Use of Advanced Composite Materials for Innovative Building Design Solutions

by

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ABSTRACT

Advanced composite materials become popular in construction industry for the innovative building design solutions including strengthening and retrofitting of existing structures. The interface between different materials is a key issue of such design solutions as the structural integrity relies much on the bond. Knowledge on durability of concrete/epoxy interface is becoming essential as the use of these systems in applications such as FRP strengthening and retrofitting of concrete structures is becoming increasingly popular. Prior research studies in this area have indicated that moisture affected debonding in a FRP-bonded concrete system is a complex phenomenon that may often involve a distinctive dry-to-wet debonding mode shift from material decohesion (concrete delamination) to interface separation (concrete/epoxy interface) in which concrete/epoxy interface becomes the critical region of failure. Such premature failures may occur regardless of the durability of the individual constituent materials forming the material systems. Thus, the durability of FRP-bonded concrete is governed by the microstructure of the concrete/epoxy interface as affected by moisture ingress. In this work, fracture toughness of concrete/epoxy interfaces as affected by combinations of various degrees of moisture ingress and temperature levels is quantified. For this purpose, sandwich beam specimens containing concrete/epoxy interfaces are tested and analyzed using the concepts of fracture mechanics. Experimental results have shown a significant decrease in the interfacial fracture toughness of concrete/epoxy bond with selected levels of moisture and temperature conditioning of the specimens. The strength of adhesive joint degrades as implied by the failure mode shift from concrete decohesion in controlled specimens to interface separation in conditioned specimens. In this thesis, primary data on the mixed mode fracture toughness of concrete/epoxy interfaces are presented as a basis for use in the design improvement of material systems containing such interfaces for better system durability, and issues related to the structural implications are also discussed.

Thesis Supervisor: Oral Büyükoztürk

Title: Professor of Civil and Environmental Engineering
This thesis is dedicated to my love Ant
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Chapter 1

Introduction

1.1 Background

A composite can be defined in a broad sense as a material containing two or more constituent materials, with each material keeping its own identity. Advanced composite materials consist of fibers embedded in a resin matrix. In particular, fiber reinforced polymer (FRP) composites contains strong and stiff fibers embedded in a matrix with distinct interfaces between them. In this combination, both fibers and matrix retain their physical and chemical identities, yet they produce new properties that cannot be achieved with either of the constituents alone. The aim of combining fibers and resins that are different in nature is to take advantage of the distinctive material features of either component to result in an advanced material with desired overall composite action for specific applications. The advantages of advanced composite materials include lightweight, strong and durable, which are the favorable characteristics as construction materials.

The development of advanced composite materials has constituted a revolution in materials applications in recent years. The remarkable stiffness and strength to weight of fibers, along with other properties such as environmental resistance, make composite materials increasingly popular as potential candidates for materials substitution. Composite materials are being increasingly utilized in many fields including both military and commercial applications. During the last 35 years, various fibers have become important. Early applications for carbon and boron fibers were in military aircraft. While military aviation was at the forefront in usage of
composites, other markets such as commercial aviation and sporting goods have grown significantly, inducing a significant reduction in the material costs due to their increased production. This also enables large potential markets in transportation and infrastructure applications. Currently, these materials such as carbon fiber reinforced polymers (CFRP) are widely used in strengthening the building and bridge structures, and retrofitting of structures for earthquake resistance. They also have a great potential for the innovative use in building structures, such as earthquake resistant structures.

The advantages of these materials include lightweight, thus easy installation, excellent material properties, and high durability, meaning that it is also an economic and effective solution for earthquake effect. Therefore, there has been an increased use of the advanced composite materials in building and bridge construction, as well as in retrofit projects, worldwide in the past two decades.

However, there are two major issues related to the use of advanced composite material in construction projects that need to be addressed. Firstly, although by themselves these materials are durable, as used in conjunction with concrete to form a concrete/composite bonded system, their durability under environmental conditions may become an issue. Environmental effects, such as the variation of moisture and temperature, can degrade the mechanical performance of the bonded system. Secondly, the assessment and non-destruction test of the bonded systems should be carried out from time to time in order to make sure their integrity.

