**STRUCTURAL PERFORMANCE AND SELF-HEALING BEHAVIOUR OF
ENGINEERED CEMENTITIOUS COMPOSITES SUBJECTED TO FATIGUE
LOADING**

By

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ABSTRACT

This research investigates the effect of fatigue loading on the flexural performance and self-healing behaviour of beams and link slabs made of Engineered Cementitious Composites (ECC). The influences of fly ash content, types/size of sand, MgO agent, fatigue stress level/cycle and age are analyzed based on strength/deflection capacity recovery and residual strength/deflection/energy absorbing capacity. The deflection evolution rate and energy absorption capacity were much higher in ECC link slabs compared to their SCC counterparts. Higher energy absorption and deflection evolution rate were observed in mortar sand based ECC specimens during fatigue loading. ECC link slabs with mortar sand having 55% fly ash content have shown the best self-healing and fatigue performance attaining high residual strength, deflection and energy absorbing capacity of up to 98.3%, 95.4% and 97.1% of control specimens, respectively besides retaining multi-cracking characteristics. This research demonstrates viability of using ECC link slab for construction of joint-free bridges.

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DEDICATION

To my parents
who sacrificed their life for my greatest success

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LIST OF SYMBOLS AND ABBREVIATIONS

AASHTO: American Association of State Highway and Transportation Officials

C: Portland Cement

ECC: Engineered Cementitious Composite

F: Class F-Fly Ash

FA/C: Fly Ash Cement Replacement Ratio

FRC: Fiber Reinforced Concrete

$f_{y\text{steel}}$: Yielding Stress

GU: General Use

HPFRCCs: High-Performance Fiber-Reinforced Cementitious Composites

HRWRA: High Range Water Reducing Admixture

ITZ: Interfacial Transition Zone

L_{dz} : Length of the Debonding Zone

L_{ls} : Length of the Link Slab

LVDT: Linear Variable Differential Transformers

MAS: Maximum Aggregate Size

MEA: MgO-type expansive agent

MOR: Modulus of Rupture

MS: Mortar Sand

PE: Polyethylene Fiber

PVA: Poly-Vinyl-Alcohol Fiber

RC: Reinforced Concrete

SCC: Self-Consolidating Concrete

SCM: Supplementary Cementing Material

SS: Silica Sand

w/b: Water to Binder Ratio

CHAPTER ONE

INTRODUCTION

1.1. General

The fast-growing building industry leads to expand the demand on construction materials used in the production of concrete. Cement-based concrete has been used worldwide and considered as the second most consumed substance after water. Cementing materials utilized in a concrete are produced at an annual rate of approximately 3.5 billion tonnes in 2011, while concrete is produced at an annual rate of approximately 5.30 billion m³ in order to meet the high demand for the construction of infrastructures (Gao et al., 2016; Ng & Justnes, 2016). The vast implementation of cementitious materials and the development of high strength concrete in several projects around the world have been successfully achieved in the last decades due to its ability to be cast into different shapes, low cost and resistance to aggressive water. However, the brittle failure resulting from fracture in tension develops cracks which can lead to severe failure affecting the service life of the structure. The deteriorated structures will also require a constant maintenance with time. Thus, research development started taking another direction towards the development of a new technology in order to enhance the concrete material properties and rehabilitation of existing infrastructures, and also the construction of new infrastructures against multi-hazards.

Therefore, a new composite with fiber reinforcement employed in cementitious materials was created in order to improve toughness and resistance in tension. Fiber reinforced concrete (FRC) is mainly characterized with tension-softening, quasi-brittle response due to developing controlled shrinkage cracks. However, the desired tensile ductility has not been attained. Hence, high-performance fiber-reinforced cementitious composites (HPFRCCs) were developed with a very high tensile ductility.

Engineered cementitious composite (ECC) is a high-performance fiber reinforced concrete which is designed in order to achieve high durability, ductility and strain capacity (300–500 times greater than the tensile strain capacity of normal concrete) as well as multiple micro-cracking characteristics (Li & Kanda, 1998; Li et al. 2002; Li, 2003; Suthiwarapirak et al., 2004).

Moreover, ECC has the ability to maintain the micro crack width to less than 60~80 μ m even under ultimate loading; this intrinsic material property can stimulate crack self-healing behaviour which improves the mechanical properties, durability and serviceability of structural elements (Sherir, 2012; Herbert and Li, 2013; Özbay, 2012). Carbonation of $\text{Ca}(\text{OH})_2$ in order to form CaCO_3 precipitation is considered the main factor to stimulate autogenous crack healing leading and recovery of mechanical properties (Ghosh, 2009; Wu et al., 2012; Sherir et al., 2017).

Comprehensive research has been conducted to develop green ECCs with self-healing capability through cement replacement by different types and sizes of local aggregates (silica sand or crushed sand), supplementary cementing materials (SCMs) (such as fly ash, slag, glass powder, volcanic ash, MgO etc.) and bacterial agents (Sahmaran et al., 2009; Hossain and Anwar, 2014; Maulin, 2012; Sherir, 2012; Sherir et al., 2016; Sherir et al., 2017; Bhaskar, 2016; Siad et al., 2017). The incorporation of micro-silica sand and crushed sand in ECC mixtures can sustain adequate stiffness and stability (Sherir, 2012, Maulin, 2012). The development of low cost green ECC mixtures with locally available aggregates instead of silica sand is important for construction of structures with enhanced durability and ductility and service life.

The superior structural performance of ECC makes it a substantial alternative for concrete in different structural applications in real life. The common design of highway bridges is multiple span steel or pre-stressed concrete girders simply supported over piers or abutments. The girders support concrete bridge decks. Mechanical expansion joints are installed at the end of the simple span deck to accommodate deck deformations caused by girder's deflection, creep effect, concrete shrinkage and temperature variations. However, mechanical expansion joints require constant maintenance due to their susceptibility to durability problems. The durability of bridge is eventually compromised by leakage of chloride-contaminated water and chemicals causing corrosion of steel girders (Kim et al., 2004; Lam et al., 2008; Hossain and Anwar, 2014; Hossain et al., 2016). The continuous and accumulative debris leads to deterioration of joint functionality, and thus deterioration in the bridge deck and substructure. Figure 1.1 shows the typical corrosion damage accompanied with mechanical joints.

Figure 1.1: Typical Corroded Mechanical Joint in Bridge Deck and Joint Free Bridge Deck with Link Slab (Hossain & Anwar, 2014; Hossain et al., 2016)

The negative economic impact accompanied by mechanical joints through the different phases of the bridge lifecycle has been a major concern to be addressed by the transportation authorities (Lepech & Li, 2009b). Several design approaches have been proposed and instituted in order to improve the durability performance of the bridge by eliminating the mechanical expansion joints (Au et al., 2013). The first approach is the development of a continuous deck system over the pier supports using haunched deck while keeping the girders as simply supported span. The second approach is the construction of semi-integral deck system with covering the girders at the piers in a transverse concrete diaphragm. The third approach is the implementation of debonded concrete link slab for providing a continuous simply supported deck slab. According to the recent applications of the joint free bridge deck, it was found to be the most economical and efficient rehabilitation strategy in comparison to other proposed approaches (Alampalli & Yannotti, 1998).

The link slab can be employed by removing the mechanical expansion joint and connecting the two adjacent decks to produce joint-free bridges (Figure 1.1). Figure 1.2 shows the schematic of link slab components according to the proposed design method by Caner and Zia (1998). The overall length of the link slab consists of the debonding and transition zone. The length of debonding zone (where a debonding mechanism is placed on the top flange of the girder with the removal of the shear connectors) is limited to 5% of the adjacent bridge span to reduce the stiffness without affecting the deformation behaviour of the structure. The transition zone represents the segment between the bridge deck and the debonding zone. For minimizing and diverting the stress

concentration from the potentially weak debonding zone, transition zone is designed with a 2.5% increase in length and 50% additional shear connectors than required by AASHTO design code (Caner & Zia, 1998; Qian et al., 2009).

Figure 1.2: Schematic of Link Slab Showing Components

The use of normal concrete in link slabs did not inhibit the development of large tensile cracks which leads to degradation of the performance due to salt water leakage and contact with the steel girder and rebar (Caner & Zia, 1998). In order to enhance the durability and structural performance of the bridge deck, ECC link slab can be employed as a better substitute to conventional self-consolidating concrete (SCC).

1.2 Research Significance

Billions of dollars are spent every year for rehabilitation and maintenance of bridge structures in North America. This can be attributed to the continuous exposure to traffic volumes, stress levels, environmental and time-dependent damages. Thus, the durability of these for reinforced and pre-stressed concrete bridges will significantly reduce creating a huge concern for highway transportation authorities. With decreasing budget allocations for bridge maintenance and rehabilitation, it became crucial to develop a new technique for enhancing durability.

Researchers' main focus was finding an alternative to the leaking expansion joints by forming a joint-less multi-span bridge using ECC link slabs which are characterized by high durability and service life due to strain hardening and multiple cracking characteristics (Caner and Zia, 1998, Kim et al., 2004, Hossain & Anwar, 2014).

Researchers at Ryerson University have started developing sustainable and cost-effective ECC mixtures for construction applications (including joint-free bridges with ECC link slab) over the last few years incorporating different types of aggregates, supplementary cementitious material (SCM) along with the addition of MgO and bacterial agent in order to enhance mechanical/durability properties (including superior ductility, strain-hardening, crack width control characteristics) and self-healing potential (to achieve crack-healing) (Shamaram et al., 2010, Sherir, 2012, Maulin, 2012; Hossain & Anwar, 2014; Gahtrehsamani, 2015; Sherir, 2016). Mechanical and durability performance of ECC materials has been investigated extensively over the years including self-healing potential related to the recovery of mechanical and durability properties over the age (Shamaram et al., 2010, Sherir, 2012, Maulin, 2012; Hossain & Anwar, 2014; Gahtrehsamani, 2015; Sherir, 2016; Bhaskar, 2016). Limited research studies are also conducted on the structural performance of ECC link slab under fatigue flexural loading (Hossain et al., 2016; Hossain et al., 2015). The recovery of strength and deflection capacities of ECC link slab subjected to monotonic loading to failure was investigated to some extent (Sherir, 2016). However, there is an urgent need to study the strength and deformation characteristics of ECC mixtures under fatigue flexural loading including the study of self-healing potential of ECC mixtures when subjected to fatigue loading. In addition, the structural performance of ECC link slab subjected to fatigue loading at various stress levels and fatigue cycles needs to be investigated along with the self-healing potential of ECC link slabs subjected to fatigue loading. The proposed research is planned to address these topics.

1.3 Research Objectives and Scopes

The main objective this research is to conduct a detailed investigation on the flexural fatigue performance of beams and link slabs made of various ECC mixtures incorporating fly ash at various cement replacement ratio, types and sizes of aggregates and MgO additive. This type of investigation can open the gates for applications of such emerging ECC mixtures in different engineering applications characterized with enhanced durability and sustainability features associated with low costs of construction and maintenance.

The main objective is achieved through:

- Evaluating the strength properties (such as compressive and flexural strength) and fatigue loading performance of five different ECC mixtures incorporated with different content of fly ash class-F, size/type of sand aggregates (such as silica sand and mortar sand) and addition of MgO-type expansive agent.
- Evaluating flexure fatigue performance and self-healing potential of beams made of five ECC mixtures. Three approaches were used. In the first approach, ECC beams are tested by applying 400000 cycles at a stress level of 50% of ultimate load at the age of 28 days and then loaded monotonically to failure. In the second approach (fatigue loading is applied on the beams at the age of 28 days continuously at increasing stress levels till reached 100% stress level) - ECC beams are tested by applying 50000 cycles at a stress level of 25% of ultimate load, followed by another 50000 cycles at a stress level of 50% of the ultimate load, then another 50000 cycles at stress level of 75% of the ultimate load and finally 100% stress level when the beams failed. The third approach deals with the study of self-healing potential of ECC mixtures where beam specimens are subjected to fatigue loading (at constant stress level of 50%) - loading the specimens for 50000 at the age of 28 days and then the specimens are left for vapor curing for 2 months (to be self-healed), then loaded for 100000 cycles and left again for vapor curing for another 1 month for self-healing and finally loaded for another 100000 cycles before loading to failure under monotonic loading at the age of 118 days to study the post-fatigue behavior of healed-fatigued specimens. The flexural fatigue performance was judged based on flexural stress-deflection response, stiffness degradation, deflection evolution, cracking, energy absorbing capacity and post-fatigued residual strength, deflection, stiffness and energy absorbing capacity. The self-healing performance was evaluated through observation of crack-healing, recovery of strength, and deflection capacity as well as residual strength, deflection and energy absorbing capacity.
- Evaluating the pre-fatigue structural performance of 1/4th scale link slabs made of five different ECC mixtures compared to their self-consolidating concrete (SCC) counterparts under flexural monotonic loading to failure. The performance is analyzed based on load-

deformation/moment-rotation responses, stiffness, crack characterization, steel/concrete strain developments, ductility and energy absorbing capacity.

- Evaluating fatigue and post-fatigue structural performance of self-healed 1/4th scale link slab specimens made of five different ECC mixtures compared to their SCC counterparts. Specimens are loaded (at constant fatigue stress level of $55\% \pm 15\%$) and self-healed as follows - first loaded 50000 cycles at the age of 28 days then vapour cured for 2 months, after that loaded for another 100000 cycles then cured for 1 month then loaded for 100000 cycles before loaded to failure at the age 118 days under monotonic loading. The performance is analyzed based on load-deformation/moment-rotation responses, stiffness, crack characterization, steel/concrete strain developments, ductility and energy absorbing capacity. The self-healing performance is evaluated through observation of crack-healing, recovery of strength, and deflection capacity as well as residual strength, deflection and energy absorbing capacity.
- Making recommendations on fatigue performance and self-healing potential of ECC mixtures for practical construction application such as joint-free bridges with link slabs.

1.4 Thesis Outline

- Chapter 1 introduces engineered cementitious composite (ECC) along with its application for bridge deck link slab and self-healing behaviour. It also describes significance, objectives and scope of the research.
- Chapter 2 presents literature review of some self-healing studies in ECC, ECC and its design considerations, materials and properties and structural application of joint-free bridge deck with link slab highlighting design and previous research studies.
- Chapter 3 describes the experimental program including ECC and SCC mixtures and material properties, link slab geometric dimensioning and model construction along with the experimental instrumentation and testing procedures.

- Chapter 4 presents the results of the experimental studies including mechanical properties, fatigue flexure performance and evaluation of self-healing behaviour of ECC beams.
- Chapter 5 presents the results of the structural performance and evaluation of self-healing behaviour on 1/4th scale ECC link slabs subjected to monotonic and fatigue loading.
- Chapter 6 summarizes the conclusions of the research and recommendations for future research studies.

CHAPTER TWO

LITERATURE REVIEW AND BACKGROUND

2.1 Introduction

Cementitious materials are widely used and implemented in infrastructure construction materials all over the world; due to its high compressive strength and low cost. However, deterioration and crack formation is unavoidable throughout the lifecycle of the concrete structure, especially with the limited tensile strength of concrete. There are several factors affecting the crack formation such as extreme loading, restrained shrinkage and harsh environmental conditions. The brittle failure in the cementitious materials has lead researchers to start developing concepts about increasing the strength of the concrete by increasing the compressive strength. However, the structural elements will stay weak in tension and fail by fracture in tension. This major concern has driven researchers and authorities towards the improvement of the tensile ductility property of the material in order to enhance the strength and reduce rehabilitation of existing infrastructures when subjected to hazards.

Hence, recent researches have introduced a new composite with fiber reinforcement employed in cementitious materials characterized with high toughness, resistance in tension, high tensile ductility, multi-cracking and self-healing characteristics. This composite is known as Engineered Cementitious Composite (ECC) and can be a material for the future to construct durable and crack-free structures with enhanced service life (Li et al., 2002; Hossain et al., 2016).

This chapter covers the literature review on ECC material and its design considerations, materials and properties, research studies on mechanical and durability properties, introduction of link slab in joint-free bridge construction, research studies on ECC link slab and finally, concluding with a review summary.

2.2 Engineered Cementitious Composites (ECC)

Engineered cementitious composite (ECC) is a class of high performance fiber reinforced concrete which is designed in order to achieve high durability, high ductility and strain capacity varying between 3 to 5% under tensile loading with only 2% fiber content by volume (Li & Kanda, 1998; Li et al., 2002; Li, 2003; Suthiwarapirak et al., 2004). The formation of the first cracking in ECCs

initiates the strain hardening that leads to the formation of multiple micro- cracks. The incorporation of 2% of fiber content leads to a vital behavior of high tensile ductility and strain capacity of 300–500 times greater than the tensile strain capacity of normal concrete. Figure 2.1 shows ductile behaviour of typical ECC mixture.

Figure 2.1: Ductile Behavior and Bending Capacity of ECC (Li, 2011)

Extensive research has been conducted at Ryerson University over the years to develop green cost effective ECCs by incorporating different types and sizes of local aggregates (silica sand or crushed sand), supplementary cementing materials (SCMs) such as fly ash, slag, waste glass powder, volcanic ash etc.) and self-healing agents (MgO and bacterial species) optimizing economic and environmental benefits as well as mechanical, durability and self-healing properties (Sahmaran et al., 2009; Hossain and Anwar, 2014; Maulin, 2012; Sherir, 2012; Sherir et al., 2016, Sheir et al., 2017, Siad et al., 2017; Bhaskar, 2016). The use of high volume of fly ash in ECC mixtures reduces the matrix toughness which must be controlled to form multiple cracking before reaching maximum stress, and also leads to fiber pullout rather than rupture during debonding which significantly enhanced the ductility of the mix. In addition, the unhydrated fly ash particles may also improve the compactness and the frictional bonding between fiber and matrix. Therefore, steady-state crack opening will be reduced and durability and sustainability of the ECC mixture will be enhanced (Maulin, 2012; Sherir, 2012; Sherir, et al., 2016; Sahmran et al., 2009; Wang and Li, 2007). The particle size of aggregate significantly affects the fracture matrix toughness, clumping and interaction of fibers which reduces the ductility of the ECC mixture (Sahmaran et

al., 2009). The incorporation of micro-silica sand and crushed sand in ECC mixtures can improve and impart adequate stiffness and stability (Sherir, 2012, Maulin, 2012). The development of low cost green ECC mixtures with locally available aggregates instead of silica sand is important for construction of structures with enhanced durability and ductility and service life.

2.2.1 ECC Design Considerations

The main principle behind ECC design is maximizing the fiber/matrix interaction and interfacial transition zone in order to form multiple microcracks. This can be achieved through developing cracks below the maximum fiber-bridging stress and allowing the crack length to extend with controlling the width of the crack. The multiple crack formation enhances the capacity of ECC under load capacity through developing tensile ductility and strain hardening response (Li et al., 2002; Li, 2011; Li and Kanda, 1998).

The tensile strain hardening behaviour of ECC is achieved by the multiple non-localized cracks that sustain and pass the load without any catastrophic damage in the structure regardless the length of the crack and type of the loading (Li, 1997, 2003). ECC is designed for several types of applications such as ECC largescale on-site construction applications, high-early-strength ECC applications, lightweight ECC applications, green ECC applications and Self-healing ECC applications; that show the ability of ECC to heal cracks over time (Tittelboom, 2013). It was proved by the gradual reduction in permeability of cracked concrete as water penetrates through the cracks; which is caused by the crack width shrinking as healing occurs.

