of the deflection-load cycle curve in the stable stage) of Group 1 to 6 were illustrated in Fig.6.14 to understand the effect of self-healing on material degradation.

In Fig.6.14, it can be seen that the deflection increase of the self-healing groups (Groups 3, 4, 5, and 6) was much reduced from the control groups (Group 1 and 2), indicating that cracking-healing session effectively decelerated the material degradation under fatigue loading. Among the groups with one self-healing session (Groups 3, 4, and 5), the earlier healing results in slower deflection increase: the deflection increasing rate of Group 3 (healing after 387 cycles) is only 0.13 µm/cycle; while that of Group 4 (healing after 1,700 cycles) and 5 (healing after 5,100 cycles) were 0.39 µm/cycle and 0.45 µm/cycle, respectively. The deflection increasing rate of Group 6 is much smaller than the control groups (Group 1 and 2), which can be attributed to the first healing session, but shows little improvement from Group 3. This again may be attributed to the fact that the second healing session was applied too late (after 5,100 cycles).
Crack width reduction by self-healing

Fig. 6.15 illustrates the ECC crack width before and after self-healing conditioning, where for Group 6, only those cracks generated in the first-time fatigue test were included to continually trace the crack width reduction in both healing sessions. Comparing the crack width in Groups 3 to 5 before self-healing, it is noticed that the ECC crack width increases with the fatigue load cycles, as the fatigue loads progressively deteriorates the fiber-bridging across the crack (the detail of which is studied and discussed in Chapter 3 and 4). The self-healing conditioning can effectively reduce the crack width by crystal-like precipitates within the crack, as shown in Fig. 6.16. The average crack width reductions, despite the difference in original crack width, are similar and fall into a relatively small range (25-40 µm).

In Fig. 6.15, the cracks under Group 6 experienced two times of fatigue-healing sessions. After the second time of fatigue (5,100 cycles), the average crack width in the healed ECC increases by 36 µm, while the average crack width in the intact ECC (Group 5) increases by 70 µm after the same fatigue loading (5,100 cycles). This indicates that self-healing not only lead to the crack width reduction, but also strengthens the fiber-bridging and decelerates the fatigue-induced crack widening. It is also observed in Group 6 that the previously healed cracks can also achieve certain degree of crack width reduction during the second time of self-healing. However, the average crack width reduction during the second healing session is only 19 µm, while that during the first healing session is 32 µm.

Flexural stiffness recovery by self-healing

Fig. 6.17 illustrates the degradation of normalized ECC flexural stiffness (Groups 3 to 6) throughout the fatigue tests. The flexural stiffness is calculated as the ratio of fatigue load amplitude \((P_{\text{max}} - P_{\text{min}})\) over the loading-point displacement amplitude. The normalized stiffness is calculated as the ratio of stiffness in current cycle over that of in the first cycle.
It can be seen that the flexural stiffness keeps decreasing with fatigue load cycles. When the self-healing conditioning is applied early, for example after 387 cycles (Group 3 and 6), the flexural stiffness recover significantly due to the healing; however, the degree of flexural recover is mitigated when 1,700 cycles are applied before the healing (Group 4); when the number of pre-cracking cycles is increased to 5,100 cycles, the flexural recovery completely vanishes. It is also observed that the flexural stiffness is not repeated in the second time of self-healing, which may be attributed to that the engagement of the second healing session was too late at 5,000 cycles.

6.2.3. Discussions

In Fig.6.12, it is noticed that the Groups 3 and 4, after the fatigue pre-cracking and healing conditioning, sustained averagely 49,262 (=49,649-387) fatigue cycles and 33,304 (=35,004-1,700) fatigue cycles respectively, which are longer fatigue life than the water-cured control Group 2. In Fig.6.14, it is also noticed that the deflection increasing rate of Groups 3 and 4, after healing conditioning, are smaller than that of Group 2. It is observed that in Groups 3 to 6 some specimens eventually ruptured at a new crack generated in the second fatigue test, rather than an old crack generated in the first fatigue test, as summarized in Table 6.5. These observations imply that the self-healing of ECC is not only a process of fiber-bridging recovery towards its original status, but also a process of fiber-bridging improvement beyond the original status. Such improvement may be attributed to the possible material changes on the micro-scale: for the fiber embedded in the unhealed specimens, the interfacial bond between the fiber and matrix are determined by the hydration products, mainly C-S-H and Ca(OH)₂ (Neville 2011); for the fiber embedded in the healed specimens, the interfacial bond may also be affected by the healing products CaCO₃, which possibly leads to stronger interfacial bond than the hydration products.

Among these groups that experienced one healing session (Group 3 to 5), the healing-induced recovery, in terms of fatigue life (Fig.6.12), deflection-increasing rate (Fig.6.14), and stiffness recovery (Fig.6.17), decreases with fatigue cycles experienced by the samples before engaging self-healing. The reason of earlier healing leading to
more significant recovery can be two-folded. First, more fatigue cycle leads to larger crack width as shown in Fig.6.15. As shown in Yang’s work (2009), self-healing degree reduces with increased crack width. Second, more fatigue cycle introduces more severe fiber abrasion and fiber rupture (refer to Chapter 4) resulting in reduction of fiber-bridging capacity at the crack. As such, even if the crack can be sealed by self-healing products, the resulting fiber-reinforced self-healing composites at the healed crack is weaker when self-healing is engaged after long fatigue cycles.