The study on the durability of concrete/composite system will be potentially the innovative solutions for new structural design and retrofit or upgrading project for buildings and bridges. It is very important because the knowledge and implications developed in this work will help ensure the public safety against unanticipated failures of concrete/composite systems. The novelty of advanced composite material in conjunction with traditional construction materials such as concrete brings a broad impact to the construction industry as it is now a worldwide used material.
1.1.1 Use of Advance Composite Materials with Structures

FRP composites were first developed in 1930s (MDA 2004). From 1930s to 1970s, the use of these materials was mostly limited to advanced applications due to high material costs. In the past 30 years, there has been a gradual shift of interest in the composites industry from performance based manufacturing to cost based manufacturing forced by the military defense spending and the low gear in civil aerospace industry. Nowadays, military and aerospace industries have the lowest share in the composites market, while construction industry occupies the second largest portion in the market. Applications of FRP composites in the construction industry include cables and tendons, girders, beams, columns, bridge deck systems, all composite pedestrian bridges and buildings, rebars and grids, external reinforcement systems, and hybrid applications (composites combined with wood, aluminum, and concrete) such as FRP-confined concrete columns and marine pilings. Various infrastructure applications of FRP composites are briefly discussed as follows.

Cables and Tendons

Composites offer superior advantages in application where unidirectional tensile strength is needed. This property together with excellent corrosion and fatigue resistance, and light weight makes composites the ideal materials for use as pre- and post-stressing tendons. FRP cables and tendons have disadvantage of higher initial cost compared to steel cables. The cost of glass fiber cables was estimated to be 2.5 times more than comparable 200-ksi steel cables, and carbon fiber cables cost about 5 times more than comparable 270-ksi steel cables. However, due to the reduced transportation and handling costs, low maintenance, and much longer service life (no cable replacement anticipated for structures having a life span of 80-100 years), composite cables become more advantageous when the total project cost and life-cycle costs
are considered rather than initial material costs.

In spite of the favorable material properties and durability characteristics of composites, there are a number of special technical and durability issues regarding their use as cables in bridges and structures. A key design issue for composite cable is how to design a suitable anchorage system which can fully utilize the tensile strength of the FRP cable. Traditional anchorage system for steel cable is not suitable for this purpose due to the low transverse strength of the composite cable. Also, the potential galvanic corrosion of carbon fiber cables used together with metals and the corrosion of glass fiber cables are critical issues of adopting composite cables. Research studies in these areas were conducted worldwide, such as United States, Switzerland, and Korea. Through newly developed FRP cable anchorage system, more than 90% of the cables can reach its ultimate strength. Durability issues can be largely decreased or even eliminated through the use of proper resin barriers that prevent contact of carbon fibers with metals, or exposure of glass fiber cables with the environment.

**Bridge Deck Systems**

Deterioration of bridges, limited budgets, and the increase of load demands have resulted in a significant bridge deck replacement market. FRP manufacturers have quickly responded to this tremendous opportunity by developing integrated bridge deck system that offer the advantages of FRP composites. Light weight of composite deck system greatly decreases the dead load on the bridge girders and stringers, and directly translates into increased bridge load capacity. Prefabrication of deck results in a better quality assurance and a reduction in construction time and cost. Cost competitiveness of FRP bridge deck system, like many other FRP applications, increases when total project cost and especially life cycle costs are considered rather than initial material costs.
**Composite Bridges and Buildings**

Usually the most economical use of FRP composites in structures is to use them when their favorable properties are needed most, and to use conventional construction materials elsewhere. It is because of the high cost of FRP composites and their relatively weaker performance under transverse and compression loading. In a limited number of applications, however, structures were made entirely from FRP composites either for demonstration purposes or because their durability characteristics make construction of such structures feasible. GFRP materials are relatively commonly used for the construction of small FRP cabin units applied in industrial plants and construction sites. However, the construction of large buildings from FRP is not quite common.