There are two different criteria needed to be satisfied in order to achieve strain-hardening and the multiple-cracking behavior; the first criterion is the First Crack Strength (FCS) and the second is the energy criterion or the Steady State Crack (SSC) criterion. The maximum bridging stress and the first crack strength of ECC are influenced by the bridging properties and toughness of the matrix. Moreover, the First Crack Strength criterion helps in initiating micro cracks and ensuring that the maximum capacity of fiber bridging is greater than the tensile loads applied to create the micro cracks intended for the formation of more micro cracks in the composite. By not applying this criterion, multiple-cracking behavior ends at early stages. After initiating the first cracking, steady state flat crack will propagate steadily along the matrix under constant tensile load, in other words, the crack length extends without the opening or the width of the crack growing

wider. Hence, the second criterion proposes the change of the crack type from the Griffith-type crack (tension-softening behavior of FRC) to steady state crack type (strain-hardening behavior of ECC). The application of the second criterion implies that the energy absorbed by the fibers located near the crack tip or in the crack opening should be equal or greater than that released from the load applied. To sum up, Kanda and Li (1998) had proposed that the micro-cracking and strain hardening behavior of ECCs can be obtained by the strength and energy criteria (Kanda & Li, 1998). After the formation of the first crack, the strain hardening behavior leads to increasing the tensile stress of ECC where the width of the micro cracking remains below 80 μm at the ultimate loading. However, the steady-state cracking criterion remains independent to the amount of fiber in the mixture (Ranade et al., 2014).

2.2.2 ECC Ingredients and Their Influence on ECC Properties Based on Past Research Studies

ECC is typically composed of cement, aggregates (silica sand and other sand), around 2% polyvinyl alcohol (PVA) fiber, supplementary cementing materials (fly ash, slag etc.), superplasticizer and water.

2.2.2.1 Role of Aggregates

Aggregates are considered to be economic filler and a framework in the paste material where the aggregates enhance the dimensional stability and restrain the large shrinkage movements in the paste. However, the size of the aggregates affects several characteristics; for instance, the use of coarse aggregates will create fibers clump and lead to tough ECC mixtures that cause delay in the initiation of the first crack and failure of the multiple crack behavior due to the low tensile ductility. However, fine aggregates are commonly used for ECC mixtures with aggregate to binder ratio (A/B) of 0.36 in order to attain the anticipated multiple cracking behavior and the strain hardening (Li et al., 1995). The binder matrix includes cementitious material such as cement, fly ash, etc. The commonly used fine aggregate in ECC is silica sand with a maximum grain size of 250 μm and a mean size of 110 μm . Hence, the size of the aggregates has an imperative impact on the properties of mixture such as workability, volume stability, stiffness and fiber dispersibility (Sherir et al., 2014).

2.2.2.2 Role of Fly Ash

In the last few decades, supplementary cementing materials such as fly ash have been widely used in the applications of ECC. Fly ash is a by-product of the coal power plant. The use of fly ash helps in reducing the flexural stresses and crack tip matrix toughness which results by the reduction of the fiber/matrix interfacial chemical bond, it can improve the ductility by enhancing the fiber pullout (Yang & Li, 2007).

It was previously noted that the use of coarse aggregate is not effective in ECC applications which results in a larger cement content. Hence, fly ash replacement will help in limiting the environmental impact from such by-products. In addition, it has been reported that using high volumes of class-F fly ash reduce the matrix toughness and minimize the interface bond between the fiber and the matrix in favour of enhancing the tensile ductility and boosting the tensile strain capacity (Wang and Li, 2007, Sahmaran & Li, 2009).

2.2.2.3 Role of Fibers

Polyvinyl Alcohol (PVA) fibers are considered to be the most common type of fiber used in ECCs. The PVA fibers with a diameter of 39 μm , a length of 6-12 mm, strength of 1620 MPa and a modulus of elasticity of 42.8 GPa are normally used. Furthermore, PVA fibers can elongate up to 6.0% of their original length. Those properties correspond to those that help in carrying the tensile stresses, strain-hardening performance and increasing the matrix's toughness due to the fiber bridging behavior. Fiber bridging behavior of ECC is based on the micromechanics of the fiber/matrix interface where fibers transfer the stresses and maintain the low crack widths (Li et al., 2001; Suthiwarapirak et al., 2004). In addition, the surface of PVA fibers is coated with 1.2% mass of oil that reduces the interfacial bonding properties between the fiber and matrix and increases the tensile strain capacity of the mix; hence, fibers tend to rupture under tensile stresses (Li et al., 2003).

2.2.3 Mechanical Properties

2.2.3.1 Compressive Strength

In general, ECC exhibits a compressive strength similar to high strength concrete ranging between 40 and 60 MPa (Sherir, 2012; Hossain & Sherir, 2014). It was noted that the increase of the fly ash cement replacement ratio up to ($\text{FA/C} = 4.2$) has a negative influence on the compressive strength.

This is attributed to the slow pozzolanic reaction and hydration rate of the fly ash compared to the cement (Mindess et al., 2003; Sahmaran et al., 2008; Sahmaran et al., 2009; Hossain and Sherir, 2014). Sherir et al. (2012) showed that the increase in Calcium Oxide (CaO) content increases the compressive strength. It was also shown that the aggregate size has a slight positive influence the compressive strength through increasing the volume of interfacial transition zone between the aggregate surface and the matrix.

2.2.3.2 Flexural Strength and Deflection Capacity

The flexural strength of the ECC varies from 5 to 16 MPa depending on the mix design parameters (Wang & Li, 2003). Several experimental investigations by Hossain and Sherir (2014) and Sahmaran et al. (2009) revealed that the increase of fly ash to cement ratio (FA/C) reduces the ultimate flexural strength and improves the bending capacity of ECC mixtures. This might be attributed to the improved frictional bond, reduced toughness and chemical bond between PVA fiber/matrix interface chemical bond and the matrix toughness (Wang and Li, 2007; Sahmaran et al., 2009; Hossain and Sherir, 2014; Li et al., 1995).

2.2.3.3 Fatigue Flexure Capacity

Several research studies have investigated the mechanical properties and the failure mechanisms of ECCs under monotonic loadings, while only few studies have focused on the performance of ECC when subjected to fatigue loading. It is vital to investigate the fatigue flexure behaviour in order to examine the performance of link slab application in bridge structure.

The first study developed on fatigue performance and properties of the polyethylene fiber based ECC compared to normal concrete was done by Matsumoto (1998). Further research on the fatigue performance of rigid pavement consists of ECC/concrete overlay has been conducted on the superior performance of ECC/concrete overlay due to doubled load and deformation capacity compared to concrete/concrete overlay (Matsumoto, 1998, Zhang & Li, 2002).

Experimental investigations on flexural fatigue performance were conducted at Ryerson University. Sherir (2012) and Hossain et al. (2016) studied the influence of type and size of aggregates (silica sand and crushed sand) as well as different fly ash content as replacement of cement on the fatigue flexural performance of ECC beams. The flexural fatigue performance study was divided into two main approaches; the fatigue stress level approach (40%, 55% and 70%) and the fatigue cycles approach (100000, 200000, 300000, 1 million). In conclusion, ECC mixtures

exhibited multiple cracking behavior, it was also noted that the increase of fatigue stress levels and cycles resulted in an increase in mid-span deflection evolution, number of micro cracks and damage level. Crushed sand showed better fatigue performance compare to silica sand.

2.3 Previous Research on Self-healing Properties of ECC

During different stages of the life of the concrete structure, cracks can occur due to the several factors such as poor design and construction, extreme loading, restrained shrinkage and harsh environmental exposure. Crack self-healing behaviour is very desirable since concrete can recover its loss of strength, safety and durability performance. The phenomenon of autogenous self-healing when cementitious materials have the ability to heal their cracks can be shown through less crack widths and water permeability along with a recovery in the mechanical properties over time (Yang, 2005; Sahmaran et al., 2013). The closely spaced small cracks developed in ECC are caused due to many different reasons such as errors in structural design, poor construction and harsh environmental conditions. Moreover, those cracks may allow ingress and leakage of aggressive agents which might significantly reduce the long-term durability and lead to the deterioration in the structure. As a result, innovative technologies associated with self-healing were discovered in order to enhance the ability of ECC to heal its own cracks and thus the durability of the structure (Mindess et al., 2003; Ghosh, 2009). In general, the small crack widths (less than 60 μm) in ECCs and specific environmental conditions stimulate the autogenous crack healing which can occur through the continuous reaction of un-hydrated particles and precipitation of CaCO_3 (Ghosh, 2009; Mihashi and Nishiwaki, 2012; Wu et al., 2012; Herbert and Li, 2013).

ECC has the ability to maintain the micro crack width to less than 60~80 μm even under ultimate loading; this intrinsic material property can stimulate crack self-healing behaviour which improves the mechanical properties, durability and serviceability of structural elements (Sherir, 2012; Herbert and Li, 2013; Özbay, 2012, Sheir et al., 2016, 2017). Early studies developed by the French Academy of Science in 1836 have discovered the phenomenon of crack-healing in concrete. It was proven that self-healing property occurs when concrete is exposed to water (Mihashi & Nishiwaki, 2012). This phenomenon can be attributed to several factors such as carbonation of calcium hydroxide ($\text{Ca}(\text{OH})_2$) from the hydrated cement pastes to produce calcium carbonate (CaCO_3) precipitation, swelling of Calcium Silicate Hydrates (C-S-H), or further hydration of un-hydrated cementitious particles (Heide and Schlangen, 2007; Şahmaran et al., 2008; Schlangen et al., 2006;

Tittelboom and De Belie, 2013). According to (Edvardsen, 1999), carbonation of calcium hydroxide ($\text{Ca}(\text{OH})_2$) in order to form (CaCO_3) precipitation is considered the main factor to stimulate autogenous crack healing leading to a reduction in crack width and recovery in the mechanical properties (Ghosh, 2009; Wu et al., 2012).

2.3.1 The Effect of Calcium Carbonate (CaCO_3)

Several researchers investigated the effect of several factors on the formation of CaCO_3 such as the presence of small crack width, high water temperature, chemical ions (carbon dioxide and calcium ions Ca^{2+}) and high pH value of water (Edvardsen, 1999; Mihashi and Nishiwaki, 2012). Earlier studies showed that the precipitation of CaCO_3 can occur a pH of ~ 9.8 which is resulted from the average pH of pore water in hardened cement paste (~ 13.5 pH) and water flow in the small cracks (5.5 - 7.5 pH). The self-healing mechanism is stimulated from the reaction of CO_3^{-2} ions dissolved in water and Ca^{2+} ions present in the cement paste. The following equations (Eqs. 2.1 and 2.2) represents the reaction of calcite crystals that fill in the cracks.

Edvardsen (1999) studied two different techniques of CaCO_3 formation for filling the cracks. The first technique is called surface controlled crystal growth and characterized by the formation of carbonates (CO_3^{-2} ions) resulted from the water flow H_2O and the CO_2 from the surrounded air as shown in Eq.1. After that, Ca^{+2} ions will be forced to leave $\text{Ca}(\text{OH})_2$ and C-S-H located at hardened cement paste near the cracks in order to react with CO_3^{-2} ions. Thus, CaCO_3 crystals that does not dissolve well in water will be formed and hence stimulate autogenous crack healing. The surface controlled crystal growth which occurs within the first few days of hydration is responsible for the formation of about 80% of CaCO_3 crystal. After that, the second technique which is called diffusion controlled crystal growth is slower than the surface technique. In this technique, Ca^{+2} ions diffuse from deeper locations of hardened cement paste and react with CO_3^{-2} ions leading to more calcite precipitation (Edvardsen, 1999).

2.3.2 The Effect of Temperature

Reinhardt and Jooss (2003) have investigated the self-healing behaviour of cracked concrete at temperatures between 20 and 80 °C. it was shown that at an ambient temperature of 20 °C, the

initial flow rate of an average crack width of 0.05 mm has dropped to approximately 45% after 25 hours, while the initial flow rate of an average crack width of 0.15 mm has dropped to approximately 75% after 25 hours. However, when temperature is increased up to 80 °C, the initial flow rates of crack widths of 0.05 and 0.15 mm have dropped to 10% and 59%, respectively after 25h. The results of this study showed that the decrease in flow rate depends on water temperature and crack width. Higher water temperatures and smaller cracks promote the self-healing property faster (Reinhardt and Jooss, 2003).

2.3.3 The Effect of Crack Width

The crack width control is considered as a key factor in the process of self-healing. Recent researches have conducted the effect of marine and natural environmental conditions on the crack width. Studies by Christophe and Raoul (1997) and Li and Li (2011) have revealed that the crack width was increasing (50 mm to 100 mm) when concrete is exposed to marine environment conditions, this may be attributed to the presence of chloride ions that accelerates the leaching of calcium hydroxide ions which reduces fiber/matrix interfacial bonds and thus the fiber bridging capacity is reduced (Christophe and Raoul, 1997; Li and Li, 2011). However, ECC showed a superior performance with respect to tensile strain capacity ($< 2.5\%$) and multiple cracking behavior even when exposed to seawater. Even under chloride exposure environment, ECC mixtures can stimulate self-healing by opening the way for the un-hydrated cement particles to produce C-S-H products to fill the cracks (Heide and Schlangen, 2007; Schlangen et al., 2006).

In the case of natural environment exposure such as rainfall events, the dissolution of $\text{Ca}(\text{OH})_2$ is faster than the kinetics of the pozzolanic reactions that consume and convert $\text{Ca}(\text{OH})_2$ to C-S-H products. Thus, ECC mixtures with tighter crack widths and higher content of fly ash are required for such conditions in order to exploit the rain water in producing C-S-H products which help concrete to recover its strength and heal its cracks (Herbert and Li, 2013).

2.3.4 The Effect of PVA Fibers

Several studies have been conducted in order to find the suitable synthetic fiber incorporated in fiber reinforced cementitious composites (FRCC) in self-healing. It was shown that the high polarity of PVA fibers promotes self-healing due to the presence of hydroxide ions OH^- that attract Ca^{2+} ions and then precipitate CaCO_3 on the crack walls (Homma et al., 2009; Nishiwaki et al., 2012).

2.3.5 The Effect of Supplementary Cementing Materials

In order to enhance the self-healing behaviour of cementitious composites incorporating fly ash, water or other form curing curing should be applied for a long time (Na et al., 2012; Sherir et al. 2016). A study by Yang et al. (2007) showed that using high volumes of fly ash as a cement replacement and high water/binder (w/b) ratio lead to more consumption of Ca(OH)_2 that comes from the cement used in the hydraulic and pozzolanic reactions. Thus, more C-S-H products which promote the self-healing behaviour are produced and the long-term mechanical and durability properties are enhanced (Yang et al., 2007; Sherir et al., 2016; Alyousif, 2016). Comprehensive study by Sherir et al. (2017) and Alyousif (2016) showed that the use of fly ash was helpful in the recovery of mechanical and durability (water permeability and sorptivity characteristics) of ECC through crack closing and micro-structural improvement due to formation of CaCO_3 based healing products.

2.3.6 The Effect of MgO-Type Expansive Agent

In 1970s, Baishan dam in China has opened the gates for engineers and researchers to develop new techniques for temperature-control measures through the addition of MgO. This phenomenon was discovered when no significant concrete cracking was observed even under severe hot and cold weather conditions. It was found that the main possible reason behind this phenomenon is MgO content in the cement, as MgO was discovered to be a reactive agent that enhances long-term properties through its expansive effect starting from 7 days to 1000 days after concrete is hardened (Du, 2005; Gao et al., 2007; Mo et al., 2012; Moradpour et al., 2013; Zheng et al., 1991).

The expansion behaviour of MgO concrete compensates the heat shrinkage in concrete through two types of stresses generated; the first one is the pre-compressive stresses which are generated at early ages and distribute in the concrete in order to partially compensate the tensile stresses, while the second type of stresses is related to the long-term expansion effect in order to reduce concrete cracking through relaxing tensile and compressive stresses at later ages (Du, 2005; Moradpour et al., 2013).

Sherir (2016) and Sherir et al. (2016, 2017) investigated the effect of MgO expansive additives on the self-healing behaviour of ECC. The best calcination system was determined for improving the self-healing technique of ECC according to the hydration activity and expansion effect of MgO. Burning temperature is the main factor that controls the expansion effect of MgO and its rate of

hydration. Thus, the best calcination system for the production of ECC mixtures was found to be “900°C_2 hours of holding time_45 µm particle size” by lightly burn 45 µm particle size of Magnesium Carbonate (MgCO_3) at 900°C for 2 hours inside the kiln (Sherir, 2016). ECC made of fly ash and MgO agent showed significant recovery of mechanical and durability (water permeability and sorptivity characteristics) through crack closing and micro-structural improvement due to formation and deposition of Ca and Mg based healing compounds products within the crack walls.

2.4 Link Slab for Joint Free Bridge Deck Construction, Design and Research Studies Including Self-healing

In 1970's, the common design of highway bridges is multiple span steel or pre-stressed concrete girders simply supported over piers or abutments. Mechanical expansion joints were provided at the end of the simple span deck to allow deck free deformations caused by girder's deflection, creep effect, concrete shrinkage and temperature variations (Kim et al. 2004, Lam et al., 2008). However, the durability problems and excessive costs of maintenance associated with the mechanical joints motivated engineers and researchers to substitute the previous bridge design to a joint-free bridge structure using link slabs as illustrated in Figure 1.1 (Caner & Zia, 1998).

Caner and Zia (1998) and other researchers had conducted extensive experimental investigations in order to develop and analyze the design methodologies and guidelines of joint-free bridge decks for the link slab (Caner & Zia, 1998; Gilani, 2001; Lepech & Li, 2009a; Hossain and Anwar, 2014; Hossain et al. 2016). After testing link slabs under monotonic loading, it was noted that the moment distribution for a two span simply supported bridge structure leads to a negative moment distribution and a downward deflected shape developed at the link slab. This finding revealed that under service conditions along with creep, shrinkage and temperature variation effects, flexural crack formations are developed at the top face of the link slab due to the development of tensile stresses (Kim et al. 2004; Caner & Zia, 1998). In order to reduce the stiffness of link slabs, the debonding length (as shown in Figure 1.2) was limited to 5% of the adjacent bridge as per Equation 2.3, and also the reinforcing ratio was minimized at girder's end rotation of 0.0015 radian (AASHTO, 2012).

$$L_{dz} = 5\% (L_1 + L_2) \quad (2.3)$$

Where L_{dz} : the total length of the debonding zone; L_1 , L_2 : the lengths of adjacent bridge spans.

After an extensive investigation on the effective debonding zone length, Qian et al. (2009) was able to finalize and determine the effective length of the ECC link slab. The link slab section is designed between the inflection points which varies between 0 and 20% of the adjacent span's length based on the stiffness. Thus, Qian et al. (2009) suggested a total length of the debonding zone (Figure 1.2) of 7.5% of adjacent span's length as shown in Equation 2.4.

$$L_{dz} = 7.5\% (L_1 + L_2) \quad (2.4)$$

The length of the transition zone (Figure 1.2) is considered to be 2.5% of the span's length in accordance with Qian et al. (2009) in order to shift the stress concentration to the debonding zone. The design procedure developed by Caner & Zia (1998) and Kim et al. (2004) has been introduced to the field application in the State of Michigan in 2005 (Li et al., 2005).

As per previous research studies, SCC link slabs did not perform well under large tensile loading due to the development of large tensile crack widths. Several studies on ECC link slab showed its ability to meet the structural requirements due ECC's tensile strain capacity, micro-cracking behaviour and high ductility (Kim et al., 2004; Li et al., 2003; Hossain et al., 2016). Kim et al. (2004), Hossain et al. (2015, 2016) conducted research on monotonic and fatigue performance of ECC link slab and the results have proved the suitability of utilizing ECC materials in link slabs for developing a low structural stiffness and controlling the crack widths within the limits even at ultimate loading which can extend the life cycle of the bridge structure.