In current study, only Group 6 is included to investigate the repeatability of self-healing. The effect of second healing sessions is limited for Group 6. It may be attributed to the fatigue damage level (after 387+5,100 cycles), under which condition the crack width was too large to introduce significant mechanical recovery. The repeatability of applying self-healing to extend fatigue life of ECC would remain questioned until the effect of earlier second healing session is engaged and checked.

6.3. Conclusions

This chapter reports two stages to engage robust self-healing that improves the fatigue resistance of ECC. ECC is further tailored on base of the ECC fatigue theory established in Chapter 3 and 4 and the mix design reported in Chapter 5. The first stage studied the effects of slag content, crack width and conditioning alkalinity on self-healing behavior of ECC. Single-cracked ECC specimens with three different slag contents (cement replacement ratio at 0%, 30%, and 60%) and crack width up to 300μm were conditioned under water/dry or NaOH (pH=13)/dry cycles. Self-healing performance was evaluated based on crack width reduction, resonant frequency recovery, and SEM/EDX analysis. Following conclusions can be made in current study.

- ECC self-healing performance is greatly affected by the slag content, crack width, and environmental alkalinity. The addition of slag up to 35% enables healing of larger crack (90 μm). After that, the allowable crack width to engage complete healing reduces with slag content. Optimum slag addition, smaller crack width and water/dry conditioning cycle favor self-healing in ECC.
CaCO$_3$ is the dominant healing products for samples subjected to the water/dry cycle while a mixture of CaCO$_3$ and C-S-H represents the major healing products for samples subjected to the NaOH/dry cycle.

NaOH/dry conditioning promotes slag hydration but suppresses the formation of CaCO$_3$ precipitates in cracks as healing products.

CaCO$_3$ precipitation through carbonation is a more effective mean to engage self-healing than the formation of C-S-H through alkali-activated slag hydration.

Design of self-healing in ECC should consider both the physical properties and the chemical environment. A design chart associating allowable crack width and slag content is illustrated for ingredients selection and component tailoring to engage robust self-healing in ECC.

The second stage studied the effect of self-healing on the fatigue performance of ECC. ECC beams were made with 30% cement replaced by GGBS, as suggested by the experimental results from the previous section. Flexural fatigue life of self-healed ECC was experimentally compared with that of control ECC specimens. Following conclusions can be drawn from the test results:

- Self-healing can effectively reduce the crack width, recover flexural stiffness, and extend the flexural fatigue life of ECC.
- The degree of mechanical improvement of ECC (fatigue life extension and flexural stiffness recovery) depends on the fatigue damage level to engage healing; in the fatigue life range in current research (healing conditioning after 387 load cycles to after 5,100 load cycles), earlier healing favors the mechanical recovery. Early engagement of healing is necessary for fatigue resistance recovery of ECC as the fatigue crack width and fiber-bridging deterioration increases with load cycles.
References

### Table 6.1 Elementary composition of GGBS

<table>
<thead>
<tr>
<th>Element</th>
<th>GGBS0</th>
<th>GGBS30</th>
<th>GGBS60</th>
</tr>
</thead>
<tbody>
<tr>
<td>CaO</td>
<td>39.43%</td>
<td>36.67%</td>
<td>34.48%</td>
</tr>
<tr>
<td>SiO₂</td>
<td>31.19%</td>
<td>5.64%</td>
<td>9.84%</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>13.41%</td>
<td>45.81%</td>
<td>45.79%</td>
</tr>
<tr>
<td>MgO</td>
<td>9.32%</td>
<td>8.50%</td>
<td>8.59%</td>
</tr>
<tr>
<td>SO₃</td>
<td>4.25%</td>
<td>4.85%</td>
<td>4.85%</td>
</tr>
<tr>
<td>Others</td>
<td>2.40%</td>
<td>3.89%</td>
<td>3.81%</td>
</tr>
</tbody>
</table>

### Table 6.2 Proportions of mix design in current research

<table>
<thead>
<tr>
<th>Mix. No</th>
<th>Cement (kg/m³)</th>
<th>GGBS (kg/m³)</th>
<th>Sand (kg/m³)</th>
<th>Water (kg/m³)</th>
<th>PVA fiber (kg/m³)</th>
<th>SP (L/m³)</th>
<th>Cement replacing ratio</th>
<th>w/b ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (GGBS0)</td>
<td>1411</td>
<td>0</td>
<td>282</td>
<td>423</td>
<td>26*</td>
<td>3.6</td>
<td>0.00</td>
<td>0.30</td>
</tr>
<tr>
<td>2 (GGBS30)</td>
<td>976</td>
<td>418</td>
<td>279</td>
<td>418</td>
<td>26*</td>
<td>3.0</td>
<td>0.30</td>
<td>0.30</td>
</tr>
<tr>
<td>3 (GGBS60)</td>
<td>551</td>
<td>827</td>
<td>276</td>
<td>414</td>
<td>26*</td>
<td>2.4</td>
<td>0.60</td>
<td>0.30</td>
</tr>
<tr>
<td>4 (GGBS0a)</td>
<td>1411</td>
<td>0</td>
<td>282</td>
<td>423</td>
<td>8.5**</td>
<td>3.6</td>
<td>0.00</td>
<td>0.30</td>
</tr>
<tr>
<td>5 (GGBS30a)</td>
<td>976</td>
<td>418</td>
<td>279</td>
<td>418</td>
<td>8.5**</td>
<td>3.0</td>
<td>0.30</td>
<td>0.30</td>
</tr>
<tr>
<td>6 (GGBS60a)</td>
<td>551</td>
<td>827</td>
<td>276</td>
<td>414</td>
<td>8.5**</td>
<td>2.4</td>
<td>0.60</td>
<td>0.30</td>
</tr>
</tbody>
</table>