**Rebars and Grids**

Cracking of reinforced concrete and consequent corrosion of the steel reinforcement is the cause of most durability problems related to existing reinforced concrete infrastructures. Deterioration is accelerated by saline or other chemically aggressive environments. FRP rebars were produced to remedy the corrosion problems in reinforced concrete structures. FRP rebar not only resists oxidation and corrosion form deicing salts, marine and other aggressive environment, but also has a better strength to weight ratio than steel. FRP rebars are mostly made of glass fibers to lower the costs, but even then they are still significantly more costly than steel rebars. The cost of GFRP rebars is about $3 per pound and CFRP rebars typically cost more. Epoxy coated steel rebar, on the other hand, costs about 30 cents per pound. Despite the significant difference in cost, light weight, durability and low maintenance requirement of FRP rebars make their use feasible in certain application such as bridge deck construction. FRP rebars are also manufactured in grids that make their installation in bridge decks much easier.

Technical problems related to the use of FRP rebars are their low stiffness compared to
steel, their brittle behavior, inability of bending and welding them on site, and long term load performance. FRP rebars displays a linear elastic stress-strain behavior followed by a brittle failure without a yielding plateau which is heavily relied upon with the conventional steel rebars. In order to prevent premature brittle failures, these rebars must be used with high design factors of safety. Also considering the long term stress relaxation and relatively low fatigue load performance of glass fibers, it is generally recommended that the tensile stress in the glass fiber rebars should not exceed 25 to 30 percent of their ultimate strength. For this reason, GFRP rebars are considered more suitable for the use in secondary structural elements rather than in primary load carrying members. Another restriction is the instability to bend and weld on site. Specific designed shapes must be coupled where needed with these rebars, which require additional considerations in the design stage.

Marine Piling

Deterioration of waterfront structures has always been a challenging problem especially in marine environment and at splash zone. Considering thousands of government and private waterfront structures, it is apparent that an advancement/solution satisfying the performance requirements will make a major economical impact. Private and government funded research in this area has conducted development programs to evaluate products that could retrofit or replace the existing members used in waterfront structures, with emphasis on FRP composite piling and FRP reinforced concrete piles. Long term and low maintenance solutions offered by FRP composites have resulted in the development of various products such as fender piles, sheet piles, and end bearing or friction piles. These products were installed in many waterfront structures in the last twenty years.
Structural Strengthening and Retrofitting

Rapid deterioration of the world's existing infrastructures, continuously updated design codes, and faults in design and construction of structures have created an enormous amount of money in the rehabilitation market worldwide. Backlog of substandard structures is increasing at a rate faster than the ability to rehabilitate them using conventional methods of repair, retrofit, and strengthening, which include concrete and steel jacketing, bonding steel plates, and integration of new structural elements. However, all these methods are costly, time consuming, and most important of all, are still susceptible to the same deterioration problems. The pressing need for new, effective and economical rehabilitation methods have lead to research in the area of using FRP composite materials for structural strengthening and retrofitting. Externally bonded FRP reinforcements were found to be a viable and promising technique for various strengthening applications due to their high performance, light weight, and superior durability.

FRP composites have been used for strengthening and retrofitting various structural members in numerous experimental, demonstration, and field projects since the 1980s. Early research and applications in this area were conducted in Japan, Germany, and Switzerland. Beginning from the early 1990s, several countries including United States, Canada, and Saudi Arabic joined efforts in this research area and have investigated various analysis, design, application, and durability aspects of retrofitting with composites. Encouraging results obtained from numerical and experimental studies, as well as pilot field applications, has brought it to the commercial retrofitting applications. Starting from late 1980s, more that 1,500 structures around the world have been strengthened or retrofitted using FRP composites [Gune (2004)].