For the link slab applications, research studies have been conducted in order to develop more economical and greener ECC mixture using locally available crushed sand and supplementary cementitious materials with self-healing capability (Sahmaran et al., 2009; Maulin, 2012; Sherir, 2012; Hossain and Anwar, 2014; Hossain et al., 2016; Sherir, 2016).

Hossain et al. (2016, 2015) and Shirin (2015) have conducted research on the of 1/4th scale link slab and 1/6th scale full joint-free bridge deck specimens in order to understand structural behaviour and to develop design guidelines using different ECC mixtures. The monotonic and fatigue performance of ECC link slabs incorporated ECCs (with type and size of aggregates as well as different types and content of fly ash and content) were evaluated and compared to SCC

link slabs based on the load-deformation/moment-rotation responses, strain developments, cracking patterns, ductility and energy absorption capacity. Research has shown superior performance of ECC link slabs compared to their SCC counterparts through exhibiting high residual strength and energy absorbing capacity. Experimental test results were verified with the requirements of serviceability and design specifications in terms of allowable ductility and rotational capacity which were found satisfactory.

More recently, Sherir (2016) investigated self-healing behaviour of MgO-ECC based link slab compared to their SCC counterparts through assessing the crack healing with strength and deflection recovery of cracked/damaged specimens for one year. ECC-MgO link slab showed superior crack-healing showing better strength and deflection recovery compared to their SCC counterparts.

In a recent study by Hossain et al. (2016), ECC link slabs showed superior long-term creep behaviour (under sustained loading) over five year study period compared to their SCC counterparts in terms deflection evolution, steel-concrete strain evolution, residual strength and deflection, deflection-strain recovery at unloading and crack formation (multiple micro-cracking and crack width).

2.5 Review Conclusion

Progress has been made in the development of ECC mixtures, however, still there is a scope to develop new greener sustainable ECC mixtures with self-healing and evaluate their mechanical and durability properties in structural applications. There is need to study the strength and deformation characteristics of ECC mixtures under fatigue flexural loading including their self-healing potential in recovering mechanical properties under fatigue loading. Limited research studies are also conducted on the structural and self-healing performance of ECC link slab under fatigue flexural loading which needs to be investigated. The proposed research concentrating on these aspects will significantly contribute to the existing self-healing technology of ECC and the construction of ECC link slab with superior self-healing capability.

CHAPTER THREE

EXPERIMENTAL PROGRAMS

3.1 Introduction

This Chapter described the experimental investigations to study the development and recovery (due to self-healing) of mechanical properties and the structural performance of engineered cementitious composite (ECC) beams and ECC or self-consolidating concrete (SCC) link slabs subjected to monotonic and flexural fatigue loading. Five ECC mixtures containing different amounts of Class-F fly ash (FA), types of sand as aggregates and 5% MgO powder amount were used in this study in addition to a control self-consolidating concrete (SCC) mixture. Different testing approaches were used to evaluate the influences of size of aggregate, amount of Class-F fly ash and MgO addition in ECC mixtures. Material properties, ECC and SCC mix proportions, mixing procedures, design of link slabs, specimen preparation and casting and testing procedures are described.

3.2 Materials and Mixture Proportions

3.2.1 ECC Mix Designs

The five ECC mixtures developed at Ryerson University were used in this study and this mix designs are presented in Table 3.1. The mixture proportions for the five ECC mixtures contained 55% and 70% (FA/C ratio of 1.2 and 2.2, respectively) Class-F fly ash (as cement replacement), types of sand aggregates (silica sand 'SS' or mortar sand 'MS') and 5% MgO powder as replacement of binder (C+FA). The water to binder (w/b) and aggregate/binder ratios were kept constant for all ECC mixtures. The fly ash content was reduced from 55% (FA/binder before MgO addition) to 50% (FA/binder after MgO addition) in MgO based ECC mixture. Details of two SS based (designated as F_1.2_SS and F_2.2_SS), two MS based (designated as F_1.2_MS and F_2.2_MS) and one MgO-SS based (designated as F_1.2_SS_MgO) ECC mixtures are presented in Table 3.1.

Table 3.1: ECC Mixture Proportions

Mixture ID	Ingredients, kg/m ³							FA/C	w/b
	Water	Cement	FA	Sand	PVA	HRWRA	MgO		
F 1.2 SS	331	570	684	455	26	5.4	-	1.2	0.27
F 2.2 SS	327	386	847	448	26	4.15	-	2.2	0.27
F 1.2 MS	327	559	671	446	26	5.4	-	1.2	0.27
F 2.2 MS	319	376	825	436	26	4.2	-	2.2	0.27
F 1.2 SS MgO	331	570	621	455	26	5.4	63	1.09	0.27

*HRWRA: High range water reducing admixture, C: Cement, FA: Class-F fly ash, w/b: water to binder ratio

3.2.2 SCC Mix Design

The commercial SCC used as a control mix for the comparative study of the structural performance of 1/4th scale ECC and SCC link slabs. Each bag of the SCC ready mix consists of a 30-kg dry content that required a 2.4 liter of water to produce the mixture. The dry content included Portland cement, slag, admixtures, coarse and fine aggregate.

3.2.3 ECC and SCC Material Properties

The materials used in the production of ECC mixtures were micro-silica sand (MS) and mortar sand (SS) with maximum grain sizes of 0.3 mm and 1.18 mm, respectively; Portland cement (C) Type I general use (GU); Class-F fly ash (FA); polyvinyl alcohol (PVA) fibers; MgO powder; a polycarboxylic-ether type high-range water-reducing admixture (HRWRA) and water were used. The sieve analysis of the two different types of fine aggregates incorporated for ECC mixtures was conducted. Table 3.2 represents the grain size distributions and Table 3.3 represents the chemical composition and physical properties of Portland cement and class-F fly ash.

Table 3.2: Sieve Analysis of Silica Sand and Mortar Sand

U.S. Sieve #	Opening (mm)	% Passed Mortar Sand	% Passed Silica Sand
16	1.18	100	-
20	0.841	94	-
30	0.60	82.5	-
40	0.42	-	100
50	0.30	40	97.8
70	0.21	-	83.1
100	0.15	9.75	35.6
140	0.105	-	6.8
200	0.074	1.25	0.4
270	0.053	-	0

Table 3.3: Chemical Composition and Physical Properties of Portland cement and Class-F Fly Ash

Chemical Composition (%)	Cement	FA
Calcium Oxide, CaO	61.4	5.57
Silicon Dioxide, SiO ₂	19.6	59.5
Aluminium Oxide, Al ₂ O ₃	4.9	22.2
Ferric Oxide, Fe ₂ O ₃	3.1	3.9
Magnesium Oxide, MgO	3	-
Sulfur Trioxide, SO ₃	3.6	0.19
Alkalis as Na ₂ O	-	2.75
Loss on ignition, LOI	2.3	0.21
Sum (SiO ₂ + Al ₂ O ₃ + Fe ₂ O ₃)	27.6	85.6
Physical properties	Cement	FA
Residue 45 μ m (%)	3	9.6
Density (g/cm ³)	3.15	2.18
Blaine fineness (m ² /kg)	410	306
Air Content (%)	7.79	-

Polyvinyl Alcohol (PVA) fibers are considered to be the most common type of fiber used in ECCs. The PVA fibers with a diameter of 39 μ m, a length of 12 mm, strength of 1620 MPa and a modulus of elasticity of 42.8 GPa were used in this study. PVA fibers can elongate up to 6.0% of their original length and help in carrying the tensile stresses, strain-hardening performance and increasing the ECC matrix's toughness due to their fiber bridging behavior. Fiber bridging behavior of ECC is based on the micromechanics of the fiber/matrix interface where fibers transfer the stresses and maintain the low crack widths (Gahtrehsamani, 2015; Li et al., 2001; Suthiwarapirak et al., 2004). In addition, the surface of PVA fibers was coated with 1.2% mass of oil that reduces the interfacial bonding properties between the fiber and matrix and increases the tensile strain capacity of the mix; hence, fibers tend to rupture under tensile stresses (Li et al. 2003).

Lightly burnt MgO is a reactive material used in concrete for its ability to provide long-term expansive effect when concrete is hardened under restrained boundary conditions. The addition of MgO expansive agent enhances the mechanical properties of ECC mixtures through filling the small pores in the mixture and strengthening the bond with hydrated products. In addition to the formation of magnesium crystals Mg(OH)₂ (Du, 2005; Moradpour et al., 2013). In order to produce MgO, Magnesium Carbonate (MgCO₃) was sieved through the standard 45 μ m sieve. The portion passed (Particle size < 45 μ m) and retained (45 μ m < Particle size < 100 μ m) MgCO₃ were used. After that, the calcination process of MgCO₃ material was completed by adding and burning the

powder in three jars in an electric kiln at 900°C for 2 hours. After the holding time, MgO powder was spread on an aluminum sheet cool down and then were kept in sealed containers filled with silica gel and soda lime to prevent moisture and CO₂ from reacting again with MgO to form MgCO₃. Calcination process in kiln is shown in Figure 3.1.

Figure 3.1: Calcination Process of MgO at 900°C for 2 hours

3.3 Design of Link Slabs, Geometric Dimensions, Testing Schemes and Material Properties

The link slabs were designed based on the research studies conducted by Caner and Zia (1998) and Hossain et al. (2015). As shown in Figure 3.2, the moment distribution for a two span simply supported bridge structure leads to a negative moment distribution and a downward deflected shape developed at the link slab. This finding describes the flexural crack formations at the top face of the link slab due to the development of tensile stresses. The link slab section is designed between the inflection points which vary between 0 and 20% of the adjacent span's length (Kim et al. 2004; Caner & Zia, 1998).

The structural performance and self-healing evaluation of the five ECC mixtures were investigated using one-fourth scale model link slab specimens. The summary of the configuration of the link slabs subjected to monotonic or fatigue loading, fatigue loading scheme and curing schemes/conditions (to induce self-healing) are provided in Table 3.4 and Table 3.5.

Figure 3.2: Two Span Bridge Deformation with Focus on Link Slab Section (Kim et al. 2004)

Table 3.4: Link Slab Specimens Configuration Subjected to Monotonic Loading

Link Slab Designation	Length (mm) (c/c between the supports)	Debonding Zone Length 5% (mm)	Transition Zone Length 2.5% (mm)	Concrete in Transition Zone	Concrete in Debonding Zone
Control-link slab specimens tested under static monotonic loading at the age of 28 days					
LS1-ECC-F-1.2-SS	810	330	150	SCC	ECC-F-1.2-SS
LS2-ECC-F-2.2-SS	810	330	150	SCC	ECC-F-2.2-SS
LS3-ECC-F-1.2-MS	810	330	150	SCC	ECC-F-1.2-MS
LS4-ECC-F-2.2-MS	810	330	150	SCC	ECC-F-2.2-MS
LS5-ECC-F-1.2-SS-MgO	810	330	150	SCC	ECC-F-1.2-SS-MgO
LS11-SCC	810	330	150	SCC	SCC

*F: Class-F fly ash; SS: silica sand; MS: mortar sand; 1.2: 55% fly ash as cement replacement, 2.2: 70% fly ash as cement replacement; LS: link slab; SCC: self-consolidating concrete

Table 3.5: Link Slab Specimens Configuration Subjected to Fatigue Loading

Link Slab Designation	Length (mm) (c/c between the supports)	Debonding Zone Length 5% (mm)	Transition Zone Length 2.5% (mm)	Concrete in Transition Zone	Concrete in Debonding Zone	Number of Cycle	Mean Fatigue Stress Level (%)	Fatigue Stress Range (%)
Self-healing study Link slab specimens fatigued over three phases and then vapour cured before loaded to failure under static monotonic loading								
LS6-ECC-F-1.2-SS	810	330	150	SCC	ECC-F-1.2-SS	50000 (28d)	55%	±15
						100000 (28d+2m)		
						100000 (28d+2m+1m)		
LS7-ECC-F-2.2-SS	810	330	150	SCC	ECC-F-2.2-SS	50000 (28d)	55%	±15
						100000 (28d+2m)		
						100000 (28d+2m+1m)		
LS8-ECC-F-1.2-MS	810	330	150	SCC	ECC-F-1.2-MS	50000 (28d)	55%	±15
						100000 (28d+2m)		
						100000 (28d+2m+1m)		
LS9-ECC-F-2.2-MS	810	330	150	SCC	ECC-F-2.2-MS	50000 (28d)	55%	±15
						100000 (28d+2m)		
						100000 (28d+2m+1m)		
LS10-ECC-F-1.2-SS-MgO	810	330	150	SCC	ECC-F-1.2-SS-MgO	50000 (28d)	55%	±15
						100000 (28d+2m)		
						100000 (28d+2m+1m)		
LS12-SCC	810	330	150	SCC	SCC	50000 (28d)	55%	±5
						100000 (28d+2m)		
						100000 (28d+2m+1m)		

*F: Class-F fly ash; SS: silica sand; MS: mortar sand; 1.2: 55% fly ash as cement replacement, 2.2: 70% fly ash as cement replacement; LS: link slab; SCC: self-consolidating concrete

The scope of this research was focused on the composite link slab section made of steel girder and concrete slab connected through stud shear connectors. The link slab section was designed between the inflection points which are located at 6.5% of the adjacent span's length based on the overall stiffness of the section. As shown in Figures 3.3(a) and Figure 3.3(b), ECC and SCC link slabs for the 1/4th scale model were designed of two W100 x 19.3 steel structural sections with a total length of 810 mm. As per Caner and Zia (1998), the length of the debonding zone was selected as 330 mm (5% of simple spans). While the length of the transition zone with additional 50% shear stud connector (total of 16 studs) was 300 mm in accordance with Qian et al. (2009). The cross-sectional dimensions for the 1/4th scale link slab models as presented in Figure 3.3(c) had a width of 176 mm and a depth 60 mm. A minimum longitudinal steel reinforcing ratio of 1.1% (three 6 mm bars) were provided for maintaining a low structural stiffness of the link slab. Additional transverse reinforcements at 210 mm c/c spacing were provided using 6 mm bars as shown in Figure 3.3(c). The reinforcing steel bars had a mean yield and ultimate strengths of 407 MPa and 550 MPa, respectively.

For this research study, five ECC and a SCC were casted as control link slab specimens for monotonic static loading. Five ECC and one SCC link slab specimens from the same ECC/SCC mixtures were also casted for the flexural fatigue testing, self-healing evaluation in terms of strength-deflection recovery and residual strength-displacement response of healed fatigued specimens through post-fatigued monotonic loading to failure compared to their virgin counterparts. The supporting steel girder for the 1/4th scale link slab was selected as W100 x19 steel structural section from the Canadian Institute of Steel Construction (CISC, 2010). Figure 3.3(c) and Table 3.6 represent the geometric dimension and sectional properties of W100x19 section.

(a) ECC Link Slab

(b) SCC Link Slab

(c) Link Slab Cross-Section

Figure 3.3: Geometry and Reinforcement Detailing of Link Slab (Dimensions in mm)

Table 3.6: Properties of W100x19 Structural Steel Section

Section	Area (mm ²)	I_{x-x} (10 ⁶ mm ⁴)	S_{x-x} (10 ³ mm ³)	I_{y-y} (10 ⁶ mm ⁴)	S_{y-y} (10 ³ mm ³)	Depth (mm)	Flange Width (mm)	Flange Thickness (mm)	Web Thickness (mm)	Yield Stress f_y (MPa)
W100x19	2480	4.77	89.9	1.61	31.2	106	103	8.8	7.1	345

* I_{x-x} and I_{y-y} denote the moment of inertia around x-x axis and y-y axis, respectively; S_{x-x} and S_{y-y} denote the section modulus around x-x axis and y-y axis, respectively

The selected shear stud for the composite action of the steel girder and the concrete bridge decks was selected in accordance with CISC (2010). The yield (f_y) and ultimate (F_u) strength of the shear stud were 350 MPa and 410 MPa, respectively. The schematic representation of the shear stud is shown in Figure 3.4.

Figure 3.4: Schematic Representation of the Shear Stud

3.4 Specimens Preparation and Construction

This section of the chapter concludes preparation and production of the beam and link slab specimens produced for the experimental investigation.

3.4.1 Mixing Procedures

Hobart type mixer with 20-liter capacity was used for the production of the five different ECC mixture used for the experimental investigation. Before mixing, all solid dry materials of ECC

mixtures were weighed separately such as Portland cement, Class-F fly ash, silica sand or Mortar sand. After 30 s of dry mixing, up to 80% of water was added for 2 min of mixing, after that, HRWRA was added to the rest of the water for about 1 min of mixing, with a possibility of a slight adjustment in the amount of the HRWRA until observing a thick flowable soupy mix. The final step was performed by adding the PVA fiber slowly into the mixer preventing the formation of fiber clotting. Thus, the final ECC mixtures should better workability and fiber dispersion. The mixing procedure steps are presented in Figure 3.5.

(a) Mixing of Solid Ingredients

(b) Water & HRWRA Addition

(c) PVA Fiber Addition

(d) Final Consistent Flowable
ECC Mixture

Figure 3.5: Mixing Sequence of ECC Production

The SCC mixture was produced using the 120-liter drum mixer. Each bag of the SCC ready mix consists of a 30-kg dry content that requires a 2.4 liter of water to produce the SCC mixture. The ready dry mix package was mixed with 75% of the required amount of water for 2 minutes. The remaining amount of the water was added slowly in order to achieve flowable mix.

3.4.2 Fabrication and Casting of Specimens

For the mechanical properties of different ECC mixtures, at least two 355 mm×50 mm×76 mm prismatic beam moulds were prepared and casted for several types of four-point bending testing. 50 mm cubic specimens were prepared for the compressive strength test at 28 days. All specimens were moisture cured in plastic bags in the curing room which has a relative humidity (RH) 95% ± 5% and a room temperature of 23 ± 2 °C until the day of testing. The beam prismatic and cubic specimens for each ECC mixture were tested at 28, 56 and 90 days to determine the flexural strength and the compressive strength, respectively. Casting of ECC prismatic/beam specimens are shown in Figure 3.6.

The flexural fatigue testing on beam specimens was performed under load control conditions at the age of 28 days only for step fatigue and 400000 cycles. However, further flexural fatigue testing was carried on in order to investigate the self-healing behavior of cracked/damaged ECC beam specimens.

Figure 3.6: ECC Prismatic Beam Specimens

Wooden deck formworks as shown in Figure 3.7 were prepared through fastening long wooden boards together for casting 1/4th scale link slab specimens. After that, the two separate steel girders at 30 mm distanced apart were installed under the wooden mold. A 15-lb roofing paper

was installed in the debonding zone of 330 mm as a barrier between the steel and concrete. The casting process of the link slab similar to the general field practice is represented in Figure 3.8. the process started by casting the bridge deck parts with flowable SCC, and left for 24 hours for setting and then casting of the ECC or SCC link slab section. Control specimens in the form of cubes/cylinders and beams were also cast determine the compressive strength and flexural strength, respectively of SCC and ECC used in the link slab specimens at the age of testing. All ECC beams and link slabs were kept continuously in a sealed container to provide hot vapor curing. At the days of testing, specimens to be tested were taken out and then returned to the tank after conducting all the tests and readings required.

(a) Mold Preparation

(b) SCC Bridge Deck Casting

(c) Link Slab Casting

(d) Final Link Slab Specimen before Demoulding

Figure 3.7: Casting Process of 1/4th Scale Link Slab Model Specimens

3.5 Test Set-Ups, Instrumentations and Testing Procedures

3.5.1 Compressive Strength of ECC Cube Specimens

The compressive strength was evaluated for all ECC at 28, 56 and 90 days by using 50 mm cubic specimens (at least 3 samples for each of age). The compression test was carried out as per ASTM C39 (2016) using a compression testing machine with a capacity of 400,000 lbs as shown in Figure 3.8.