*The fiber content was fixed at 2% in volume fraction.
** The fiber content was fixed at 0.65% in volume fraction.

### Table 6.3 Elementary compositions (by mass) of self-healing products

<table>
<thead>
<tr>
<th>Element</th>
<th>GGBS0</th>
<th>GGBS30</th>
<th>GGBS60</th>
<th>GGBS0</th>
<th>GGBS30</th>
<th>GGBS60</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ca</td>
<td>21.19%</td>
<td>36.67%</td>
<td>34.48%</td>
<td>26.29%</td>
<td>20.24%</td>
<td>18.97%</td>
</tr>
<tr>
<td>Si</td>
<td>3.41%</td>
<td>5.64%</td>
<td>6.06%</td>
<td>7.09%</td>
<td>8.59%</td>
<td>18.13%</td>
</tr>
<tr>
<td>C</td>
<td>14.10%</td>
<td>14.80%</td>
<td>9.84%</td>
<td>10.15%</td>
<td>14.88%</td>
<td>7.52%</td>
</tr>
<tr>
<td>O</td>
<td>55.05%</td>
<td>39.01%</td>
<td>45.81%</td>
<td>45.79%</td>
<td>46.99%</td>
<td>48.96%</td>
</tr>
<tr>
<td>Others</td>
<td>6.26%</td>
<td>3.89%</td>
<td>3.81%</td>
<td>10.68%</td>
<td>9.30%</td>
<td>6.42%</td>
</tr>
<tr>
<td>Ca/Si</td>
<td>7.18%</td>
<td>6.45%</td>
<td>5.63%</td>
<td>3.81%</td>
<td>2.39%</td>
<td>1.21%</td>
</tr>
</tbody>
</table>
Table 6.4 Mix proportion design of ECC used for self-healing fatigue test

<table>
<thead>
<tr>
<th>Portland Cement (kg/m³)</th>
<th>GGBS (kg/m³)</th>
<th>Sand (kg/m³)</th>
<th>Rubber powder (kg/m³)</th>
<th>Water (kg/m³)</th>
<th>PVA fiber (kg/m³)</th>
<th>SP (L/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>822</td>
<td>352</td>
<td>176</td>
<td>117</td>
<td>423</td>
<td>26</td>
<td>3.6</td>
</tr>
</tbody>
</table>

Table 6.5 Probability of specimen rupture at a new crack generated after self-healing

<table>
<thead>
<tr>
<th>Group No.</th>
<th>Number of fatigue cycles experienced before self-healing</th>
<th>Total number of specimens</th>
<th>Number of specimen ruptured at a new crack location</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>387</td>
<td>6</td>
<td>5</td>
<td>83%</td>
</tr>
<tr>
<td>4</td>
<td>1,700</td>
<td>6</td>
<td>2</td>
<td>33%</td>
</tr>
<tr>
<td>5</td>
<td>5,100</td>
<td>7</td>
<td>1</td>
<td>14%</td>
</tr>
</tbody>
</table>
Fig. 6.1 Illustration of (a) dog-bone specimen dimensions and (b) uniaxial tensile test set-up (50kN UTM, loading rate 0.04mm/min)
Fig. 6.2 Crack width of pre-cracked GGBS0/0a, GGBS30/30a, and GGBS60/60a ECC specimens before and after 14 water/dry conditioning cycles.
Fig. 6.3. Crack width of pre-cracked GGBS0/0a, GGBS30/30a, and GGBS60/60a ECC specimens before and after 14 NaOH/dry conditioning cycles.
Fig. 6.4 Crack width reduction ratio of pre-cracked ECC specimens
Fig. 6.5 Normalized transverse resonant frequency (TRF) as a function of crack width before and after 14 water/dry conditioning cycles
Fig. 6.6 Normalized transverse resonant frequency (TRF) as a function of crack width before and after 14 NaOH/dry conditioning cycles
Fig. 6.7 (a) A healed crack in GGBS30 specimen subjected to 14 water/dry conditioning cycles; (b) and (c) Typical morphology of the healing products at 2,000× and 10,000× magnifications, respectively.
Fig. 6.8 Self-healing products precipitated on PVA fiber
Fig. 6.9 ECC self-healing design chart subjected to water/dry conditioning cycles

Fig. 6.10 Particle size distribution of rubber powder used in current study
Fig. 6.11 Testing program of fatigue load and self-healing

Group 1
- Fatigue load until failure

Group 2
- 14 wet-dry cycles
- Fatigue load until failure

Group 3
- Fatigue load 387 cycles
- Self-healing 14 wet-dry cycles
- Fatigue load until failure

Group 4
- Fatigue load 1700 cycles
- Self-healing 14 wet-dry cycles
- Fatigue load until failure

Group 5
- Fatigue load 5100 cycles
- Self-healing 14 wet-dry cycles
- Fatigue load until failure

Group 6
- Fatigue load 387 cycles
- Self-healing 14 wet-dry cycles
- Fatigue load 5100 cycles
- Self-healing 14 wet-dry cycles
- Fatigue load until failure
Fig. 6.12 Fatigue life of ECC specimens from Group 1 to Group 6, the vertical bars represent the time of healing conditioning.