In United States, research and applications of FRP composite in strengthening and retrofitting projects lagged almost a decade behind Japan and Europe. In west coast, research concentrated mainly on seismic retrofitting of columns due to the large number of earthquake prone bridges in that area with substandard columns in urgent need of retrofit and repair. By
In 1994, more than fifteen projects were completed in California and several other states [Fyfe (1994)]. Field applications on beams and slabs lagged behind because of the problems associated with delamination and shear failures [Triantafillou and Plevris (1992), Berset (1992)]. A considerable research effort has been invested in this area and much progress has been made although the problem has not yet been solved [Büyükozturk et al (2002)]. Continued research is needed to develop reliable debonding models for safe design of beam strengthening. More recently, seismic retrofitting of beam-column connections using FRP composites have been investigated and successfully tested in laboratory and field studies [Gergely et al. (1998), Geng et al. (1998)]. Since the 1990s, field applications of FRP strengthening have been performed on columns, beams, slabs and chimneys by several specialized. For beams, slabs and shear walls, strengthening by FRP materials can improve the flexural and shear capacities of the corresponding structural elements [Grace et al. (1996), Triantafillou (1998), Deniaud and Cheng (2003)]. For columns, FRP sheets are used for wrapping the columns to improve their ductility and axial load capacity [Saadatmanesh et al. (1994), Seible et al. (1997)]. FRP composites have also been used in strengthening of steel girders [Sen et al. (2001), Liu et al. (2001)] and repair of fatigue-damaged steel members [Basetti et al. (2000), Büyükoztürk and Hearing (1998), Büyükoztürk et al. (2002)] as well as wood and masonry structures [Triantafillou (1998), Saadatmanesh (1997)].

In the past 20 years, there has been an extensive research on the use of advanced composite materials with structures, especially for reinforced concrete structures. The bonding of FRP plates/sheets to reinforced concrete structures has become a popular method for strengthening and retrofitting the existing structures. In the strengthening or retrofitting process, epoxy is most commonly used to bond FRP and concrete together. Prior research studies [Au and Buyukozturk (2006)] showed that the existence of epoxy is critical and the FRP-bonded concrete system should be considered as a tri-layer system consisting concrete, epoxy and FRP,
rather than simply a bi-material system of concrete and FRP.

1.1.2 Durability of Advanced Composite Materials

Durability of composites against environmental factors such as temperature cycles and extremes, moisture, chemical attack, and ultraviolet (UV) radiation is a major concern in structural applications. Although composites have not been around long enough to develop extensive knowledge on their durability, these materials are known to be significantly more durable than the conventional construction materials such as steel, reinforced concrete, and timber. Composites are inherently corrosion-resistant and can show substantial cost benefits when used in aggressive environments. The effect of temperature cycles and extremes on the properties of most commercial fibers is found to be insignificant. However, depending on the type and properties of fiber and the matrix, certain environmental factors can cause degradation in the mechanical properties of composites. For example, moisture is known to accelerate static fatigue in glass fibers. Many polymer-matrix composites tend to absorb moisture from the surrounding environment, resulting in dimensional changes as well as adverse internal stresses within the material. To remedy this, resin systems having very good resistance to the effects of moisture can be selected. Epoxy, for instance, is known to exhibit very low moisture absorption. UV radiation is known to cause degradation in polymers by scission of the polymer chains. Using appropriate coatings that screen the UV radiation can largely diminish this problem.

Although the use of advance composite materials, such as epoxy and fiber reinforced polymer (FRP) composite, for the retrofit of deteriorating and under-strength reinforced concrete structures through external bonding and wrapping has been extensively characterized through proper laboratory testing and numerous field installations, the long term durability of these bonded systems remains largely uncertain and unanswered. This is mainly due to the lack of validated databases for materials and systems used in civil infrastructure applications in conjunction with the use of manual technique which results in significant variability.