Figure 3.8: Compressive Strength Test Set Up

3.5.2 Flexural Strength of ECC Beam Specimens

Beam prismatic specimens for each ECC mixture were tested at 28, 56 and 90 days to determine the flexural strength (Modulus of rupture) in accordance to ASTM C78-b (2015). All ECC specimens were sanded to obtain a flat surface. At the three different ages, static monotonic loading was applied through the four-point bending test under displacement control condition at a loading rate of 0.005 mm/s on a closed-loop servo-controlled loading system. As shown in Figure 3.9, the four point bending test for measuring the flexural strength requires a span length of 304.8 mm with a center span length of 101.6 mm. Finally, the load and mid-span deflection throughout the flexural test were recorded on a computerized data acquisition system.

Figure 3.9: Four-Point Bending Test Setup

3.5.3 Fatigue Flexure Performance of ECC Beam Specimens and Self-healing Evaluation

The flexural fatigue loading was applied on ECC specimens in three different approaches. The first approach (fatigue flexural performance applying 400000 cycles at constant stress level) was divided into three various stages. The first stage was presented by the pre-fatigue behavior which was determined by applying monotonic loading to failure at the age of 28 days (this stage specified the control specimens of each mixture). The second stage was the behavior under fatigue flexural loading for 400000 cycles and 4 Hz cyclic loading at a stress level of 50% ($\pm 15\%$) of ultimate loading at the age of 28 days. The final stage was the post-fatigue behavior which was analyzed through applying monotonic loading to failure of fatigued specimens at the age of 28 days.

The second approach (fatigue flexural performance under step fatigue loading at different stress levels) was divided into two main stages. The first stage is similar to that of first approach. The second stage determined at four different stress levels (25%, 50%, 75% and 100% of the ultimate loading of the controls) and 4 Hz cyclic loading. In every stress level. It started by applying 50000 cycles at the first stress level of 25% ($\pm 15\%$). At the second 50% ($\pm 15\%$) stress level, same specimen was loaded for another 50000 cycles and hence, reaching 100000 in total. After that, the third stress level of 75% ($\pm 15\%$) of ultimate loading was applied on the same specimen by applying a third 50000 cycles. Eventually, 100% of the ultimate loading was applied on the same specimen to reach up to 200000 cycles in total. It should be noted that all ECC specimens failed after 75% (between 150000 and 200000 cycles) and none of them completed to the 100% stress level.

The third approach (self-healing characterization under fatigue loading at constant stress level) was conducted in five main stages. The first stage was similar to that of first approach which was required in order to evaluate pre-fatigue behavior of the control specimens of each mixture. The second stage was conducted by loading the specimens for 50000 cycles and 4 Hz cyclic loading at a stress level of 50% of ultimate loading at the age of 28 days and then the specimens were left for vapor curing for 2 months in order to evaluate the self-healing property of ECC. The third stage was conducted by loading the specimens up to 100000 cycles and 4 Hz cyclic loading at a stress level of 50% of ultimate loading and left again for vapor curing for another 1 month. The fourth stage was conducted by loading the specimens up to 100000 cycles and 4 Hz cyclic loading at a stress level of 50% of ultimate loading. The final stage is the post-fatigue behavior of fatigue damaged healed specimens which was analyzed through applying monotonic loading to failure at the age of 118 days.

The four-point bending test was conducted under both static and fatigue loading. Two specimens were prepared for static loading tests as control specimens which were performed under displacement control conditions, while the fatigue loading tests were carried out under load control conditions. According to the second approach discussed previously, two samples were loaded for 50000 cycles at 28 days and then loaded for 100000 cycles at 28 days + 2 months. After that, one of the two sample was loaded to failure under monotonic loading at 28 days + 2 months + 1 month, while the other one was loaded for an extra 100000 cycles at 28 days + 2 months + 1 month followed by monotonic loading to failure. In order to investigate the self-healing behavior of fatigued samples compared to their virgin counterparts, two virgin samples were tested at the age of 118 days.

Static monotonic loading tests were applied through the four-point bending test under displacement control condition at a loading rate of 0.005 mm/s. The average of the flexural strength results of ECC control specimens was used to determine the fatigue stress levels for the different fatigue loading tests. To avoid any slip or collapse of specimens during fatigue testing, static loading rate of 0.50kN/min was used for the load to reach the maximum stress level. During the cyclic fatigue tests, the mid-span deflection evolutions were recorded. At the end, flexural monotonic testing was applied on the exhausted fatigued specimens for determining the residual strength and bending capacity.

3.5.4 Monotonic Loading of One Fourth Scale Link Slab Specimens – Testing and Instrumentation

The link slabs were tested in an inverted orientation for simulating actual link slab behaviour in a joint-free bridge deck (Figure 3.10). Five ECC and one SCC link slab specimens were tested to failure under monotonic loading at the age of 28 days. As shown in Figure 3.11a, the span length between the supports was 810 mm. Strain gauges were installed at the mid-span on the rebar, concrete tensile and compressive surfaces for recording and monitoring strains during the application of monotonic loading up to failure. Figure 3.10(a) also shows the location of the inclinometer near the support for recording the support-end rotation.

Static testing procedure was performed for all link slab specimens under displacement control condition at a loading rate of 0.005 mm/s on a closed-loop servo-controlled loading system. The load and mid-span deflection throughout the flexural test were recorded on a computerized data acquisition system (MTS machine). Another data acquisition system was used for recording the strain developments, rotation and LVDTs deformation at the supports. The crack characterization including the number of cracks, crack widths and lengths were visually observed and measured using the crack-scope.

Figure 3.10 (a): Test Set-Ups for 1/4th Scale Link Slab Specimen (Dimensions in mm)

Figure 3.10 (b): Testing and Instrumentation of 1/4th Scale Link Slab Specimen

3.5.5 Fatigue Flexure Performance of One Fourth Scale Link Slab Specimens – Testing, Instrumentation, Curing Process and Self-healing Evaluation Scheme

Flexural fatigue tests were performed to investigate the fatigue and self-healing behavior of the 1/4th scale ECC link slab compared with the control SCC link slabs. Two link slab specimens of each of the five ECC and one SCC mixtures were prepared. At the age of 28 days, one specimen of each mixture was tested by applying monotonic loading to failure in order to evaluate pre-fatigue behavior of the control specimens in terms of the ultimate strength, bending capacity and

strain development. After that, the second specimen of each mixture was tested under flexural fatigue loading at three different stages. The first stage was conducted by loading the specimens for 50000 cycles and 4 Hz cyclic loading at a stress level of 55% of ultimate loading at the age of 28 days and then the specimens were left for vapor curing for 2 months in order to evaluate the effect of self-healing of ECC on structural performance. The second stage was conducted by loading the specimens up to 100000 cycles and 4 Hz cyclic loading at a stress level of 55% of ultimate loading and left again for vapor curing for another 1 month. The third stage was conducted by loading the specimens up to 100000 cycles and 4 Hz cyclic loading at a stress level of 55% of ultimate loading. Finally, the post-fatigue behavior was analyzed through applying monotonic loading of specimens to failure at the age of 118 days to determine the residual strength, bending capacity, stiffness and energy absorbing capacity compared to their SCC or ECC counterparts.

The testing procedure and instrumentation used for the pre-fatigue and post-fatigue loading tests were the same as the procedure adopted in Section 3.5.4. While, flexural fatigue testing was carried out under load control condition, at a rate of 0.5 kN/min as a ramp until the load reached the mean fatigue stress level of 55%. Then, the fatigue cycles started with an upper and lower limit of $\pm 15\%$ of the applied mean stress level until the end of the fatigue cycles (50000 cycles at 28d, 100000 cycles at 28d+2m, 100000 cycles at 28d+2m+1m).

Throughout the fatigue flexural tests, the load and mid-span deflection were recorded on a computerized data acquisition system (MTS machine). Another data acquisition system was used for recording the strain developments, rotation and LVDTs deformation at the supports. The crack characterization including the number of cracks, crack widths and lengths were visually observed and measured using the crack-scope.

In summary, after determining the ultimate strength of control link slab specimens at the age of 28 days for the five ECC and one SCC mixtures under monotonic loading (pre-fatigue behavior using six specimens), one link slab specimen for each of the mixtures (six specimens) was fatigued for 50000 cycles. The cracks formed on the tensile surface in the debonding zone were exposed to vapour curing for 2 consecutive months in order to stimulate the self-healing property of ECC. After that, the same specimens were loaded for 100000 cycles followed by another month of vapour curing for self-healing (as described earlier).

Figure 3.11 shows the typical process of vapour curing stimulating self-healing of the fatigued link slab specimens. The link slab specimens were placed in the container and the strain gauges were covered by small plastic bags. The humidifier was placed in one of the corners after being filled with water. Finally, the container was sealed and duct-taped by polyethene sheets. After 3 months of vapour curing during a total 250000 cycles of fatigue loading, all fatigued-self-healed link slab specimens were loaded under static monotonic loading to failure. The performance of pre-fatigued (control) and post-healed fatigues link slab specimens was compared based on load-deflection/moment-rotation response, initial stiffness, energy absorption capacity, ductility, concrete/steel strain development and crack characterization was evaluated to determine the self-healing capability of different types of ECCs used.

Figure 3.11: Vapor Curing Steps; A) Placing the Link Slab Specimens and Covering the Strain Gauges; B) Placing the Humidifier in the Container; C) Sealing the Container from the Inside and Outside; D) Vapor Curing of Specimens for 3 Months

3.6 Summary

Chapter 3 covers material properties, ECC and SCC mix proportions, mixing procedures, detailed design and dimensioning of link slabs, specimen preparation and casting along with testing procedures under monotonic and fatigue flexure loading.

This chapter also presented the experimental programs which included mechanical properties (compressive strength, flexural strength, fatigue flexural loading) and structural performance investigations on 1/4th scale link slab models made with ECC and SCC when subjected to monotonic and flexural fatigue loading in order to evaluate the self-healing behaviour.

CHAPTER FOUR

MECHANICAL PROPERTIES, FATIGUE FLEXURE PERFORMANCE AND EVALUATION OF SELF-HEALING BEHAVIOUR OF ECC BEAMS

4.1 Introduction

Chapter 4 presents the results of the experimental investigation on ECC mixtures made of mortar sand, silica sand, fly ash class F and MgO expansive agent. The mechanical properties such as compressive and flexural strengths characteristics at different ages are analyzed in order to determine the ECC mixture with the best performance. The influences of different percentages of fly ash (55% and 70%), types/size sand and MgO expansive agent are analyzed. This chapter also includes the investigation of the pre-fatigue, fatigue and post-fatigue flexural performance of beams made of all five ECC mixtures based on flexural strength, bending capacity, stiffness and energy absorbing capacity. The main variables in the fatigue testing were the application of different number of cycles and fatigue stress levels. The self-healing behaviour of ECC beams under repeated fatigue loading are also described based on investigations conducted under two stage repeated curing between the three stage application of total 250000 fatigue cycles over total self-healing duration of 118 days before fatigued/self-healed beams were loaded failure under monotonic loading. The self-healing performance was analyzed based on crack-healing, recovery of strength, deflection and energy capacities.

4.2 Compressive Strength Properties of ECC Mixtures

Compressive strength test was performed by testing least 3 control cubic specimens for each ECC mixture at 28, 56, and 90 days in order to analyze the compressive performance over different ages. The compressive strength results of five ECC mixtures at three different ages are summarized in Table 4.1 and trend of variation with age is shown in Figure 4.1.

Table 4.1: Compressive Strength of ECC Mixtures at Ages of 28, 56, 90 Days

Mix No.	Mix Designation	Compressive Strength (MPa)		
		28 days	56 days	90 days
Control beam specimens – tested to failure under static monotonic loading after n days of lab curing				
1	ECC_F_1.2_SS	56±1.79	57±0.25	60±0.85
2	ECC_F_2.2_SS	43±0.84	46±2.19	50±1.46
3	ECC_F_1.2_MS	59±1.55	62±1.31	67±0.73
4	ECC_F_2.2_MS	48±1.06	51±1.67	58±0.98
5	ECC_F_1.2_SS_MgO	52±1.05	*	*

*Not tested

As per Table 4.1, the increase of Class F-fly ash content 1.2 to 2.2 without changing the type and size of aggregates reduces the compressive strength. This is attributed to the slower reaction rate of FA compared to the hydration of cement lower content of CaO (Sahmaran et al. 2008; Mindess et al. 2003). On the other hand, the incorporation of larger aggregate size in ECC, such as using mortar sand (1.18 mm) instead of micro-silica sand (0.30 mm), without changing the cement replacement rate improved the compressive strength. The reason behind this finding is the available volume between the aggregate that allows more paste content between the aggregate and thus leads to an increase of the water content in the interfacial transition zone (ITZ) (Mehta & Monteiro, 2006). The low compressive strength of ECC-MgO at the age of 28 days might be caused by the replacement of cementitious materials by MgO expansive agent. The low compressive strength of MgO displays a slow hydration rate at this age compared to the high hydration rate of the cementitious materials.

Figure 4.1: Compressive Strength of ECC Mixtures over Ages of 28, 56 and 90 Days

4.3 Flexural Strength of ECC Mixtures

The 28-day and 56-day flexural stress-mid-span deflection response of beam specimens made of ECC mixtures compared in Figure 4.2 (a-b) and the results of flexural strength and mid-span beam deflections at the age of 28, 56 and 90 days are summarized in Table 4.2. The flexural strength of the ECC mixtures in this study ranged between 8.73 and 12.16 MPa at 28 days, ranged between 8.80 and 11.24 at 56 days and ranged between 8.92 and 10.53 MPa at 90 days. In general, based on the mix design parameters, ECC can exhibit a range of flexural strength that varies from 5 to 16 MPa (Wang and Li 2007). The 28-day bending capacity based on mid-span deflection at peak stress ranged between 1.58 mm and 7.04 mm. All ECC mixtures showed tensile strain hardening behaviour with ECC_F_2.2_SS showing the highest. Generally, the tensile strain hardening behaviour of ECC is achieved by the multiple non-localized cracks that sustain and pass the load without any catastrophic damage in the structure regardless the length of the crack and type of the loading (Li, 1997, 2003).

From Table 4.2, Figure 4.2 (a-b) and Figure 4.3(a), it is noted that the increase of Class F-fly ash cement replacement from 1.2 to 2.2 without changing the type and size of aggregates resulted in a reduction in the flexural strength behaviour, while the mid-span deflection and the bending capacity were notably enhanced. Therefore, both ECC_F_1.2 mixtures produced with either mortar or silica sand developed maximum flexural strengths at all ages when compared with ECC mixtures produced with cement replacement of 2.2. The reduction in flexural strength might be attributed to the reduction of the PVA fiber/matrix chemical bonding as well as the matrix toughness. In addition, Figure 4.3(b) represents the enhancement in bending capacity of ECC mixtures when produced with more fly ash (FA/C = 2.2). This finding might be related to the reduction in chemical bond capacity as well as the improvement in frictional bond between the PVA fibers and matrix. This might also be attributed to the presence of the extra un-hydrated fly ash particles that might function as filler materials (Sherir et al., 2014; Yang et al. 2007; Wang and Li, 2007; Sahmaran et al., 2009).

Table 4.2: Flexural Strength and Ultimate Mid-Span Deflection of ECC Mixtures at 28, 56 and 90 Days

Mix No.	Mix Designation	Flexural Strength and Peak Flexural Deflection					
		28 days		56 days		90 days	
		Strength (MPa)	Deflection (mm)	Strength (MPa)	Deflection (mm)	Strength (MPa)	Deflection (mm)
Control beam specimens – tested to failure under static monotonic loading after n days of lab curing							
1	ECC_F_1.2_SS	11.18 ± 1.04	2.41	10.18 ± 0.25	2.12	10.06 ± 2.36	1.69
2	ECC_F_2.2_SS	8.73 ± 1.66	7.04	8.90 ± 1.47	2.91	8.92 ± 3.27	1.88
3	ECC_F_1.2_MS	12.16 ± 0.34	1.58	11.24 ± 0.71	1.46	10.53 ± 1.94	1.43
4	ECC_F_2.2_MS	9.41 ± 1.35	2.76	9.76 ± 0.19	2.82	10.29 ± 2.55	1.55
5	ECC_F_1.2_SS_MgO	10.82± 0.87	2.44	*	*	*	*

*Not tested

Figure 4.2 (a): Flexural Strength-Mid Span Deflection Curves of ECC Mixtures at 28 Days

Figure 4.2(b): Flexural Strength-Mid Span Deflection Curves of ECC Mixtures at 56 Days

(a) Flexural Strength

(b) Peak Flexural Deflection

Figure 4.3: Evolution of Flexural strength and Maximum Flexural Deflection at Failure of ECC Mixtures with Age

Based on Figure 4.3 (a-b), deflection/bending capacity of ECC beams decreased with age while flexural strength showed increasing or decreasing trend. According to Figures 4.3(a-b), Table 4.2 and Figure 4.2, the incorporation of larger aggregate size in ECC mixtures, such as using mortar sand (1.18 mm) instead of micro-silica sand (0.30 mm), without changing the cement replacement rate improved the ultimate loading capacity while reduced the mid-span deflection. Therefore, both ECC_F_MS mixtures produced with either 1.2 or 2.2 FA/C ratios developed maximum flexural strengths and lowered mid-span deflection at all ages when compared with ECC mixtures produced

with silica sand. This might be related to the enhancement of aggregate interlocking and matrix toughness with the use of larger aggregate size. Moreover, the reduction of the fiber-matrix frictional interface bond and the uniform dispersion of fibers due to balling effect when large size of the aggregates used did not aid the fibers to be coated with the matrix (Sahmaran et al. 2009, Li et al., 1995, Sherir et al., 2014).

ECC-MgO system was prepared through the calcination system of exposing Magnesite (Magnesium Carbonate, MgCO_3) to 900°C for 4 hours by using $45\text{ }\mu\text{m}$ particle size of MgCO_3 . As shown in Figure 4.1 and 4.3 (a-b), the addition of MgO to ECC mixture shows similar behaviour in terms of flexural strength and bending capacity when FA/C ratio (1.2) and aggregate type and size (silica sand) were fixed. Therefore, flexural strength and bending capacity (based on peak deflection) of ECC_F_1.2_SS_MgO were 97% (10.82/11.18) and 101% (2.44/2.41), respectively of their counterpart ECC_F_1.2_SS. The concept of adding MgO to ECC mixtures was to enhance the healing capability of ECC mixtures when cracking occurs and this aspect was studied thoroughly by Sherir (2016) and Sherir et al. (2016, 2017). However, the addition of MgO in ECC_F_1.2_SS_MgO to the control specimens of ECC (without the presence of cracking or damage) did not help the concept of MgO expansion and self-healing mechanism due to two reasons. Firstly, the non-existence of cracks might not provide enough room for MgO hydration within the cement microstructure to accommodate forming new hydration products to supplement both pozzolanic reaction and cement hydration. Secondly, the reaction of MgO is very slow and could be activated at later ages (within the period of 7-1000 days) (Yang et al., 2007; Sherir, 2016).