Fig. 6.13 Typical loading point deflection increase with load cycles in flexural fatigue test.
Fig. 6.14 Deflection increasing rate of ECC beams under flexural fatigue loading
Fig. 6.15 Crack width in ECC specimens (Group 3 to 6) before and after self-healing conditioning, for Group 6 only the cracks generated in the first fatigue test are included.

Fig. 6.16 Typical crack width reduction and self-healing products observed under optical microscope.
Fig. 6.17 Crack width in ECC specimens (Group 3 to 6) before and after self-healing conditioning, for Group 6 only the cracks generated in the first fatigue test are included.
CHAPTER 7 CONCLUDING REMARKS

This thesis develops a novel design approach to understand the fatigue response and to improve the fatigue resistance of engineered composites. Unlike traditional design approach which is (semi-)empirical, the methodology used in this thesis is to understand the causes of fatigue dependency in component micro-scale so that proper composite re-engineering can be taken. Specifically, this approach is demonstrated by tailoring engineered cementitious composites (ECC), a group of fiber-reinforced strain-hardening cement-based material with ultra-high ductility, to achieve high fatigue resistance under the guidance of a newly developed micromechanics-based fatigue model of ECC.

To trace the sources of fatigue dependency of ECC, fatigue sensitivity in the component micro-scale level, i.e. fibers, matrix, and the interface, was experimentally investigated. A micro-mechanics based fatigue model integrating the above fatigue dependent component was developed. ECC fatigue behavior in the macro-scale can be quantified, predicted, and tailored. Novel approaches were proposed to re-engineer and enhance the fatigue life of ECC.

Research activities are planned and conducted accordingly. First, the fatigue-dependency of the properties of fibers and fiber-matrix interface was identified and characterized with single-fiber tests and single-fiber pullout tests under fatigue, respectively (Chapter 3). With the characterized fatigue-dependency of ECC component, a multi-scale model that predicts ECC fatigue behavior, from the micro-scale to the macro-scale, is then established (Chapter 4). These two parts form the fatigue theory of ECC, which guides the material tailoring attempts: high-MOR is developed into ECC, which greatly enhance the flexural fatigue performance of ECC (Chapter 5); self-healing of ECC is optimized and engaged to ECC, which greatly extends the fatigue life of this material (Chapter 6).
7.1. Findings and accomplishments

Significant findings and accomplishments made in this thesis are summarized in the following section.

7.1.1. ECC fatigue theory

The fatigue degradation of ECC is caused by the material deterioration of the components, i.e. fibers, matrix, and especially the fiber-matrix interface, on the micro-scale. In order to fundamentally understand the material property change during fatigue loading, a series of fatigue/monotonic single-fiber tests and single-fiber pullout tests were conducted on PVA fibers with or without surface oil treatment.

In the course of this study, it is found that fatigue loading would significantly reduce the in-situ tensile strength of embedded fibers, as the fiber-matrix relative movement causes severe abrasion to the fiber surface. The degree of fiber strength reduction increases with number of fatigue cycles.

The phenomenon of fiber-matrix interfacial crack propagation under fatigue loads is noticed for the first time, and referred as fatigue-induced fiber debonding. The debonding rate depends on the fatigue load level and follows Paris’ Law. Fatigue loading does not change the chemical bond between fiber and matrix, but increases the interface frictional bond, probably because the reciprocation abrades and roughens the surface of debonded fiber. The fatigue-induced frictional bond increase leads to higher tensile stiffness in the debonding stage and the pull-out stage, where are referred as fatigue hardening and fatigue slip-hardening, respectively. The degree of fatigue hardening and fatigue slip-hardening depends on the load level and number of load cycles experienced in fatigue test. Higher load level and more number of cycles would lead to stronger fatigue hardening and fatigue slip-hardening effect.

Comparing the fiber strength reduction, fatigue hardening, and fatigue slip-hardening between the non-oil-coated fibers and oil-coated fibers, it is concluded that oil treatment mitigates the fatigue-induced fiber strength reduction, fatigue hardening, and fatigue slip-hardening as it lessens the abrasion to the fiber surface. It is also observed
that large fiber slippage, which often occurs to the oil-coated fibers, would decrease the in-situ fiber strength.

The fatigue-dependency of ECC components, i.e. fatigue-induced fiber debonding, fatigue hardening, fatigue slip-hardening, and fatigue-induced fiber strength reduction, was quantitatively determined, which was then incorporated into a theoretical model based on the micromechanics principle (Yang et al. 2008). The model is able to predict and describe the fiber-bridging constitutive law $\sigma(\delta)$ of ECC under fatigue loading. The modeling results show that on the meso-level, the pseudo strain-hardening coefficients, i.e. PSH energy and PSH strength of ECC would significantly reduce with increased load cycles and load level, which successfully explain the widely reported phenomena of premature failure of ECC subject to fatigue (Suthiwarapirak et al. 2004, Qian 2007).