While research of the short-term behavior in these bonded systems has been quite
extensively studied, research in durability has been very limited. Nevertheless, recent experimental evidence [Grace (2004)] has suggested that structural performance of reinforced concrete structure retrofitted by advanced composite materials (FRP) could be significantly impeded due to the introduction of various environmental effects, and that the debonding at the interface region constitutes a major reason to such premature failures. In particular, moisture effect has been identified as an important environmental deterioration mechanism promoting premature system failures. Grace [Grace (2004)] has conducted an array of tests on 78 large-scale retrofitted RC beams using carbon FRP (CFRP) plates and fabrics that were subjected to various environmental effects including water, saltwater, freeze-thaw, dry heat, and alkalinity. After 10,000 hours of respective continuous exposure, it was concluded that moisture could do the most damage in terms of ultimate beam strength, resulting in a 30% and 10% respective reduction for CFRP plate and CFRP sheet bonded beams. Debonding was observed as the primary failure mode, although details of such failures were not described.

The findings in this comprehensive investigation regarding the detrimental effect of moisture were inline with other limited number of independent studies conducted on FRP retrofitted concrete beams (either with or without rebars) that were subjected to accelerated wet/dry cycles. Reduction in ultimate strength and stiffness was generally observed and at times at alarming magnitudes. Chajes et al. [Chajes et al. (1995)] have shown a 36% decrease in ultimate strength for glass FRP (GFRP) retrofitted specimens, subjected to 100 wet/dry cycles, while a 19% reduction for carbon FRP (CFRP) bonded specimens. Toutanji and Gomez [Toutanji and Gomez (1997)], on the other hand, observed a strength reduction that ranged from 3% to 33% on specimens made of various epoxy and FRP systems that were subjected to 300 wet/dry cycles in salt water. The failure was reported as a debonding mode that generally took place near the FRP/concrete interface. Mukhopadhyaya et al. [Mukhopadhyaya et al. (1998)] found similar degradation trends on GFRP double lap shear specimens that were related to comparable wet/dry conditioning. Karbhari et al. [Karbhari et al. (1998)] have observed micro-cracking in
the GFRP laminate in retrofitted cement mortar beams that were mechanically tested to failure under a four-point bend configuration after 120 days of continuous moisture exposure and reported a 40% reduction in flexural strength. The research group of Professor Oral Büyükoztürk [Au and Büyükoztürk (2006)] has also recently discovered a consistent shift in failure mode from the material decohesion type when dry (e.g. concrete delamination, epoxy decohesion, and FRP delamination depending on the epoxy being used) to an interface separation mode at the epoxy/concrete interface when the CFRP plate-bonded concrete systems under the prolonged continuous moisture exposure.

1.1.3 Interface Fracture Mechanics

Interface fracture mechanics is used to describe the fracture phenomena which take place in a multi-layer material system where interfaces exist between adjoining materials. Unlike normal fracture of a homogeneous isotropic solid, interface fracture in a multi-layer system usually involves high order of complexity in terms of stress fields, crack propagation and kinking. Although loading and geometric symmetry usually implies the pure fracture mode in an isotropic solid, elastic mismatch in most cases lead to mixed mode behavior even under symmetric loading and specimen geometry. Analysis of mixed mode behavior can also be very tedious which often involving complex variable, numerical solutions and advanced elasticity analyses. Despite the above difficulty, interface fracture mechanics is an essential tool to characterize debonding behavior in a layered material system, one of which, of course is the concrete/epoxy system that has been the central theme of this thesis.

The concept of kink criterion – crack propagation always follow the minimum energy paths – helps the understanding of the crack propagation under multi-layer material system. In a FRP bonded concrete systems, although the pre-crack (or parent crack) can initially form at a biomaterial interface (e.g. epoxy/concrete), subsequent crack propagations do not necessarily
follow the direction of the parent crack. In many instances, this parent crack will kink out of the interface and enter the more fracture prone concrete substrate. The behavior of crack kinking is governed by the relative fracture toughness of the interface over the substrate material. When

\[ \frac{\Gamma_i}{\Gamma_s} > \frac{G_i}{G_{\text{max}}} \]  

(1.1)
is satisfied, the crack is likely to kink out of the interface since the interface is relatively tougher than the substrate material [He and Hutchinson (1989), He et al. (1991)]. Here, \( \Gamma_i \) is the interface fracture toughness, \( \Gamma_s \) the substrate fracture toughness in mode I, \( G_i \) the interface fracture energy release rate (of the parent crack), and \( G_{\text{max}} \) the maximum fracture energy release rate for the kink crack at any putative kink angle. Since \( G_i \) is a case of \( G_{\text{max}} \), the \( G_i/G_{\text{max}} \) is always between 0 and 1. The fracture toughness values should be determined experimentally.