4.3.1 Crack Characterization in ECC Beams

The tensile strain hardening behaviour of ECC is achieved through forming multiple non-localized cracks that sustain and pass the load without any catastrophic damage in the structure regardless of the length of crack and type of loading (Li, 1998, 2003). All ECC specimens developed cracks at the center of beam at the tension face under monotonic flexural loading. The load and deflection after the first cracking continued to increase in order to initiate multiple cracking. The increase of the inelastic deformation was accompanied by tendency to increase the crack width (which was resisted by fiber bridging effect) initiating new micro-cracks. Figure 4.4 shows a typical cracking pattern of ECC specimens. Once the tensile load exceeds the maximum fiber-bridging capacity, no new additional cracks will be formed and localized cracks will result. This will result in bending failure in ECC specimens. At unloading of the failed specimens, it was observed that the crack

width was decreased typically by 30%. The width and length of all cracks were measured on the tension surface using a crack microscope. Table 4.3 represents the crack characterization for all ECC specimens at different ages including average number of cracks, average crack widths and average crack lengths.

Table 4.3: Crack Characterization of Beams Made of ECC Mixtures at 28, 56 and 90 Days

Beam No.	Mix Designation	Crack Characterization: Number of crack and crack width					
		28 days		56 days		90 days	
		Number	Width (μm)	Number	Width (μm)	Number	Width (μm)
Control beam specimens – tested to failure under static monotonic loading after n days of lab curing							
1	ECC_F_1.2_SS	23	30-80	29	<80	28	<80
2	ECC_F_2.2_SS	55	20-70	40	<80	44	<80
3	ECC_F_1.2_MS	18	30-60	22	<80	20	<80
4	ECC_F_2.2_MS	46	20-60	30	<80	31	<80
5	ECC_F_1.2_SS_MgO	20	30-70	-	-	-	-

Figure 4.4: Typical Cracking Pattern of ECC Beam Specimen

Figure 4.5: Number of Cracks Formed at 28, 56 and 90 Days

It can be noted from Table 4.3 and Figure 4.5 that the number of cracks was directly proportional to the bending capacity and inversely proportional to the flexural strength as can be interpreted from Figure 4.3 a-b. Specifically, ECC_F_2.2_SS revealed the maximum number of cracks (55 cracks, the lowest flexural strength and the maximum deflection capacity between all studied ECC mixtures. The increase in the FA/C ratio from 1.2 to 2.2, without changing the type and size of aggregates, in the ECC mixtures tends to increase the number of cracks at all ages (Table 4.3) due to the presence of un-hydrated fly ash particles that might act as fillers. This might also be attributed to the enhancement of frictional bond of Interfacial Transition Zone (ITZ) between PVA fiber and ECC matrix.

However, the incorporation of larger aggregate size (maximum aggregate size 'MAS' 1.18 mm) increased the path of crack propagation around mortar sand leading to increase the fracture energy in MS-ECC matrix compared to SS-ECC counterpart (MAS = 0.3 mm of silica sand). The increase in fracture energy tends to delay the crack formation by increasing the crack length instead of providing more number of cracks (Sherir et al., 2014). Even though MS-ECC matrix might produce more fracture energy than SS-ECC matrix due to the presence of larger aggregate size, still MS-ECC matrix developed steady state crack propagation mode within the permissible limit of the second energy criterion.

The addition of MgO expansive agent to ECC helped in developing less number of cracks compared to their ECC counterpart. The crack width is considered a decisive key factor for many reinforced concrete applications which can reflect and develop the performance of the structure under service load, durability and further properties in the structure. ECC has the ability to maintain the micro crack width to less than 60~80 μ m even under ultimate loading as is the case for current study (Table 4.3); this intrinsic material property can stimulate crack self-healing behaviour which improves the mechanical properties, durability and serviceability of structural elements (Herbert and Li, 2013, Sahmaran et al., 2008, Sahmaran et al., 2009).

4.3.2 Initial Stiffness

The stiffness values of all ECC specimens calculated from flexural stress-mid-span deflection curves is presented in Table 4.4 along with peak load and deflection as well as 1st cracking load and deflection.

Table 4.4: Initial Stiffness of ECC Mixtures at 28, 56 and 90 Days

Mix No.	Mix Designation	Flexural Load and Deflection				
		Load (kN)		Mid-Span Deflection (mm)		Initial Stiffness (kN/mm)
		Peak	1st Crack	Peak	1st crack	
Control beam specimens – tested to failure under static monotonic loading after 28 days of lab curing						
1	ECC_F_1.2_SS	10.32	4.87	2.41	0.43	11.33
2	ECC_F_2.2_SS	7.80	3.02	7.04	0.50	6.04
3	ECC_F_1.2_MS	10.69	3.75	1.58	0.32	11.72
4	ECC_F_2.2_MS	9.80	4.62	2.76	0.59	7.83
5	ECC_F_1.2_SS_MgO	10.00	3.02	2.44	0.75	4.03
Control beam specimens – tested to failure under static monotonic loading after 56 days of lab curing						
1	ECC_F_1.2_SS	9.39	6.19	2.12	0.52	11.90
2	ECC_F_2.2_SS	8.21	5.66	2.91	0.53	10.68
3	ECC_F_1.2_MS	10.37	6.06	1.46	0.41	14.78
4	ECC_F_2.2_MS	9.01	5.70	2.82	0.42	13.57
Control beam specimens – tested to failure under static monotonic loading after 90 days of lab curing						
1	ECC_F_1.2_SS	9.28	6.41	1.69	0.49	13.08
2	ECC_F_2.2_SS	8.23	3.15	1.88	0.35	9.00
3	ECC_F_1.2_MS	9.72	4.44	1.43	0.29	15.31
4	ECC_F_2.2_MS	9.49	2.72	1.55	0.20	13.60

Figure 4.6: Initial Stiffness of ECC Mixtures at 28, 56 and 90 Days

According to Table 4.4 and Figure 4.6, initial stiffness increased with the increase of age and ECC_F_1.2_SS_MgO showed lowest stiffness at 28 days which may be attributed to the slow rate of development of matrix toughness due to slow rate of hydration. The increase in the FA content from 1.2 to 2.2 lowered the initial stiffness of ECC specimens at all ages due to the reduction in matrix toughness. While, the incorporation of larger aggregate size resulted in higher stiffness of

ECC specimens possibly due to the enhancement of aggregate interlocking, matrix toughness and friction. It can be concluded from combined information derived from Figures 4.2 ad 4.6 that low stiffness was coupled with high ductility, strain hardening and multi-cracking since higher stiffness promotes the mixture to develop higher stresses at faster rates. Moreover, the stiffness of the cement paste phase and ITZ phase increased with the age due to further curing and hydration associated with the age. This can be indicated through the increase in first cracking load as percentage of the ultimate load at the age of 56 days compared to its counterpart at the age of 28 days (Table 4.4). However, less room availability for further hydration led to a slight reduction in the stiffness at the age of 90 days.

4.4 Fatigue Flexure Performance and Self-healing under Fatigue Loading

This study involved three approaches described in Chapter Three. The flexural strength characteristics of the control ECC specimens were determined through flexural monotonic testing at the age of 28 days and the results were used to determine the load at various stress levels for the fatigue flexural loading in three approaches.

4.4.1 First Approach – Fatigue Flexural Performance Applying 400000 Cycles

Beam specimens made from all five ECC mixtures were subjected to fatigue loading for 400000 cycles at 4 Hz and 50% ($\pm 15\%$) stress level at the age of 28 days. At the end of the fatigue flexural testing, fatigued specimens were failed under monotonic loading in order to determine post-fatigue residual strength and deflection capacity. The mid-span deflection after every cycle was recorded on a computerized data acquisition system during the fatigue loading. The mid-span deflection evolution curves shown in Figure 4.7 and Table 4.5 illustrate that the development of large deformation in the ECC specimens at the initial fatigue cycles. This was caused by the stress jump from the mean stress range to high fatigue stress ranges ($\pm 15\%$). Figure 4.7 show the development of higher mid span deflection for ECC beams with mortar sand (MS) compared to their silica sand counterparts and lower deflection evolution of ECC-MgO-SS compared to its ECC-SS counterpart.

Figure 4.7: Mid-span Deflection Responses of ECC Beams Subjected to $(50\% \pm 15\%)$ Fatigue Stress Level

The fatigue flexural tests were conducted for 4000000 cycles at 4 Hz cyclic loading rate which means that the duration of the test was 1664 min. The difference between initial and final value of mid-span deflection was calculated as well as the evolution speed rate (presented in Table 4.5) that demonstrates the relationship between speed rate and the aggregate type. The results shown in Table 4.5 are consistent with the mid-span deflection evolution curves for all ECC mixtures. The high deflection evolution speed rate associated with the FA-ECC mixtures with mortar sand reflected the tendency of generating more deflection under fatigue loading reflecting more ductility behavior and higher energy absorbing capacity as can be seen from the higher fatigue band width energy (Table 4.5). ECC-MgO specimens showed slightly higher deflection evolution speed rate and energy absorption compared to their ECC counterpart (Table 4.5).

Table 4.5: Mid-Span Deflection Evolution at (50% $\pm$ 15%) Fatigue Stress Level

Mix Designation	Flexural Strength (kN)	Mean Fatigue Stress Level (%)	Fatigue Stress Range (%)	Initial Deflection (mm)	Final Deflection (mm)	Mid-Span Evolution Speed Rate ($\mu\text{m}/1666.64 \text{ min}$)	Band Width Absorption Energy (Joule)
400000 Cycles							
ECC F 1.2 SS	10.32	50%	$\pm 15\%$	0.91	1.61	0.42	1.08
ECC F 2.2 SS	7.80	50%	$\pm 15\%$	0.99	1.77	0.47	0.91
ECC F 1.2 MS	10.69	50%	$\pm 15\%$	0.69	1.87	0.71	1.90
ECC F 2.2 MS	9.80	50%	$\pm 15\%$	0.65	1.79	0.68	1.68
ECC F 1.2 SS MgO	10.00	50%	$\pm 15\%$	0.71	1.45	0.44	1.11

Figure 4.8 compares the flexural performance of ECC beams - monotonically loaded to failure without fatigue loading (control specimens) and after subjected to fatigue loading (fatigued beams) up to 400000 cycles. The performance was judged based flexural strength and mid-span deflection at ultimate/peak load of fatigued ECC specimens as percentage of their control counterparts without fatigue as summarized in Table 4.6.

Table 4.6: Summary of Pre-Fatigue and Post-Fatigue Flexural Performance at the Age of 28 Days

Mix No.	Mix Designation	Flexural Strength (MPa)		Mid-Span Deflection at peak (mm)	
		Strength	Residual Strength (% of control)	Deflection	Residual Deflection (% of control)
Control beam specimens – tested to failure under static monotonic loading after 28-days of lab curing					
1	ECC F 1.2 SS	11.18	-	3.04	-
2	ECC F 2.2 SS	8.73	-	7.61	-
3	ECC F 1.2 MS	12.16	-	2.38	-
4	ECC F 2.2 MS	9.41	-	3.91	-
5	ECC F 1.2 SS MgO	10.82	-	3.70	-
Beam specimens – fatigued (400000 cycles) before loaded to failure under static monotonic loading					
1	ECC F 1.2 SS	5.18	46.33	1.34	44.08
2	ECC F 2.2 SS	3.57	40.89	1.01	13.27
3	ECC F 1.2 MS	7.67	63.08	1.43	60.08
4	ECC F 2.2 MS	5.21	55.37	1.09	27.88
5	ECC F 1.2 SS MgO	6.50	60.07	1.36	36.76

Figure 4.8: Flexural Strength-Mid Span Deflection Response - Post-Fatigue after 400000 Cycles and Pre-Fatigue

It can be observed from Figure 4.8 that applications of 400000 cycles of fatigue loading deteriorated the strength and deflection capacity of all ECC beam specimens as evident from the smaller flexural strength-deflection envelope of fatigued sample compare to their non-fatigued counterparts. According to Table 4.6, residual flexural strength and deflection capacity of fatigued specimens ranged from 40.89% to 63.08% and from 13.27% to 60.08%, respectively of their control non-fatigued counterpart. ECC_F_1.2_MS specimens showed highest residual strength and deflection capacities while ECC_F_1.2_SS_MgO specimens showed higher residual strength than their ECC_F_1.2_SS counterparts. The addition of MgO expansive agent improved the residual strength due to the high energy absorption capacity developed during the application of fatigue cycles. The improved residual energy can be resulted from the new room formed within the cracks and therefore, allowing MgO to relax both tensile and compressive stresses which intern reduced cracking. Fatigued specimens from all five ECC mixtures were not able to produce as many micro-cracks as observed in their control counterparts upon monotonic loading to failure as shown in Figure 4.9. It can be attributed to exhausting of the fiber bridging capacity through fiber pull-out or rupture. It can be observed from Table 4.6 that the increase in the FA content from 1.2 to 2.2 lowers the performance (induced more damage due to fatigue) of ECC specimens in terms of both residual flexural energy and deflection capacity – ECC_1.2 showed superior performance than ECC_2.2. This might be related to the presence of unhydrated particles acting as filler materials which increases the interfacial frictional bond. Thus, enhancing both strain hardening and multiple-cracking behaviors and lowering the residual energy. The incorporation of larger

aggregate size resulted in higher stress recovery which can be related to developing a steady state crack propagation mode. Cracks would be initiated around the aggregates through weaker Interfacial Transition Zone (ITZ) resulting in higher fracture energy required for the longer path of crack propagation.

(a) Fatigued Specimen

(b) Control Specimen

Figure 4.9: Cracking Patterns of Fatigued and Control Specimens

4.4.2 Second Approach - Fatigue Flexural Performance under Step Fatigue Loading at Different Stress Levels

The fatigue flexural loading on 28-day ECC beam specimens in this approach were applied consecutively at different higher stress levels at 4Hz. It started by applying 50000 cycles at a stress level of $25\% \pm 15\%$ of ultimate load followed by another 50000 cycled at $50\% \pm 15\%$ stress level, then another 50000 cycles at $75\% \pm 15\%$ stress level and finally at the stress level of 100% when all specimens were failed.

The load-mid span deflection responses of beams made of five ECC mixtures under step fatigue loading applied at increasing stress levels are shown in Figure 4.10. It can be observed that all ECC beams failed when reached at a fatigue stress level of $100\% \pm 15\%$. This finding can be noted from the accelerated sudden increase in the mid-span deflection. The mid-span deflection evolution speed rate was calculated by dividing the difference between the initial and final deflection by the time required to finish 50000 cycles which was found to be 208.33 minutes. Furthermore, the band width absorption energy was obtained by calculating the load-deflection area of the 50000 cycles of load-deflection evolution curves shown in Figure 4.10. The mid-span deflection evolution speed rate and the band width absorption energy for ECC beam specimens are summarized in Table 4.7.

Figure 4.10: Load-Deflection Curves of ECC Beam Specimens at Different Fatigue Stress Levels

The evolution of the mid-span deflection for all ECC beams with the increase of fatigue stress levels is plotted in Figure 4.11. The results showed that the increase of the fatigue stress level led to developing more deflection evolution, less stiffness and larger load-deflection band width (energy absorption) as can be seen from Figure 4.10. From Figs. 4.10 and 4.11, it can be inferred that mortar sand ECCs developed larger band width or deflection capacity compared to their sand counterparts while low fly ash content (FA/C = 1.2) ECCs showed higher deflection capacity compared to their sand counterparts. ECC-MgO developed higher deflection capacity than its ECC counterpart.

Figure 4.11: Mid-span Deflection Responses of ECC beams subjected to Different Fatigue Stress Levels

Table 4.7: Mid-span Evolution and Energy Absorption of ECC Beam Specimens at Different Fatigue Stress Levels

Mix Designation	Flexural Strength (kN)	Mean Fatigue Stress Level (%)	Fatigue Stress Range (%)	Initial Deflection (mm)	Final Deflection (mm)	Mid-Span Deflection Evolution Speed Rate ($\mu\text{m}/\text{min}$)	Band Width Absorption Energy (Joule)
50000 Cycles							
ECC F 1.2 SS	10.32	25%	$\pm 15\%$	0.25	0.28	0.18	0.03
ECC F 2.2 SS	7.80	25%	$\pm 15\%$	0.65	0.73	0.36	0.04
ECC F 1.2 MS	10.69	25%	$\pm 15\%$	0.37	0.44	0.33	0.06
ECC F 2.2 MS	9.80	25%	$\pm 15\%$	0.70	0.79	0.45	0.07
ECC F 1.2 SS MgO	10.00	25%	$\pm 15\%$	0.25	0.37	0.55	0.09
50000 Cycles							
ECC F 1.2 SS	10.32	50%	$\pm 15\%$	0.39	0.51	0.56	0.19
ECC F 2.2 SS	7.80	50%	$\pm 15\%$	1.01	1.26	1.17	0.14
ECC F 1.2 MS	10.69	50%	$\pm 15\%$	0.68	1.07	1.86	0.31
ECC F 2.2 MS	9.80	50%	$\pm 15\%$	1.03	1.25	1.02	0.16
ECC F 1.2 SS MgO	10.00	50%	$\pm 15\%$	0.73	0.93	0.96	0.15
50000 Cycles							
ECC F 1.2 SS	10.32	75%	$\pm 15\%$	0.68	1.88	5.76	0.93
ECC F 2.2 SS	7.80	75%	$\pm 15\%$	1.50	2.47	4.65	0.57
ECC F 1.2 MS	10.69	75%	$\pm 15\%$	1.34	3.91	12.35	2.06
ECC F 2.2 MS	9.80	75%	$\pm 15\%$	1.98	2.97	4.77	0.73
ECC F 1.2 SS MgO	10.00	75%	$\pm 15\%$	1.84	2.44	2.88	0.65

Under low fatigue stress levels, the increase of fly ash replacement ratio developed a higher displacement evolution between the maximum stress and minimum stress levels. It was noted that low fly ash content of ECC mixtures incorporated with mortar sand exhibited higher deformation capacity under higher fatigue stress level. This can be attributed to the space available between the mortar sand particles due to the small fly ash content which enhanced the micromechanics within the specimen. As shown in Figure 4.9, there was a jump in the deflection developed at a stress level of 75% of the ultimate loading. The incorporation of larger aggregate size in ECC, such as using mortar sand (1.18 mm) instead of micro-silica sand (0.30 mm) developed a higher displacement evolution throughout the increase of the fatigue stress levels. This is due to the enhancement of aggregate interlocking and matrix toughness. Under low fatigue stress levels, the addition of MgO expansive agent a higher displacement evolution between the maximum stress and minimum stress levels. Based on Table 4.9, deflection evolution rate, deflection evolution and

band width absorption energy increased with the increase of stress level representing softening or stiffness degradation as fatigue loading continued.

4.4.3 Third Approach – Self-Healing Characterization under Fatigue Loading at Constant Stress Level

In the third approach, the self-healing behaviour of ECC beams subjected to repeated fatigue loading at constant stress level of $50\% \pm 15\%$ at 4Hz in between vapor curing self-healing periods were investigated. Two beam specimens of each ECC mix were fatigued for 50000 cycles at the age of 28 days, then vapor cured for 2 months to apply fatigue loading for 100000 cycles (one sample was then loaded to failure under monotonic loading) and another one left again for vapor curing for another 1 month to apply fatigue loading for another 100000 cycles before loaded to failure under monotonic loading for analyzing post-fatigue residual performance after 118 days of total healing period. On the other hand, specimens made from each ECC mixes were continuously vapour cured water cured for 118 days in the laboratory (designated as virgin) and then loaded to failure under monotonic loading to assess self-healing performance without application of fatigue loading along with those control specimens loaded to failure under monotonic loading at 28d of lab curing in water.