The fatigue dependent $\sigma(\delta)$ constitutive model was then used to modify an established monotonic model that predicts the flexural fatigue propagation of normal FRC (Li and Matsumoto 1998). The new flexural fatigue model can accurately predict the flexural fatigue life of ECC.

7.1.2. Developing high-MOR ECC

Flexural strength of concrete is always a big concern in designing the fatigue life of concrete structures like road pavement and bridge decks. High flexural strength is important to extend the service life of such infrastructure. A three-step ISMD approach was used, in the course of this study, to develop high-MOR ECC with locally available ingredients for precast pavement slab: determining the target material properties; developing ECC with the target properties; and verifying the structural performance of ECC precast pavement slab.

It was concluded that high tensile strength is the key to achieve high MOR in ECC while moderate tensile ductility and sufficient compressive strength are necessary. Increased GGBS content in replacement of cement (within the range of 0% to 60%) can significantly enhance the tensile strength and therefore the MOR of ECC. The optimized mix design using local GGBS produces MOR of 13 MPa and extends the
flexural fatigue life of ECC by several orders. The structural analysis indicates that the slab thickness can be reduced from 150 mm to 90 mm while the service life is doubled from 20 years to 40 years by replacing traditional concrete with newly developed ECC.

7.1.3. Engaging robust self-healing

Fatigue of fiber-reinforced cementitious composites like ECC manifests itself as the fiber-bridging loss and crack widened. On the other hand, ECC has self-healing capability to seal the crack and to restore the mechanical properties [Ref.]. This study attempted to enhance the self-healing robustness of ECC by experimentally investigating the effect of chemical composition (GGBS content) on self-healing of ECC and to engage such healing to extend ECC fatigue life.

In the course of this study, it is discovered that besides crack width (Yang et al. 2009), ECC self-healing performance is greatly affected by the slag content and environmental alkalinity. The increase of slag content up to 35% enables healing of larger crack; after which further increase of slag content would reduce the potential of self-healing. Low alkalinity (water-dry conditioning) leads to higher degree of self-healing, compared to high alkalinity (NaOH-dry conditioning). It is also found that the main healing product of GGBS-ECC is mainly CaCO$_3$ with a relatively smaller portion of C-S-H, and more robust healing happens when the chemical composition and alkalinity favors the formation of CaCO$_3$. The design of self-healing ECC should consider both physical properties (crack width) and the chemical environment (chemical composition and alkalinity). A design chart associating allowable crack width and slag content is proposed to engage robust self-healing in the GGBS-ECC system.

In study of the fatigue behavior of self-healed ECC, it is discovered that self-healing can effectively reduce fatigue crack (width?), recover the flexural stiffness, and extend the flexural fatigue life of the material. The degree of recovery (fatigue life extension and flexural stiffness recovery) depends on the timing of healing, i.e. damage degree. In the fatigue life range studied (healing conditioning after 387 load cycles to after
5,100 load cycles), earlier healing favors the mechanical recovery, as the fatigue crack width and fiber bridging deterioration increase with load cycles.

7.2. Impact of research

This research demonstrates a systematic approach to discover the underlying mechanisms of fatigue response of fiber-reinforced composites and feasibility of improving fatigue performance of engineered composites on multi-scales. Specifically, it identifies methodologies to improve fatigue performance of one type of engineered material, i.e. engineered cementitious composites (ECC). Nevertheless, the multi-scale design philosophy and approach can also be applied to other engineered materials.

The study demonstrates the necessity to drive the research on fatigue of fiber-reinforced composites to the more fundamental micro-scale, by successfully demonstrating and characterizing the fatigue-induced component property change on the micro-scale. It further completes the ECC design theory by adding the fatigue-dependency into the micromechanical models. The modeling work establishes a quantitative basis for computing the fatigue behavior of ECC, which can also be extended to other fiber-reinforced composites.

The study further completes the work to design robust self-healing behavior in ECC by including the effect of chemical composition on self-healing degree. It also confirms the feasibility of designing the self-healing feature into fatigue-dominated ECC structures.

7.3. Recommendations for future research

In this study, the fatigue-dependency of ECC microstructures are characterized as three effects that contribute to the fiber-bridging degradation and loss of strain-hardening, namely fiber strength reduction, fatigue hardening, and fatigue slip-hardening. Besides oil-coating content, many other factors, especially from the matrix composition, are expected to influence the degree of fiber strength reduction, fatigue hardening, and fatigue slip-hardening. Such factors could be identified by experimental investigation with single-fiber tests and single-fiber pullout tests under fatigue. Specifically, it is
expected that content of slag (or fly ash), aggregate size, and water cement ratio would be potential factors. In current study, only the effects of tensile fatigue on fiber are considered as it is considered as the major deteriorating factor. In the future study, the effects of compressive fatigue should be investigated, especially for flexible fibers which tend to be bended under compression.