4.4.3.1 Flexure fatigue performance of Self-healing specimens and Self-healing Evaluation

Figures 4.12 (a-d) shows load-deflection responses of ECC beams from each of the five mixes subjected to fatigue loading after vapor curing at three different ages during 118 days. Table 4.8 summarized the results including deflection evolution rate and band width energy while Table 4.9 summarized crack formation and crack width. It can be noted from Figure 4.12 that the stiffness of the ECC beams increased, deflection evolution decreased, deflection evolution rate decreased and band width energy absorption decreased (also can be noted from Table 4.8) after each curing period. The increase in stiffness can be related to self-healing phenomena where cracks were closed and self-healed. The self-healing behavior is generally stimulated through two phases. The first one is called surface-controlled crystal growth which is responsible for the major formation of CaCO_3 crystals on the crack walls during the first 3 to 5 days. While the second one is called diffusion-controlled crystal growth is responsible for the formation of additional CaCO_3 crystals through diffusing more Ca^{2+} ions from deeper locations of hardened cement paste. In case of ECC-MgO, the hydration of MgO follows the second phase due to the slow rate of MgO hydration. The

formation of MgCO_3 crystals occurred after a long period of vapor curing in order to supplement the formation of CaCO_3 at later ages.

Figure 4.12: Load-Deflection Curves of ECC Beam Specimens at Different Fatigue Cycles

Table 4.8 shows that the highest trend of energy absorption development was attributed to both ECC_F_MS beams produced with either 1.2 or 2.2 FA/C ratios at all fatigue stages when compared with their silica sand counterparts - deflection evolution rate was also found higher for mortar sand beams.

Table 4.8: Mid-span Evolution and Energy Absorption of ECC Beam Specimens at Different Fatigue Cycles

Mix Designation	Flexural Strength (kN)	Mean Fatigue Stress Level (%)	Fatigue Stress Range (%)	Initial Deflection (mm)	Final Deflection (mm)	Mid-Span Evolution Speed Rate ($\mu\text{m}/\text{min}$)	Band Width Absorption Energy (Joule)
50000 Cycles applied after 28d							
ECC F 1.2 SS	10.32	50%	$\pm 15\%$	0.98	1.38	1.92	0.62
ECC F 2.2 SS	7.80	50%	$\pm 15\%$	1.44	1.80	1.76	0.43
ECC F 1.2 MS	10.69	50%	$\pm 15\%$	0.66	1.32	3.20	1.06
ECC F 2.2 MS	9.80	50%	$\pm 15\%$	0.92	1.49	2.72	0.84
ECC_F_1.2_SS_MgO	10.00	50%	$\pm 15\%$	0.83	1.31	2.32	0.72
100000 Cycles applied after 28d+2m							
ECC F 1.2 SS	10.32	50%	$\pm 15\%$	0.87	1.27	0.96	0.62
ECC F 2.2 SS	7.80	50%	$\pm 15\%$	1.08	1.43	0.84	0.40
ECC F 1.2 MS	10.69	50%	$\pm 15\%$	0.55	1.09	1.28	0.87
ECC F 2.2 MS	9.80	50%	$\pm 15\%$	0.67	1.14	1.12	0.69
ECC_F_1.2_SS_MgO	10.00	50%	$\pm 15\%$	0.75	1.20	1.08	0.67
100000 Cycles applied after 28d+2m+1m							
ECC F 1.2 SS	10.32	50%	$\pm 15\%$	0.85	1.22	0.88	0.57
ECC F 2.2 SS	7.80	50%	$\pm 15\%$	0.97	1.29	0.77	0.37
ECC F 1.2 MS	10.69	50%	$\pm 15\%$	0.47	0.98	1.22	0.82
ECC F 2.2 MS	9.80	50%	$\pm 15\%$	0.64	1.03	0.94	0.57
ECC_F_1.2_SS_MgO	10.00	50%	$\pm 15\%$	0.63	1.03	0.96	0.60

Table 4.9: Crack characterization of ECC Beam Specimens at Different Fatigue Cycles

Beam No.	Mix Designation	Crack Characterization	
		Number of Crack	Crack Width (μm)
Beam specimens – fatigued 50000 Cycles applied after 28d			
1	ECC F 1.2 SS	10	<60
2	ECC F 2.2 SS	12	<60
3	ECC F 1.2 MS	6	<60
4	ECC F 2.2 MS	10	<60
5	ECC F 1.2 SS MgO	9	<60
Beam specimens – fatigued (50K+100K) after 28d+2m			
1	ECC F 1.2 SS	13	<60
2	ECC F 2.2 SS	19	<60
3	ECC F 1.2 MS	11	<60
4	ECC F 2.2 MS	16	<60
5	ECC F 1.2 SS MgO	10	<60
Beam specimens – fatigued (50K+100K+100K) after 28d+2m+1m			
1	ECC F 1.2 SS	11	<60
2	ECC F 2.2 SS	16	<60
3	ECC F 1.2 MS	10	<60
4	ECC F 2.2 MS	14	<60
5	ECC F 1.2 SS MgO	11	<60

Figure 4.13 (a) represents the cracks formed of the ECC specimen after applying 50000 cycles of fatigue loading. However, the cracks were self-healed after 2 months of vapor curing in a sealed tank. The widths of cracks were less than $60\mu\text{m}$ wide. Figure 4.13 (b) shows all the cracks were closed/self-healed completely by precipitation of healing compounds after 2 months of vapor curing in a sealed tank.

(a)

(b)

Figure 4.13: (a) Formation of Cracks after Applying 50000 Cycles of Fatigue Loading (b) Self-healed Cracks after 2 Months of Vapor Curing Conditions

During subsequent healing period under vapor all cracks were healed or closed completely and new cracks (crack width remained $<60 \mu\text{m}$) were formed (as shown in Table 4.9) upon re-application of fatigue loading of 100000 cycles at 28d + 2m and 28d + 2m + 1m. The width of The formation of new cracks at new locations can be attributed to the healing of old cracks (confirming formed healing compounds within the cracks were good enough to withstand the loading applied and capable preventing reopening of old cracks at damaged locations) motivating new cracks to be initiated at new locations. Moreover, it can be noted that MgO-ECC mixture is the only mixture that showed an increase in the number of new cracks at all stages. This can be attributed to the delayed formation of MgO hydration which supplemented the formation of C-S-H products and increased the strength of the crack fillers.

4.4.3.2 Self-healing using Static Loading

Figure 4.14 shows the load-deflection response of control (tested after 28 days of water curing), virgin-self-healed (tested after 118 days of vapour curing) and fatigued-self-healed (50000 +100000 cycles -28 d + 2 m and 50000 +100000 + 100000 cycles -28 d + 2 m +1 m = 118 d) obtained from monotonic loading to failure.

Figure 4.14: Flexural Strength-Mid Span Deflection Curves of ECC Beam Specimens

Table 4.10: Flexural Strength and Ultimate Mid-Span Deflection of ECC Mixtures

Mix No.	Mix Designation	Flexural Strength (MPa)		Mid-Span Deflection (mm)	
		Peak	Residual Stress (%)*	Peak	Residual Deflection (%)*
Control beam specimens – tested to failure under static monotonic loading after 28-days of lab curing					
1	ECC F 1.2 SS	11.18	-	2.41	-
2	ECC F 2.2 SS	8.73	-	7.04	-
3	ECC F 1.2 MS	12.16	-	1.58	-
4	ECC F 2.2 MS	9.41	-	2.76	-
5	ECC F 1.2 SS MgO	10.82	-	2.44	-
Virgin Beam specimens – vapor cured for 4 months before loaded to failure under static monotonic loading					
1	ECC F 1.2 SS	9.78	-	1.33	-
2	ECC F 2.2 SS	9.37	-	2.05	-
3	ECC F 1.2 MS	10.64	-	1.53	-
4	ECC F 2.2 MS	8.74	-	1.59	-
5	ECC F 1.2 SS MgO	8.36	-	1.66	-
Beam specimens – fatigued (50K+100K) and vapor cured for 3 months before loaded to failure under static monotonic loading					
1	ECC F 1.2 SS	9.11	93.15* (81.4) ⁺	1.76	132.33* (73.0) ⁺
2	ECC F 2.2 SS	7.15	76.31 (81.9)	1.90	92.68 (26.9)
3	ECC F 1.2 MS	12.43	116.82(102.2)	3.37	220.26 (213.3)
4	ECC F 2.2 MS	9.61	109.95 (102.1)	4.87	306.29 (176.4)
5	ECC F 1.2 SS MgO	10.29	123.09 (95.1)	1.35	81.33 (55.3)
Beam specimens – fatigued (50K+100K+100K) and vapor cured for 3 months before loaded to failure under static monotonic loading					
1	ECC F 1.2 SS	8.73	89.26 (78.0)	2.10	157.89 (87%)
2	ECC F 2.2 SS	7.05	75.24 (80.7)	2.12	103.41 (30.1)
3	ECC F 1.2 MS	11.75	110.43 (96.7)	2.98	194.77 (188.6)
4	ECC F 2.2 MS	9.37	107.21 (99.5)	2.01	126.42 (72.8)
5	ECC F 1.2 SS MgO	9.45	113.04 (87.3)	1.50	90.36 (61.4)

*% of virgin beams; + % of control beams

Tables 4.10 and 4.11 summarized various parameters of test results to characterize self-healing behaviour. As per Table 4.11, all fatigued ECC beam specimens developed less number of cracks compared to their control and virgin ECC counterparts. This might be explained by the self-healing of cracks through the precipitation of calcium and Magnesium based healing compounds. The crack width of failed fatigued-self-healed specimens under monotonic loading ranged between 20 and 80 μm (Table 4.11). It can be noted from Figure 4.10 and Table 4.11 that the residual strength of fatigued 118 days self-healed beam specimens made of mortar sand ECC and ECC-MgO were higher than virgin beams (7 to 13 % higher) which signified fatigue damage repair due to self-healing of cracks. On the other hand, self-healed fatigued beam specimens made of mortar sand ECC and ECC-MgO retained about 87% to 99.5% strength of control specimens - which showed

excellent self-healing potential. All ECC beams except ECC-MgO showed higher residual strength compared to their virgin counterparts while all ECC beams except ECC_F_1.2_MS showed lower residual deflection capacity compared to their control counterparts (Table 4.10). Over all, all ECC beams showed good self-healing potential with MS based ECC showing higher strength regaining potential, followed by ECC-MgO and SS based ECC. The increase in the FA content from 1.2 to 2.2 had reduced the residual strength and hence self-healing potential. The addition of MgO to ECC also improved the residual strength and self-healing capability.

The improvement in residual strength in the self-healed specimens may be caused by the formation of new C-S-H products due to pozzolanic reactions and the formation of CaCO_3 . This chemical reaction is boosted with the presence of small crack width, high water temperature, chemical ions (carbon dioxide and calcium ions Ca^{2+}) and high pH value of water (Edvardsen, 1999; Mihashi and Nishiwaki, 2012). With the vapor curing conditions, water (H_2O) which is carrying carbon dioxide (CO_2) flows inside the small cracks which leads to the formation of carbonates (CO_3) and hydroxide (OH). Those ions will react with the available Ca^{2+} ions from calcium hydroxide ($\text{Ca}(\text{OH})_2$) and calcium silicate hydrate (C-S-H) found at hardened cement paste near the crack walls. As a result, the combination of CO_3 and Ca^{2+} ions forms CaCO_3 crystals that stimulate crack self-healing (Şahmaran et al., 2008).

The fatigued ECC beam specimens incorporated with larger size of aggregate showed a superior performance due to the self-healing property stimulated by vapor curing and small number and widths of cracks. Thus, the use of larger aggregate size results in the increase of residual strength in the self-healed specimens since it will spare a room for the hydration of un-hydrated cementitious particles and CaCO_3 precipitation. A study by Sherir (2012) that the fatigue vibration in larger aggregate size (as in the case of 1.18 mm) tends to increase the friction between cement crystals, un-hydrated fly ash particles and PVA fibers. Thus, the chemical reaction might be stimulated and accelerated resulting in higher residual strength.

The increase in the FA content from 1.2 to 2.2 has reduced the residual strength and residual deflection since the increase in cement replacement ratio reduces the content of Ca^{2+} ions which stimulates the self-healing process and leads to the presence of un-hydrated fly ash particles which are relatively low reactive. The better self-healing potential of ECC-MgO specimens can be attributed to the availability of new cracks provided enough room for MgO hydration within the

cement microstructure to accommodate newly hydration products to supplement both pozzolanic reaction and cement hydration.

Table 4.11: Crack Characterization of ECC Mixtures

Mix No.	Mix Designation	Cracks Characterization	
		Number	Width (μm)
Control Beam specimens – tested to failure under static monotonic loading after 28-days of lab curing			
1	ECC F 1.2 SS	23	30-80
2	ECC F 2.2 SS	35	20-70
3	ECC F 1.2 MS	18	30-60
4	ECC F 2.2 MS	32	20-60
5	ECC F 1.2 SS MgO	20	30-70
Virgin Beam specimens – vapor cured for 4 months before loaded to failure under static monotonic loading			
1	ECC F 1.2 SS	17	30-80
2	ECC F 2.2 SS	22	20-70
3	ECC F 1.2 MS	13	30-60
4	ECC F 2.2 MS	19	20-60
5	ECC F 1.2 SS MgO	20	30-70
Beam specimens – fatigued (50K+100K) and vapor cured for 3 months before loaded to failure under static monotonic loading			
1	ECC F 1.2 SS	15	30-80
2	ECC F 2.2 SS	18	20-70
3	ECC F 1.2 MS	10	30-60
4	ECC F 2.2 MS	12	20-60
5	ECC F 1.2 SS MgO	14	30-70
Beam specimens – fatigued (50K+100K+100K) and vapor cured for 3 months before loaded to failure under static monotonic loading			
1	ECC F 1.2 SS	14	30-80
2	ECC F 2.2 SS	16	20-70
3	ECC F 1.2 MS	7	30-60
4	ECC F 2.2 MS	10	20-60
5	ECC F 1.2 SS MgO	9	30-70

4.5 Chapter summary

Flexure fatigue behaviour of beams made of various ECC mixtures along with their self-healing potentials through description and extensive analysis of experimental results. The fatigue and self-healing performance are described based on the influence of fly ash to cement ratio, size and type of aggregate, MgO addition, fatigue loading and curing schemes - these parameters are found to

affect the structural performance and self-healing potential. The conclusions derived from this chapter will be summarised in Chapter 6.

CHAPTER FIVE

STRUCTURAL PERFORMANCE OF ECC LINK SLABS UNDER MONOTONIC AND FATIGUE LOADING AND EVALUATION OF SELF-HEALING BEHAVIOUR

5.1 Introduction

This chapter presents the results of the experimental investigation on 1/4th scale link slab specimens loaded to failure under static monotonic loading before and after fatigue loading tests along with the fatigue performance developed during 3 months of self-healing in vapour curing. The influence five different types of ECCs (including an ECC made of MgO expansive agent) incorporating different type and size of aggregates (silica sand ‘SS’ and mortar sand ‘MS’), different fly ash (FA) content (as replacement of cement ‘C’) on the structural performance and self-healing potential of link slabs is described compared with self-consolidating concrete (SCC). The comparative study is based on load-deflection responses, moment-rotation response, pre-cracking stiffness, energy absorbing capacity, steel-concrete strain developments, cracking/crack propagation, and ductility as well as recovery of these mechanical properties in self-healed fatigued damaged link slabs.

5.2 Structural Performance of 1/4th Scale ECC Link Slab Specimens Subjected to Monotonic Loading

5.2.1 Experimental Load-Deflection or Moment-Rotation Responses and Influence of ECC material parameters

According to Figure 5.1(a), all ECC link slabs exhibited strain hardening and ductility behaviour by showing a steady increase in the load and deflection up to the peak load with no major drop, and also a gradual decrease in the load with the increase of deflection in the post-peak behavior. This proves the formation of multi-cracks. However, SCC link slab developed a single crack which is associated with a sudden drop in load where the load is transferred to the steel reinforcement. Figure 5.1 (b) represents the moment-rotation responses which exhibited similar patterns as shown in the load-deflection responses. ECC link slabs incorporated with a smaller size (0.30 mm) of aggregate (silica sand) were found to produce a lower rotational stiffness or higher bending capacity which can improve the flexibility of the bridge structure, while the use of larger (1.18 mm) aggregate size (mortar sand) enhanced the load or moment capacity of the link slabs. Compromising the lower bending capacity with the higher ultimate loading capacity may have

negative effects on the ductility of the link slabs leading to the formation of continuous stiff bridge deck with lower rotation capacity of the simply supported adjacent spans connected by link slabs (Lepech & Li, 2009). However, FA-ECC mixtures incorporated with mortar sand show a superior performance with respect of ultimate load and bending capacity.

Furthermore, the increase in FA/C replacement ratio from 1.2 to 2.2 shows a reduction in the load or moment capacity and improvement in the rotation or bending capacity due to the presence of the extra un-hydrated fly ash particles that functioned as filler materials which reduce in chemical bonding and improve the frictional bond between the PVA fibers and matrix (Sahmaran et al. 2008, Yang et al. 2007; Wang & Li 2007; Hossain et al. 2014, 2016). Moreover, improvement of bending capacity which causes lower rotational stiffness of the link slabs enhances the formation of the joint-free bridge decks by allowing a free deformation of the simply supported adjacent spans rather than the formation of continuous stiff bridge spans (Lepech & Li, 2009). According to the finding of Chapter 4, the incorporation of silica sand and FA of 70% of cement replacement (FA/C = 2.2) should result in higher deflection capacity and lower rotational stiffness than of LS1_ECC_F_1.2_SS and of LS4_ECC_F_2.2_MS. The addition of MgO did not improve the flexural strength capacity ECC-MgO link slab compared to the control ECC link slab. This finding of the effect of MgO expansion can be attributed to the large scale of link slabs and the slow reaction of MgO causing a delay in the cement hydration process that produces hydration products C-S-H. According to (Sherir, 2016), specimens that were tested after 450 days did not reveal the expansion effect of MgO on the load/strength performance. The performance of link slab LS2_ECC_F_2.2_SS in terms of load-deflection and moment-rotation responses should not be conclusive.

The key parameters derived from load-deflection and moment-rotation responses for the ECC and SCC link slabs are tabulated in Table 5.1. The peak load of ECC link slab specimens tested to failure under monotonic loaded at the age of 28 days (designated as control specimen) varied from 10.36 to 11.87 KN while the mid-span deflection varied from 9.49 and 13.88 mm. However, the control SCC link slab has shown flexural strength of 5.85 KN and mid-span deflection of 4.18 mm. The ultimate moment capacity of all ECC link slabs varied between 2.10 kNm and 2.40 kNm which was higher compared to SCC link slab.

(a)

(b)

Figure 5.1: Comparison of (a) Load-Deflection and (b) Moment-Rotation Responses of Different ECC and SCC Link Slab Specimens

Table 5.1: Comparative Study Using Load-Deflection and Moment-Rotation Responses

Link Slab No.	Link Slab Designation	Load (kN)		Mid-Span Deflection (mm)			Rotation (radian)		Moment (kN.m)
		Peak	1 st Crack	Peak	1 st crack	Failure	Peak	Failure	Peak
Control-link slab specimens – tested to failure under static monotonic loading after 28-days of lab curing									
1	LS1_ECC_F_1.2_SS	11.56	8.84	13.88	7.54	17.37	0.034	0.044	2.34
2	LS2_ECC_F_2.2_SS	10.41	2.58	10.03	2.01	26.31	0.026	0.068	2.11
3	LS3_ECC_F_1.2_MS	11.87	3.42	9.49	1.51	12.04	0.025	0.029	2.40
4	LS4_ECC_F_2.2_MS	10.89	3.25	11.06	1.70	22.80	0.030	0.065	2.21
5	LS5_ECC_F_1.2_SS_MgO	10.36	2.04	11.84	1.51	25.17	0.029	0.063	2.10
6	LS11_SCC	5.85	5.33	4.18	1.88	10.63	0.012	0.028	1.20

5.2.2 Evolution Initial Stiffness

Table 5.2 summarizes the first cracking stiffness of ECC and SCC link slabs calculated from load-deflection curves (Figure 5.1). Figure 5.2 represents the variation of stiffness with load ECC and SCC link slab specimens.