The model established in current study is still not able to predict the ECC strain-capacity under tensile fatigue. The missing link lies in the difficulty to determine the crack spacing based on the PSH indices, which not only prevents the prediction of strain capacity under fatigue but also that under monotonic loading. Future work should focus on how to calculate the crack spacing.

In the study of self-healing behavior of ECC, it is found that there is significant mechanical recovery. However, on the micro-scale level, it is still unclear what contributes such mechanical recovery. It is assumed that the mechanical recovery either results from the filling of cracks or the re-bonding between fiber and healed matrix. It would be necessary to investigate the self-healing behavior of ECC on the micro-scale. The self-healing repeatability of fatigue-cracked ECC is observed but the better recovery is expected if the second healing session is planed earlier. It requires additional experiment to confirm such hypothesis. So far the ECC fatigue life extension by self-healing conditioning has been engaged under unloaded state in laboratory conditions, it is important to examine the self-healing efficiency under in-situ conditions, when the specimens are loaded and in the field condition.
References


APPENDIX A CALCULATION OF ECC MOR

In Fig. A.1, an ECC beam section subjected to the highest bending moment $M$ is shown, where the section is assumed to remain plane and strain vary linearly along with the depth. In this case, the stress and strain distributions along the beam depth are given by Eqn. A.1 and A.2 respectively as following:

$$\sigma(x) = \sigma_1(x) = \tau_{tc} + \frac{\tau_{tu} - \tau_{tc}}{\epsilon_{tu} - \epsilon_{tc}} [\epsilon(x) - \epsilon_{tc}] \quad \text{for } 0 \leq x \leq a \quad \text{(Eqn. A.1a)}$$

$$\sigma(x) = \sigma_2(x) = \frac{\tau_{tc}}{\epsilon_{tc}} \epsilon(x) \quad \text{for } a < x \leq c \quad \text{(Eqn. A.1b)}$$

$$\sigma(x) = \sigma_3(x) = -2 \frac{\epsilon_{cp}}{\epsilon_{cp}} \epsilon(x) \quad \text{for } c < x \leq e \quad \text{(Eqn. A.1c)}$$

$$\sigma(x) = \sigma_4(x) = -\frac{1}{2} \epsilon_{cp} [1 + \frac{\epsilon(x)}{\epsilon_{cp}}] \quad \text{for } e < x \leq d \quad \text{(Eqn. A.1d)}$$

$$\epsilon(x) = \frac{x(\epsilon_{tc} - \epsilon_t)}{a} + \epsilon_t \quad \text{for } 0 \leq x \leq c \quad \text{(Eqn. A.2a)}$$

$$\epsilon(x) = \frac{x(\epsilon_{tc} - \epsilon_t)}{a} - \epsilon_t \quad \text{for } c < x \leq d \quad \text{(Eqn. A.2b)}$$

where $x$ is the distance from the extreme tension fiber to an arbitrary point along the depth of the beam; $\epsilon_t$ is the tensile strain at the extreme tension fiber; $a$ is the depth of inelastic microcracking zone; $c$ is the distance from the extreme tension fiber to the neutral axis; and $e$ is the distance from the extreme tension fiber to the point where the material changes stiffness in compression.

With a complete set of material properties along with given beam width $b$ and depth $d$, the external bending moment $M$ and the tensile strain at the extreme tension fiber $\epsilon_t$ can be determined by solving equilibrium Eqn. A.3 and A.4.

$$\int_0^a b \sigma_1(x) dx + \int_a^c b \sigma_2(x) dx + \int_c^e b \sigma_3(x) dx + \int_e^d b \sigma_4(x) dx = 0 \quad \text{(Eqn. A.3)}$$
\[
\int_{a}^{b} b\sigma_1(x)x\,dx + \int_{a}^{c} b\sigma_2(x)x\,dx + \int_{c}^{e} b\sigma_3(x)x\,dx + \int_{e}^{d} b\sigma_4(x)x\,dx = M
\]  
(Eqn. A.4)

In order to solve these equilibrium equations, three distinct stages of crack propagation must be clarified. Stage 1 starts from the beginning of loading until \(\varepsilon_t\) reaches \(\varepsilon_{tc}\), i.e. the occurrence of the first tensile crack; Stage 2 starts from the first tensile crack until \(\varepsilon_c\) reaches one third of \(\varepsilon_{cp}\), i.e. the change of stiffness in compression; Stage 3 starts from the change of stiffness in compression until the final failure of beam. It should be noticed that if the final failure happens before \(\varepsilon_c\) reaches one third of \(\varepsilon_{cp}\), there would be no Stage 3. Both \(M\) and \(\varepsilon_t\) have two expressions, for Stage 2 and Stage 3 respectively. As a result following expressions do not apply for Stage 1.