Table 5.2: Stiffness of Different ECC and SCC Link Slabs

Link Slab No.	Link Slab Designation	Load (kN)		Mid-Span Deflection (mm)		Initial Stiffness (kN/mm)	Stiffness Degradation Loss (%)
		Peak	1 st Crack	Peak	1 st crack		
Control-link slab specimens – tested to failure under static monotonic loading after 28-days of lab curing							
1	LS1_ECC_F_1.2_SS	11.56	8.84	13.88	7.54	1.17	26.32
2	LS2_ECC_F_2.2_SS	10.41	2.58	10.03	2.41	1.07	40.19
3	LS3_ECC_F_1.2_MS	11.87	3.42	9.49	1.51	2.26	43.06
4	LS4_ECC_F_2.2_MS	10.89	3.25	11.06	1.70	1.86	47.09
5	LS5_ECC_F_1.2_SS_ MgO	10.36	2.04	11.84	1.51	1.35	38.52
6	LS11_SCC	5.85	5.33	4.18	1.88	2.96	87.16

Figure 5.2: Stiffness of ECC and SCC Link Slabs

After the first cracking, the stiffness of all ECC link slab specimens were steadily decreased with the increase in load which proves the fiber bridging and strain-hardening characteristics. On the other hand, the SCC link slab developed a higher first cracking stiffness of 2.96 kN/mm followed by a sudden and rapid decrease with the increase of load to reach a stiffness of 0.95 kN/mm. The increase in the aggregate size (use of MS) produces a higher initial stiffness of the ECC link slabs due to producing higher loading capacity through the enhanced friction of aggregate interlocking. Moreover, the increase in the Fly Ash content reduced the stiffness of the link slabs. SCC link slab showed the highest stiffness degradation loss with a percentage of 87.16%, while the loss associated with mortar sand based ECC link slabs (43.06% and 47.09%) is higher than silica sand based ECC link slabs (26.32%, 40.19%, 38.52%).

5.2.3 Energy Absorbing Capacity and Ductility of Link Slabs

The energy absorbing capacity for all ECC and SCC link slabs is summarized in Table 5.3. It is determined by calculating the area under the load-deflection graphs in Figure 5.1 up to the post-peak load of 85% of the peak load. From Table 5.3, the high energy absorbing capacity of ECC link slabs (2.44 to 3.00 times) compared to SCC link slabs demonstrates the strain hardening behaviour accompanied with ECC. Table 5.3 also shows the ductility index calculated based on the ratio of energy at the 85% of the post-peak load to the peak load energy of the link slabs (Wang

& Belarbi, 2011). The high ductility associated with ECC link slabs (1.07 to 1.18 times) is resulted from the gradual post peak failure compared to the brittle failure of the SCC link slab. The increase in FA content improved the energy absorption capacity due to the enhancement in ductility behaviour which allowed the link slabs to achieve higher bending capacity before failure which is resulted from toughness reduction and frictional bonding development between the matrix and the PVA fiber. However, the increase if the size of aggregate energy absorbing capacity and ductility index due to achieving higher peak load at higher stiffness.

Table 5.3: Energy Absorption and Ductility of Different ECC and SCC Link Slabs

Link Slab No.	Link Slab Designation	Load (kN)		Mid-Span Deflection (mm)		Energy (Joules)		Ductility Index	
		Peak	85% of Peak	First Peak	85% of First Peak	Joules	Ratio w.r.to SCC	%	Ratio w.r.to. SCC
Control-link slab specimens – tested to failure under static monotonic loading after 28-days of lab curing									
1	LS1_ECC_F_1.2_SS	11.56	9.83	13.88	11.00	87.04	2.44	114.00	1.08
2	LS2_ECC_F_2.2_SS	10.41	8.85	10.03	13.24	103.61	2.90	123.85	1.18
3	LS3_ECC_F_1.2_MS	11.87	10.09	9.49	15.76	99.71	2.79	113.14	1.07
4	LS4_ECC_F_2.2_MS	10.89	9.26	11.06	14.80	106.93	3.00	118.68	1.13
5	LS5_ECC_F_1.2_SS_MgO	10.36	8.81	11.84	14.13	98.59	2.76	120.05	1.14
6	LS11_SCC	5.85	5.05	2.05	2.16	35.68	-	105.36	-

5.2.4 Strain Development and Crack Characterization

The developments of tensile strain in concrete and steel strain in steel reinforcement are compared in Figure 5.3 and 5.4. A comparative study using strain developments in addition to the crack characteristics (crack width, crack length and crack number) ECC and SCC link slab specimens is presented in Table 5.4.

Table 5.4: Strain Development of Different ECC and SCC Link Slabs

Link Slab No.	Link Slab Designation	Load (kN)	Concrete Tensile Strain at Peak Load (micro-strain)	Steel Strain at Peak Load (micro-strain)	Steel Yield Load		Steel Yielding	Crack Number	Crack Width (μm)	Crack Length (cm)
		Peak			kN	% of peak load				
Control-link slab specimens – tested to failure under static monotonic loading after 28-days of lab curing										
1	LS1_ECC_F_1.2_SS	11.56	3812	3394	9.06	78.3	Yes	23	<80	3-4
2	LS2_ECC_F_2.2_SS	10.41	5094	856	-	-	No	30	<80	3-4
3	LS3_ECC_F_1.2_MS	11.87	5166	3111	9.23	77.76	Yes	25	<80	3-4
4	LS4_ECC_F_2.2_MS	10.89	7173	289	-	-	No	28	<80	3-4
5	LS5_ECC_F_1.2_SS_MgO	10.36	3156	2723	8.83	85.23	Yes	26	<80	3-4
6	LS11_SCC	5.94	116	1371	4.27	71.89	Yes	1	2000	5

Figure 5.3: Concrete Tensile Strain Development of ECC and SCC Link Slabs

Figure 5.4: Steel Strain Development of ECC and SCC Link Slabs

The high tensile ECC strain development at peak load (between 3157 and 7173 microstrain) compared to only 116 microstrain for SCC demonstrates strain hardening characteristics of ECC (Figure 5.3 and Table 5.4). ECC link slabs incorporated with mortar sand developed higher ECC tensile strain capacity due to the better aggregate interlocking (Hossain & Anwar, 2014) which promoted better fiber bridging and micro-cracking characteristics. The high tensile strain capacity in ECC is explained through the fiber bridging behaviour that leads to the formation of additional new cracks until the rupture of PVA fiber (Li et al., 2001). Hence, the tensile behavior of ECC helps in improving ductility, ultimate load and energy absorption capacity of link slab compared to SCC as described earlier (Hossain and Anwar, 2014; Hossain et al., 2015). Whereas, the formation of a sudden major crack in the SCC link slabs led to transferring the load to the reinforcing rebar much earlier as result rebar yielding in SCC link slab occurred at lower load (71.8% of peak load) compared to ECC link slabs (78% to 85% of peak load) (Table 5.4). This can be attributed to the fiber bridging characteristics ECC where at onset of cracking load is transferred to the fiber and transfer of load to the rebar is gradual. The reinforcing steel in ECC incorporated with lower fly ash replacement ratio (FA/C =1.2) reached yield strain of 2100 micro-

strain at lower loads while reinforcement was found to be not yielded for link slabs with higher FA/C ratio.

ECC links showed multiple cracking behaviour the multiple cracks formed in order to endure and pass the load is represented in Table 5.4. This finding is attributed to the tensile strain hardening behaviour of ECC (Li, 1998, 2003). The cracks in ECC link slabs were formed in the debonding zone at the center span surface of the tension face which indicated the flexural behaviour and the stress concentration associated with the designed 5% debonding zone and 2.5% transition zone length with the addition of the shear studs as proposed by Qian et al. (2009).

Figure 5.5 shows a typical cracking pattern of ECC link slabs. ECC link slabs showed multiple cracking (23 to 30 cracks) behaviour with reduced crack width ($<80 \mu\text{m}$). This can be attributed to the tensile strain hardening behaviour of ECC (Li, 1998, 2003). The cracks in ECC link slabs are formed in the debonding zone at the center span on the tension face which indicated the flexural behaviour and the stress concentration associated with the designed 5% debonding zone and 2.5% transition zone length with the addition of the shear studs as proposed by Qian et al. (2009). The SCC link slab developed a major crack and after that the load is transferred to the reinforcing rebars. The crack width was increased to reach 2 mm at failure.

(a)

(b)

Figure 5.5: Crack Characterization of a (a) Typical ECC Link Slab and (b) Typical SCC Link Slab

5.3 Self-Healing Performance of Link Slabs under Fatigue Loading

The five ECC and one SCC link slab specimens were tested under fatigue loading to evaluate the flexural fatigue and self-healing performance. The pre-fatigue behavior of link slab made of each ECC and SCC was determined by applying monotonic loading to failure at the age of 28 days which was required to serve as control specimens (already presented in section 5.2). For self-healing under fatigue loading, link slab specimens made of each ECC and SCC were subjected to fatigue loading at 4 Hz for 50000 cycles at a stress level of 55% of ultimate loading at the age of 28 days and then left for self-healing under vapor curing for 2 months, then loaded for 100000 cycles at a stress level of 55% and left for vapor curing for another 1 month. The specimens were

then loaded to further 100000 cycles at a stress level of 55% of ultimate loading. Finally, the post-fatigue behavior was analyzed by loading specimens under monotonic loading to failure at 118 days of self-healing.

5.3.1 Fatigue Performance under Repeated Fatigue and Curing for Self-healing

The flexural fatigue testing was carried out under load control condition, at a rate of 0.5 kN/min as a ramp until the load reached the mean fatigue stress level of 55%. Then, the fatigue cycles started with an upper and lower limit of $\pm 15\%$ of the applied mean stress level for ECC link slab specimens until the end of the fatigue cycles (50000 cycles at 28d, 100000 cycles at 28d+2m, 100000 cycles at 28d+2m+1m). The mean fatigue stress level was 55% with a stress range of $\pm 5\%$ for the SCC link slab specimen due to the weak performance under fatigue cycles.

5.3.1.1 Load-Deflection Responses

During the fatigue flexural tests, the load and mid-span deflection for every cycle were recorded on a computerized data acquisition system (MTS machine). The load deflection responses for the ECC and SCC link slabs at the three different stages (50000 cycles at 28d, 100000 cycles at 28d+2m, 100000 cycles at 28d+2m+1m) are presented in Figures 5.6-5.9. For the comparative study, key results from load-deflection responses are summarized in Table 5.5 similar to that described in Chapter 4.

It can be noted from Figs. 5.6-5.9 that the deflection evolution, deflection evolution rate and band width energy absorption decreased (also can be noted from Table 5.5) after each curing period. The deflection evolution, deflection evolution rate and band width energy absorption are much higher in ECC link slab compared to their SCC counterparts. Table 5.5 shows that the higher energy absorption development was found in link slab with mortar sand produced with either 1.2 or 2.2 FA/C ratios at all fatigue stages when compared with their silica sand counterparts - deflection evolution rate was also found higher for mortar sand specimens.

Figure 5.6: Load-Deformation Responses of ECC and SCC Link Slabs at Different Fatigue Cycles

Figure 5.7: Load-Deformation Responses of ECC and SCC Link Slabs at 50000 Fatigue Cycles (after 28 days)

Figure 5.8: Load-Deformation Responses of ECC and SCC Link Slabs at 100000 Fatigue Cycles (after 28 days+2months)

Figure 5.9: Load-Deformation Responses of ECC and SCC Link Slabs at 100000 Fatigue Cycles (after 28 days+2months+1month)

Table 5.5: Load-Deformation Responses of ECC and SCC Link Slabs at Different Fatigue Cycles

Link Slab Designation	Load (kN)	Mean Fatigue Stress Level (%)	Fatigue Stress Range (%)	Initial Deflection (mm)	Final Deflection (mm)	Mid-Span Evolution Speed Rate (μm/min)	Band Width Absorption Energy (Joule)
50000 Cycles applied after 28d							
LS6_ECC_F_1.2_SS	11.56	55%	±15%	5.63	6.41	3.75*	1.49
LS7_ECC_F_2.2_SS	10.41	55%	±15%	5.13	5.72	2.83	1.01
LS8_ECC_F_1.2_MS	11.87	55%	±15%	5.75	6.63	4.22	1.72
LS9_ECC_F_2.2_MS	10.89	55%	±15%	4.47	5.10	3.02	1.13
LS10_ECC_F_1.2_SS_MgO	10.36	55%	±15%	4.63	5.31	3.26	1.16
LS12_SCC	5.85	55%	±5%	3.81	4.26	2.16	0.15
100000 Cycles applied after 28d+2m							
LS6_ECC_F_1.2_SS	11.56	55%	±15%	4.01	4.88	2.09	1.66
LS7_ECC_F_2.2_SS	10.41	55%	±15%	4.22	4.92	1.68	1.20
LS8_ECC_F_1.2_MS	11.87	55%	±15%	3.49	4.45	2.30	1.88
LS9_ECC_F_2.2_MS	10.89	55%	±15%	3.80	4.65	2.04	1.53
LS10_ECC_F_1.2_SS_MgO	10.36	55%	±15%	4.21	4.91	1.68	1.20
LS12_SCC	5.85	55%	±5%	3.09	3.45	0.86	0.12
100000 Cycles applied after 28d+2m+1m							
LS6_ECC_F_1.2_SS	11.56	55%	±15%	3.91	4.56	1.56	1.24
LS7_ECC_F_2.2_SS	10.41	55%	±15%	4.10	4.55	1.08	0.77
LS8_ECC_F_1.2_MS	11.87	55%	±15%	3.25	4.15	2.14	1.74
LS9_ECC_F_2.2_MS	10.89	55%	±15%	3.65	4.20	1.32	0.99
LS10_ECC_F_1.2_SS_MgO	10.36	55%	±15%	3.62	4.25	1.51	1.08
LS12_SCC	5.85	55%	±5%	2.70	2.94	0.56	0.08

*(6.41-5.63)/208.33 =3.75

All ECC link slab specimens developed the behaviour of multiple cracking which stimulates the self-healing property. Self-healing mechanisms in ECCs are explained in Chapter 4 and will not be repeated here. It can be noted from Table 5.5 that the band width energy absorption increased for all ECC link slab specimens after the first vapor curing period (2 months) which might be attributed the recovered strength and healed cracks. However, the energy was decreased after the second curing period which can be resulted from the short time that did not allow produce stronger filler materials. The decreased mid-span evolution speed rate after self-healing revealed that ECC link slabs developed less damage after vapor curing.

5.3.1.2 Stiffness Evolution

Table 5.6 shows the degradation of stiffness at the beginning and end of fatigue cycles. Stiffness degradation is attributed to the initiation and formation of cracks. Hence, the fiber bridging and hardening characteristics of ECC link slabs showed a superior performance in the recovery of stiffness compared to SCC. The gradual stiffness degradation (Difference between the initial and final stiffness) occurred due to the repetitive fatigue loading.

Table 5.6: Stiffness Degradation of ECC and SCC Link Slabs at Different Fatigue Cycles

Link Slab Designation	Load (kN)	Mean Fatigue Stress Level (%)	Fatigue Stress Range (%)	Initial Stiffness (kN/mm)	Stiffness at the end of the fatigue cycles kN/mm	Stiffness Degradation %
50000 Cycles applied after 28d						
LS6_ECC_F_1.2_SS	11.56	55%	±15%	2.05	1.80	12.17
LS7_ECC_F_2.2_SS	10.41	55%	±15%	2.03	1.82	10.31
LS8_ECC_F_1.2_MS	11.87	55%	±15%	2.06	1.79	13.27
LS9_ECC_F_2.2_MS	10.89	55%	±15%	2.44	2.14	12.35
LS10_ECC_F_1.2_SS_MgO	10.36	55%	±15%	2.24	1.95	12.81
LS12_SCC	5.94	55%	±5%	1.54	1.07	30.52
100000 Cycles applied after 28d+2m						
LS6_ECC_F_1.2_SS	11.56	55%	±15%	2.88	2.37	17.83
LS7_ECC_F_2.2_SS	10.41	55%	±15%	2.47	2.12	14.23
LS8_ECC_F_1.2_MS	11.87	55%	±15%	3.40	2.67	21.57
LS9_ECC_F_2.2_MS	10.89	55%	±15%	2.87	2.34	18.28
LS10_ECC_F_1.2_SS_MgO	10.36	55%	±15%	2.46	2.11	14.26
LS12_SCC	5.94	55%	±5%	1.89	1.25	33.86
100000 Cycles applied after 28d+2m+1m						
LS6_ECC_F_1.2_SS	11.56	55%	±15%	2.96	2.39	19.33
LS7_ECC_F_2.2_SS	10.41	55%	±15%	2.54	2.14	15.80
LS8_ECC_F_1.2_MS	11.87	55%	±15%	3.65	2.71	25.79
LS9_ECC_F_2.2_MS	10.89	55%	±15%	2.98	2.44	18.12
LS10_ECC_F_1.2_SS_MgO	10.36	55%	±15%	2.86	2.29	20.06
LS12_SCC	5.94	55%	±5%	2.17	1.38	36.41

The three different stages of fatigue loading and vapor curing durations showed consistent findings. First, the increase in the aggregate size (mortar sand) increased the stiffness degradation percentage (maximum 25.8%) of link slab which was attributed to the reduction of the fiber-matrix frictional interface bond and the uniform dispersion of fibers as large size of the aggregates did

not aid the fibers to be coated with the matrix Table 5.6). Second, the increase of fly ash (from FA/C ratio from 1.2 to 2.2) decreased the stiffness degradation percentage of link slab (due to the reduction in chemical bonding and improved frictional bond between the PVA fibers and matrix due to the presence of the extra un-hydrated fly ash particles that functioned as filler materials.

The SCC link slab showed the highest stiffness degradation percentage (36.4%) at a lower stress level ($55\pm 5\%$) compared to maximum 25.9% of ECC link slabs at higher stress level of ($55\pm 5\%$) which can be explained through the low fatigue resistance and stiffness recovery after the formation of a major crack (Table 5.6). It can be noted from Table 5.6 that the initial stiffness is higher after the vapor curing which reflects the self-healing behaviour as the cracks are closed. However, the increase in stiffness degradation after 2 months of vapor curing may be caused by the low cementitious properties of newly formed fillers in the small cracks which can be reopened again after applying higher fatigue cycles.

5.3.1.3 Strain Developments and Crack Characterization

The rebar strain and concrete tensile strain developments of the link slab specimens at different stages of fatigue testing and vapor curing durations are presented in Figures 5.10 and 5.11, respectively. Key results extracted from the graphs are summarized in Table 5.7.