The strain at extreme tension fiber \(\varepsilon_t\) should be solved before \(M\). Strains at extreme tension fiber for Stage 2 and Stage 3, i.e. \(\varepsilon_{t2}\) and \(\varepsilon_{t3}\) are expressed in Eqn.A.5 and A.6 respectively.

\[
\varepsilon_{t2} = \frac{-a_1 - \sqrt{a_1^2 - 4a_0a_2}}{2a_0} \tag{Eqn. A.5a}
\]

where

\[
a_0 = -2\sigma_{cp}(\varepsilon_{tu} - \varepsilon_{tc})(1 - r)^2 + \varepsilon_{cp}(\sigma_{tu} - \sigma_{tc})r^2 \tag{Eqn. A.5b}
\]

\[
a_1 = 4\sigma_{cp}\varepsilon_{tc}(\varepsilon_{tu} - \varepsilon_{tc})(1 - r) + 2\varepsilon_{cp}(\varepsilon_{tc}\sigma_{tu} - \varepsilon_{tu}\sigma_{tc})r^2 \tag{Eqn. A.5c}
\]

\[
a_2 = \varepsilon_{tc}[-2\sigma_{cp}\varepsilon_{tc}(\varepsilon_{tu} - \varepsilon_{tc}) - \varepsilon_{cp}(\varepsilon_{tc}\sigma_{tu} - \varepsilon_{tu}\sigma_{tc})r^2 \tag{Eqn. A.5d}
\]

\[
\varepsilon_{t3} = \frac{-a_1 - \sqrt{a_1^2 - 4a_0a_2}}{2a_0} \tag{Eqn. A.6a}
\]

where

\[
a_0 = -6\sigma_{cp}(\varepsilon_{tu} - \varepsilon_{tc})(1 - r)^2 + 12\varepsilon_{cp}(\sigma_{tu} - \sigma_{tc})r^2 \tag{Eqn. A.6b}
\]
\[ a_1 = 12\sigma_{cp}(\varepsilon_{tu} - \varepsilon_{tc})(\varepsilon_{tc} - \varepsilon_{cp}r)(1 - r) + 24\varepsilon_{cp}(\varepsilon_{tc}\sigma_{tu} - \varepsilon_{tu}\sigma_{tc})r^2 \]  
(Eqn.A.6c)

\[ a_2 = 2\sigma_{cp}(\varepsilon_{tu} - \varepsilon_{tc})(\varepsilon_{cp}^2 r^2 + 6\varepsilon_{cp}\varepsilon_{tc}r - 3\varepsilon_{tc}^2) - 12\varepsilon_{tc}\varepsilon_{cp}(\varepsilon_{tc}\sigma_{tu} - \varepsilon_{tu}\sigma_{tc})r^2 \]  
(Eqn.A.6d)

After \( \varepsilon_t \) is determined, strain at the extreme compression fiber \( \varepsilon_c \) can be calculated with Eqn.A.7, resulting in determination of stress distribution along the depth (Eqn.A.1 and A.2). Fig.A.1 and A.2 show the stress distribution at beam failure of Stage 2 and Stage 3 respectively. As shown in Fig.A.1 and A.2, the stress blocks are divided into regular geometries, which is denoted as \( F_i \) \((i=1, 2, 3, 4, 5, \text{ or } 6)\). For \( F_i \), there is a corresponding \( L_i \), which is the distance between the geometrical center of \( F_i \) to neutral axis. Expression of \( F_i \) and \( L_i \) are given in Eqn.A.8 (failure in Stage 2) and A.9 (failure in Stage 3) respectively.

\[ \varepsilon_c = \frac{1-r}{r} \varepsilon_t - \frac{1}{r} \varepsilon_{tc} \]  
(Eqn.A.7)

\[ F_1 = \frac{b}{2} \left( \frac{\varepsilon_c d}{\varepsilon_t + \varepsilon_c} \right) (\varepsilon_c E_{ct1}) \]  
(Eqn.A.8a)

\[ F_2 = \frac{b}{2} \left( \frac{\varepsilon_{tc} d}{\varepsilon_t + \varepsilon_c} \right) \sigma_{tc} \]  
(Eqn.A.8b)

\[ F_3 = b \left[ \frac{(\varepsilon_t - \varepsilon_{tc})d}{\varepsilon_t + \varepsilon_c} \right] \sigma_{tc} \]  
(Eqn.A.8c)

\[ F_4 = \frac{b}{2} \left[ \frac{(\varepsilon_t - \varepsilon_{tc})d}{\varepsilon_t + \varepsilon_c} \right] [(\varepsilon_t - \varepsilon_{tc})E_{ct2}] \]  
(Eqn.A.8d)

\[ L_1 = \frac{2}{3} \left( \frac{\varepsilon_c d}{\varepsilon_t + \varepsilon_c} \right) \]  
(Eqn.A.8e)

\[ L_2 = \frac{2}{3} \left( \frac{\varepsilon_{tc} d}{\varepsilon_t + \varepsilon_c} \right) \]  
(Eqn.A.8f)
\[ L_3 = \left( \frac{\varepsilon_{tc} d}{\varepsilon_t + \varepsilon_c} \right) + \frac{1}{2} \left( \frac{\varepsilon_t - \varepsilon_{tc} d}{\varepsilon_t + \varepsilon_c} \right) \]  
(Eqn.A.8g)

\[ L_4 = \left( \frac{\varepsilon_{tc} d}{\varepsilon_t + \varepsilon_c} \right) + \frac{2}{3} \left( \frac{\varepsilon_t - \varepsilon_{tc} d}{\varepsilon_t + \varepsilon_c} \right) \]  
(Eqn.A.8h)