Figure 5.10: Rebar Strain Developments for the Link Slab Specimens

Figure 5.11: Concrete Tensile Strain Developments for the Link Slab Specimens

Table 5.7: Strain Development and Crack Characterization of Link Slab during Fatigue Performance

Link Slab Designation	Peak Load (kN)	Initial Concrete Tensile Strain (micro-strain)	Final Concrete Tensile Strain (micro-strain)	Initial Steel Strain (micro-strain)	Final Steel Strain (micro-strain)	Number of Crack	Crack Width (μm)
50000 Cycles applied after 28d							
LS6_ECC_F_1.2_SS	11.56	1444	1655	510	840	11	30-80
LS7_ECC_F_2.2_SS	10.41	1197	1501	368	390	14	20-70
LS8_ECC_F_1.2_MS	11.87	2209	2243	1266	1325	10	30-60
LS9_ECC_F_2.2_MS	10.89	1872	2178	511	830	12	20-60
LS10_ECC_F_1.2_SS_MgO	10.36	945	1171	514	647	8	30-70
LS12_SCC	5.85	1170	1172	652	11960*	1	1000
100000 Cycles applied after 28d+2m							
LS6_ECC_F_1.2_SS	11.56	587	840	820	990	13	30-80
LS7_ECC_F_2.2_SS	10.41	984	1197	940	1075	17	20-70
LS8_ECC_F_1.2_MS	11.87	1250	1535	1564	1913	11	30-60
LS9_ECC_F_2.2_MS	10.89	1158	1246	810	885	16	20-60
LS10_ECC_F_1.2_SS_MgO	10.36	773	1020	903	1022	12	30-70
LS12_SCC	5.85	-	-	-	-	1(old)	2000
100000 Cycles applied after 28d+2m+1m							
LS6_ECC_F_1.2_SS	11.56	528	606	984	1102	8	30-80
LS7_ECC_F_2.2_SS	10.41	1060	1253	905	990	11	20-70
LS8_ECC_F_1.2_MS	11.87	1243	1380	1818	2030	5	30-60
LS9_ECC_F_2.2_MS	10.89	973	1198	821	900	7	20-60
LS10_ECC_F_1.2_SS_MgO	10.36	609	856	1343	1461	5	30-70
LS12_SCC	5.85	-	-	-	-	1(old)	2000

- strain gauge damaged *reinforcing steel yielded; SCC link slab tested at lower stress level

It can be noted that all ECC link slab specimens developed a large tensile strain band width compared to the SCC link slab specimen. This is attributed to strain hardening characteristic that helps in improving the bending and strain development.

As shown in Figure 5.10, the rebar strain was increased for each ECC link slab specimen with the duration of vapour curing. However, the rebar strain did not yield (2100 micro-strain) even at the end of the final 100000 cycles (28d+2m+1m) (Table 5.7). This finding demonstrates gradual transfer of load to the steel rebar due to the formation of micro-cracks throughout the fatigue cycles.

The ECC tensile strain development was decreased along the curing duration as it is represented in Figure 5.11, even though the number of cycles was increased to 100000 cycles which demonstrated a low damage level compared to the state before self-healing. However, rebar and concrete strain development showed a superior performance of ECC link slabs in terms of flexural fatigue resistance, controlled damage level, ductility and energy absorbing capacity.

All the cracks formed during the fatigue testing in ECC link slabs were self-healed and fully closed after the period of vapor curing. This was not the case of SCC where single crack formed with a width of 2000 μm compared to multiple cracking (maximum 17) with a maximum crack width of 80 μm in ECC link slab (Table 5.7). Figure 5.12 represents the typical self-healing of cracks in ECC link slabs. In every fatigue testing stage, new cracks were formed at different locations. It can be noted from Table 5.7 that the increase in the aggregate size led to the development of less number of cracks and lower crack widths. However, the increase of FA replacement enhanced the link slab specimens to develop more cracks. The addition of MgO expansive agent reduced the number cracks through enhanced the weak bonding materials produced by the calcium precipitations (mainly CaCO_3) through the formation of delayed MgO hydration products as discussed in Chapter 4.

Figure 5.12: Typical Self-healing of Micro-Cracks in ECC Link Slabs during Curing Period in Between Fatigue Loading

5.4 Post-Fatigue Monotonic Performance of Self-healed Specimens and Self-healing Evaluation

After the final 100000 fatigue cycles applied at the age of 118 days, the ECC and SCC link slabs were subjected to flexural monotonic loading to failure in order to determine the residual load-deflection response, energy absorbing capacity and ductility.

Figure 5.13(a-b) represents the residual load-deformation and moment-rotation responses of the ECC and SCC link slab specimens after 3 months of fatigue and self-healing. As shown in Figure 5.13(b), the residual support end-rotation of the self-healed fatigued link slabs satisfied the maximum allowable rotation proposed by AASHTO (2012) of 0.00375 radian at the ultimate/peak loading.

Self-healing performance is assessed in terms of residual mechanical properties (strength, deflection, stiffness and energy absorbing capacity) of healed-fatigued link slab specimens (compared to their control counterparts) after loading to failure under static monotonic loading as summarized in Table 5.8. It should be noted that SCC link slab was tested at lower fatigue stress level as it could not survive higher stress levels applied to ECC link slab. For such inferior structural performance, SCC link slab should be not suitable for joint-free bridge deck construction. Therefore, residual properties of ECC links slabs cannot be compared with those of their ECC counterparts for self-healing assessment.

(a)

(b)

Figure 5.13: Comparison of Post Fatigue (a) Load-Deflection and (b) Moment-Rotation Responses of Different ECC and SCC Link Slab Specimens

The concrete and steel strain developments for the self-healed ECC link slab specimens were lower than their control counterparts (Figures 5.14 and 5.15). The steel rebars of the ECC link slabs did not yield even at failure load which demonstrated that post-fatigued healed ECC matrix is strong/stiff (due to accumulation of stronger healing compounds that justified the lower tensile strain development in ECC) enough to sustain the applied load and reduce the transfer of load to the steel. This is attributed to the better self-healing due to fiber bridging and micro-cracking characteristics of ECC. Moreover, all fatigued healed ECCs retained their multi-cracking characteristics (developing 14 to 18 cracks compared to 1 in SCC) and tight crack width (ranged between 20 and 80 μm) characteristics which can improve log-term strength, ductility, durability of link slab in practical joint-free bridge construction.

Figure 5.14: Steel Rebar Strain Development for Link Slab Specimens

Figure 5.15: Concrete Tensile Strain Development for Link Slab Specimens

Table 5.8: Comparative Study of Peak Load, Deflection, Stiffness and Energy Absorbing Capacity

Link Slab No.	Link Slab Designation	Strength/Load (kN)		Mid-Span Deflection (mm)		Stiffness (kN/mm)		Energy (Joules)	
		Peak	% of control	Peak	% of control		% of control		% of control
Control-link slab specimens – tested to failure under static monotonic loading after 28-days of lab curing									
1	LS1_ECC_F_1.2_SS	11.56	-	13.88	-	1.17	-	87.04	-
2	LS2_ECC_F_2.2_SS	10.41	-	10.03	-	1.07	-	103.61	-
3	LS3_ECC_F_1.2_MS	11.87	-	9.49	-	2.26	-	99.71	-
4	LS4_ECC_F_2.2_MS	10.89	-	11.06	-	1.86	-	106.93	-
5	LS5_ECC_F_1.2_SS_MgO	10.36	-	11.84	-	1.35	-	98.59	-
6	LS11_SCC	5.85	-	4.18	-	2.96	-	35.68	-
Healed link slab specimens – fatigued and vapor cured for 3 months before loaded to failure under static monotonic loading									
7	LS6_ECC_F_1.2_SS	6.72	58.13	3.52	25.36	2.17	185.47	57.64	66.22
8	LS7_ECC_F_2.2_SS	7.65	73.49	4.53	45.16	2.92	272.90	48.21	48.35
9	LS8_ECC_F_1.2_MS	11.17	94.10	9.05	95.36	1.59	70.35	103.79	97.06
10	LS9_ECC_F_2.2_MS	10.2	93.66	3.05	27.58	3.62	194.62	82.93	67.09
11	LS10_ECC_F_1.2_SS_MgO	10.18	98.26	8.38	70.78	2.00	148.15	87.88	89.14
12	LS12_SCC*	3.99*	68.21	2.26	54.07	3.92	132.43	10.53	29.51

*Fatigue loading was applied at lower stress level and results cannot be compared

According to Table 5.8, the residual strength, stiffness and energy absorbing capacity of self-healed fatigued ECC link slabs ranged between 58.3% and 98.26%, between 70.35% and 194.62% and between 48.35% and 97.06%, respectively of control link slab. Use of ECC with large aggregate size (mortar sand) exhibited excellent strength recovery of link slab ranging between of 93.66% and 98.26% of their control counterparts. The larger aggregate size facilitated the hydration process of self-healing and enhanced the strength by providing a higher degree of aggregate interlocking. However, the addition of MgO showed a significant recovery of the strength/peak load after self-healing which demonstrates the expansion effect MgO and capability to produce self-healing compounds. All ECC link slabs showed high stiffness recovery of up to 273% except LS8_ECC_F_1.2_MS which can be explained by the very high stiffness of its control counterpart amongst all other ECC link slab specimens. LS8_ECC_F_1.2_MS and LS10_ECC_F_1.2_SS+MgO showed the best structural performance due to attaining high residual

strength of 94.1% and 98.3%, respectively of control, residual deflection capacity of 95.36% and 70.78% respectively and residual energy absorbing capacity of 97.06% and 89.14%, respectively as well as retaining strain hardening and multiple cracking characteristics even after applying fatigue loading over 3 months.

5.5 Chapter Summary

Experimental results of ECC link slabs are described based on monotonic loading to failure as well as fatigue loading coupled with self-healing and post-fatigued monotonic loading to failure with assessment of self-healing compared to SCC link slab. The conclusions derived from this study are described in Chapter 6.

CHAPTER SIX

CONCLUSIONS

6.1 General

This research investigated the mechanical properties (such as compressive/flexural strength) and self-healing behaviour under flexural fatigue loading of five different Engineered cementitious composite (ECCs) mixtures incorporated with different contents of fly ash (FA) class F as cement (C) replacement, two sizes/types of aggregates (silica ‘SS’ and mortar sand ‘MS’) and addition of MgO-type expansive agent. Percentages of supplementary cementing materials (fly ash class F) were 55% and 70% (with FA/C ratios of 1.2 and 2.2 by mass, respectively) and the maximum grain sizes of SS and MS were 0.30 mm and 1.18 mm, respectively while the aggregate/binder ratio and water/binder ratio were kept constant at 0.36 and 0.27, respectively. The main variables in the fatigue testing were the application of different number of cycles (up to 400000 cycles at 4Hz) and different fatigue stress levels (25%, 50%, 75% and 100%) while self-healing behaviour of ECC beams under repeated fatigue loading were investigated under two stage repeated curing between the three stage application of total 250000 fatigue cycles over total self-healing duration of 118 days before fatigued/self-healed beams were loaded failure under monotonic loading. Moreover, a series of tests were carried out in order to study the structural performance and self-healing behaviour of link slabs made of five different ECC mixtures compared to their self-consolidating concrete (SCC) counterparts under fatigue loading at a constant fatigue stress level of 55% and total 118 days of curing (two stage) during a total 250000 cycles (at 4Hz) applied in three stages before loaded to failure under monotonic loading. The performance was analyzed based on load-deformation/moment-rotation responses, stiffness, crack characterization, steel/concrete strain developments, ductility, energy absorbing capacity, deflection evolution speed rate, crack-healing, recovery of strength/deflection capacity and residual strength/deflection/energy absorbing capacity.

6.2 Conclusions

The following conclusions were drawn from the study:

- The compressive strength decreased with the increase of Class F-fly ash replacement with cement from 1.2 to 2.2 without changing the type and size of aggregates. This is attributed to the slower pozzolanic reaction of FA compared to the hydration of cement. On the other hand, the incorporation of larger aggregate size in ECC, such as using mortar sand (1.18 mm) instead of micro-silica sand (0.30 mm), without changing the cement replacement rate increased the compressive strength due to the available volume between the aggregate that allows more paste content between the aggregate and thus leads to an increase of the water content in the interfacial transition zone (ITZ).
- The compressive strengths of all ECC mixtures ranged between 43 and 59 MPa at the age of 28 days, 46 and 62 MPa at the age of 56 days, and 50 and 67 MPa at the age of 90 days. Those values make the proposed ECC mixtures suitable and adequate for bridge deck application.
- All five ECC mixtures exhibited multiple-cracking behaviour under flexural monotonic loading. The average ultimate flexural strengths of ECC mixtures ranged from 8.73 to 12.16 MPa and the bending capacity of ECC beams varied from 1.88 to 7.61 mm at the age of 28, 56 and 90 days.
- It was found that the increase of Class F-fly ash replacement with cement from 1.2 to 2.2 without changing the type and size of aggregates resulted in a reduction in the flexural strength while the mid-span deflection and the bending capacity were notably enhanced. This may be attributed to the reduction in chemical bonding and improvement of the frictional bond between the polyvinyl alcohol (PVA) fibers and matrix due to the presence of the extra un-hydrated fly ash particles that functioned as filler materials. However, the reduction of the PVA fiber/matrix chemical bonding and the matrix toughness reduces the flexural strength.

- The incorporation of larger aggregate size in ECC, such as using mortar sand (1.18 mm) instead of micro-silica sand (0.30 mm), without changing the cement replacement rate improved the ultimate load capacity while reduced the mid-span deflection. This improvement is attributed to the enhancement of aggregate interlocking and matrix toughness with the use of larger aggregate size. On the other hand, the reduction in mid-span deflection was caused by the reduction of the fiber-matrix frictional interface bond and the uniform dispersion of fibers as large size of the aggregates did not aid the fibers to be coated with the matrix.
- The addition of MgO did not improve the flexural strength capacity ECC-MgO specimens compared to the ECC specimens. This might be related to the slow reaction of MgO causing a delay in the cement hydration process that produces hydration products C-S-H.
- For all five ECC mixtures, the widths of micro cracks were less than 60~80 μ m even under ultimate loading; this intrinsic material property can stimulate crack self-healing behaviour. The increase in the FA content from 1.2 to 2.2 in the ECC mixtures tends to increase the number of crack and decrease the average crack widths at all ages due to the reduction in PVA fiber/matrix interface chemical bond and matrix toughness. However, the incorporation of larger aggregate size enhances the aggregate interlocking and matrix toughness for cracks propagation which leads to lower number of the cracks and a slight decrease in the average crack widths compared to the use of micro-silica sand. Moreover, the addition of MgO expansive agent to ECC specimens helped in developing less number of cracks compared to their ECC counterparts.
- When applying 4000000 fatigue cycles, the high deflection evolution speed rate associated with the FA-ECC mixtures with mortar sand reflected the tendency of generating more deflection under fatigue loading due to ductility behavior.

- According to the step fatigue approach, the increase of the fatigue stress level led to developing more ductility, less stiffness and high band width absorption energy in terms of the mid-span evolution speed rate. Under low fatigue stress levels, the increase of fly ash replacement ratio developed a higher displacement evolution between the maximum stress and minimum stress levels. However, low fly ash content of ECC mixtures incorporated with mortar sand exhibited higher deformation capacity under higher fatigue stress level. This can be attributed to the space available between the mortar sand particles due to the small fly ash content which enhanced the micromechanics within the specimen.
- The fatigued ECC beam specimens incorporated with larger size of aggregate showed a superior performance due to the self-healing property stimulated by vapor curing and small number and widths of cracks. This might be attributed to the aggregate interlocking that inhibits widening of the cracks. However, the virgin specimens at the age of 118 days showed an inferior performance since there was no more room for hydration and reaction of un-hydrated cementitious particles.
- The use of larger aggregate size results in the increase of residual strength and residual deflection in the fatigued self-healed beam specimens since it will spare a room for the hydration of un-hydrated cementitious particles and CaCO_3 precipitation. However, the increase in the FA content from 1.2 to 2.2 has reduced the residual strength and residual deflection since the increase in cement replacement ratio reduces the content of Ca^{2+} ions which stimulates the self-healing process and leads to the presence of un-hydrated fly ash particles.
- ECC link slabs incorporated with a smaller size (aggregate (silica sand)) were found to produce a lower rotational stiffness or higher bending capacity while the use of larger size aggregate (mortar sand) enhanced the load or moment capacity of the link slabs. FA-ECC mixtures incorporated with mortar sand show a superior performance with respect of ultimate load and bending capacity.

- The increase in FA/C replacement ratio from 1.2 to 2.2 shows a reduction in the load or moment capacity and improvement in the rotation or bending capacity of link slabs. The addition of MgO did not improve the flexural strength capacity ECC-MgO link slab compared to the control ECC link slab.
- SCC link slab showed the highest stiffness degradation loss under static monotonic loading with a percentage of 87.16%, while the loss associated with mortar sand based ECC link slabs was (higher than silica sand based ECC link slabs).
- The high energy absorbing capacity and ductility of ECC link slabs were 2.44 to 3.00 times and from 1.07 to 1.18 times higher respectively than their SCC counterparts.
- It was observed that all cracks formed during fatigue loading either in beam or link slab specimens were completely closed through the deposition of calcium and magnesium hydration products in the cracks during vapour cured stepped curing between each stage application of fatigue loading.
- ECC link slabs showed multiple cracking behaviour with reduced crack width ($<80\text{ }\mu\text{m}$) while SCC link slab developed only a major crack at failure.
- The deflection evolution, deflection evolution rate and band width energy absorption are much higher in ECC link slab compared to their SCC counterparts. Higher energy absorption development and deflection evolution rate were observed in mortar sand based specimens during fatigue loading coupled with self-healing.
- The SCC link slab showed the highest stiffness degradation of 37% even at lower range of stress level compared to maximum 26% of ECC link slabs tested at higher stress range during fatigue loading.

- Rebar and concrete strain development during fatigue loading coupled with self-healing showed a superior performance of ECC link slabs in terms of flexural fatigue resistance, controlled damage level, ductility and energy absorbing capacity.
- All multiple cracks (with maximum crack width of 80 μm) formed during the fatigue testing in ECC link slabs were self-healed and fully closed which was not the case for SCC where single crack formed with a width of 2000 μm .
- The residual support end-rotation of the self-healed fatigued link slabs satisfied the maximum allowable rotation proposed by AASHTO (2012) of 0.00375 radian at the ultimate/peak load.
- SCC link slab was tested at lower fatigue stress level as it could not survive higher fatigue stress levels applied to ECC link slab. SCC link slab should be not suitable for joint-free bridge deck construction due to such inferior structural performance.
- The residual strength, stiffness and energy absorbing capacity of self-healed fatigued ECC link slabs ranged between 58.3% and 98.26%, between 70.35% and 194.62% and between 48.35% and 97.06%, respectively of control link slab.
- ECC link slabs with mortar sand having fly ash content of 55% with and without MgO showed the best self-healing and fatigue loading performance attaining high residual strength, deflection capacity and energy absorbing capacity of up to 98.3%, 95.36% and 97.06% respectively of control as well as retaining strain hardening and multiple cracking characteristics even after applying fatigue loading over 3 months.
- This research demonstrated the viability of using ECC link slab for the construction of joint-free bridges with enhanced structural performance, durability and self-healing capability.

6.3 Recommendations for Future Work

Further extensive experimental and theoretical investigations need to be conducted in order to achieve a better understanding of the mechanical and structural performance of ECC link slab and the full bridge as well. This can be done through testing specimens at higher fatigue stress levels and fatigue number of cycles along with changing different key parameters such as debonding zone length and reinforcing ratio.

The following recommendations are made for future research studies:

- Additional fatigue testing needs to be carried out at higher fatigue stress levels and fatigue number of cycles to achieve a better understanding of the mechanical and structural performance along with the failure modes of ECC link slab.
- Further investigation related to steel reinforcement ratio is recommended in order to reach an adequate correlation between the concrete strength and steel yielding in ECC link slab bridge deck.
- Developing new mix designs that are characterized with the early age strength properties of ECC mixtures and the long term self-healing properties of MgO-ECC mixtures.
- For better understanding, research should be conducted on ECC link slab behaviour in real bridge applications. Finite element modelling and field demonstration should also be carried out.

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