\[ F_1 = \frac{b}{2} \left[ \frac{(\varepsilon_{cp}/3)d}{\varepsilon_t + \varepsilon_c} \right] (2\sigma_{cp}/3) \]  
(Eqn.A.9a)

\[ F_2 = \frac{b}{2} \left( \frac{\varepsilon_{tc} d}{\varepsilon_t + \varepsilon_c} \right) \sigma_{tc} \]  
(Eqn.A.9b)

\[ F_3 = b \left[ \frac{(\varepsilon_t - \varepsilon_{tc}) d}{\varepsilon_t + \varepsilon_c} \right] \sigma_{tc} \]  
(Eqn.A.9c)

\[ F_4 = \frac{b}{2} \left[ \frac{(\varepsilon_t - \varepsilon_{tc}) d}{\varepsilon_t + \varepsilon_c} \right] \left[ (\varepsilon_t - \varepsilon_{tc}) E_{t2} \right] \]  
(Eqn.A.9d)

\[ F_5 = b \left[ \frac{(\varepsilon_c - \varepsilon_{cp}/3) d}{\varepsilon_t + \varepsilon_c} \right] (2\sigma_{cp}/3) \]  
(Eqn.A.9e)

\[ F_6 = \frac{b}{2} \left[ \frac{(\varepsilon_c - \varepsilon_{cp}/3) d}{\varepsilon_t + \varepsilon_c} \right] \left[ (\varepsilon_c - \varepsilon_{cp}/3) E_{c2} \right] \]  
(Eqn.A.9f)

\[ L_1 = \frac{2}{3} \left[ \frac{(\varepsilon_{cp}/3) d}{\varepsilon_t + \varepsilon_c} \right] \]  
(Eqn.A.9g)

\[ L_2 = \frac{2}{3} \left( \frac{\varepsilon_{tc} d}{\varepsilon_t + \varepsilon_c} \right) \]  
(Eqn.A.9h)

\[ L_3 = \left( \frac{\varepsilon_{tc} d}{\varepsilon_t + \varepsilon_c} \right) + \frac{1}{2} \left( \frac{\varepsilon_t - \varepsilon_{tc} d}{\varepsilon_t + \varepsilon_c} \right) \]  
(Eqn.A.9i)

\[ L_4 = \left( \frac{\varepsilon_{tc} d}{\varepsilon_t + \varepsilon_c} \right) + \frac{2}{3} \left( \frac{\varepsilon_t - \varepsilon_{tc} d}{\varepsilon_t + \varepsilon_c} \right) \]  
(Eqn.A.9j)

\[ L_5 = \left[ \frac{(\varepsilon_{cp}/3) d}{\varepsilon_t + \varepsilon_c} \right] + \frac{1}{2} \left[ \frac{(\varepsilon_c - \varepsilon_{cp}/3) d}{\varepsilon_t + \varepsilon_c} \right] \]  
(Eqn.A.9k)

\[ L_6 = \left[ \frac{(\varepsilon_{cp}/3) d}{\varepsilon_t + \varepsilon_c} \right] + \frac{2}{3} \left[ \frac{(\varepsilon_c - \varepsilon_{cp}/3) d}{\varepsilon_t + \varepsilon_c} \right] \]  
(Eqn.A.9l)

In Eqn.A.8 and A.9, \(E_{c1}, E_{c2},\) and \(E_{t2}\) are slope of the first section of compressive stress-strain curve, slope of the second section of compressive stress-strain curve, and slope
of the second section of tensile stress-strain curve, respectively. Their expressions are shown in Eqn.A.10.

\[ E_{c1} = \frac{2\sigma_{cp}}{\varepsilon_{cp}} \]  
\[ E_{c2} = \frac{\sigma_{cp}}{2\varepsilon_{cp}} \]  
\[ E_{t2} = \frac{\sigma_{tu} - \sigma_{tc}}{\varepsilon_{tu} - \varepsilon_{tc}} \]  

(Eqn.A.10a)  
(Eqn.A.10b)  
(Eqn.A.10c)

Finally, the bending moment of ECC beam in Stage 2 and Stage 3 are expressed as:

\[ M_2 = \sum_{i=1}^{4} F_i L_i \]  
\[ M_3 = \sum_{i=1}^{6} F_i L_i \]  

(Eqn.A.11a)  
(Eqn.A.11b)
Fig. A.1 Stress distribution when ECC beam failed in Stage 2

Fig. A.2 Stress distribution when ECC beam failed in Stage 3
## APPENDIX B PARAMETRIC STUDY RESULTS

Table B.1 Results of bendable concrete property parametric study (Continued)

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<tr>
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<th>$\sigma_{cp}$ (N/m²)</th>
<th>$\varepsilon_{tu}$</th>
<th>Failure Mode*</th>
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<th>$\varepsilon_c$ at failure</th>
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*1T=tensile failure, C=compressive failure, T/C=tensile and compressive failure simultaneously
Table B.1 Results of bendable concrete property parametric study (Continued)

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<th>$\sigma_{tu}$ (N/m$^2$)</th>
<th>$\sigma_{cp}$ (N/m$^2$)</th>
<th>$\varepsilon_{tu}$</th>
<th>Failure Mode*</th>
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</tbody>
</table>

$^1$T=tensile failure, C=compressive failure, T/C=tensile and compressive failure simultaneously