In this thesis, the concrete/epoxy interface is our main interest in the sense of the study on multi-layer structural systems. It is because based on the prior research conducted at M.I.T. [Au (2005), Au and Büyüköztürk (2006), Büyüköztürk et al. (2008)], it has been showed that concrete/epoxy interface is the most critical region when subjected to moisture attack while FRP/epoxy interface is more durable against moisture.

### 1.2 Research Objectives

The objective of this research is to develop an in-depth mechanistic understanding through proper quantification of interfacial properties in concrete/epoxy system with the consideration of moisture degradation so as to form the basis for a systematic guideline to the future development in innovative building design solutions.

### 1.3 Research Approach
From the review of prior studies on strengthening and retrofitting of reinforced concrete systems using FRP composite, it can be seen that while FRP-bonded concrete system has been extensively studied, the structural performance of FRP-bonded concrete system when subjected to moisture ingress at the interface remains largely uncertain and unanswered. In particular, little is known regarding the durability of concrete/epoxy interface which has been shown as a critical region under the effect of moisture. While the strength approach is capable of quantifying and analyzing material decohesion type of debonding, it intrinsically lacks the ability to describe adhesion related phenomenon. Interface fracture toughness is, on the other hand, considered as a bond property of the multi-layer material system, and is a quantification parameter central to this thesis.

A practical study on such phenomenon is essentially experimental. A linear elastic interface fracture mechanics theory as developed by Hutchinson and his co-workers [Hutchinson et al. (1992), Hutchinson (1990)] is used to process the data from fracture tests. The specimens were sandwiched four-point bending beams, consisting of an epoxy layer bearing an artificial crack at the interface between concrete block and epoxy layer. The specimens were moisture conditioned with different durations ranging from 2 weeks to 10 weeks. They were then tested with the loading angle, which is a measurement of the contribution of Mode I versus Mode II. (Refer to Chapter 7 for more detailed information about the research approach.)

This research is motivated from the durability concerns of this widely used method for concrete retrofit applications. It is anticipated that this work can yield much needed scientific knowledge and quantitative information; and through the studies on the concrete/epoxy system, it can provide the essential knowledge as a basis for a life cycle predictive capability for FRP-bonded concrete system subjected to moisture ingress.

1.4 Thesis Organization
The thesis consists of eight subsequent chapters.

**Chapter 1** presents a review on the advanced composite materials used in concrete structures with particular concerns on durability issue, and examines the characteristics of interface and interface fracture in concrete/epoxy system when subjected to moisture.

**Chapter 2** reviews the diffusion theory which is applicable to concrete and epoxy.

**Chapter 3** reviews the interface fracture mechanics theory as applicable to concrete/epoxy system.

**Chapter 4** explains the theoretical background of sandwich beam fracture specimens.

**Chapter 5** presents the results from diffusion characterization of both concrete and epoxy which are subjected to moisture exposure.

**Chapter 6** presents the results from the extensive material testing that characterize the mechanical properties of both concrete and epoxy with increasing moisture content.

**Chapter 7** details the fracture test program that features both mode I and mixed mode fracture characterization of concrete/epoxy system. Sample preparation, experimental setup, loading and instrumentation are discussed. Results including interfacial fracture toughness variations and debonding modes are presented and summarized.

**Chapter 8** summarizes the thesis and draws conclusions from the work. Areas for future investigation are also presented.
Chapter 2

Moisture Diffusion

This chapter provides the theoretical background regarding the moisture diffusion in concrete and epoxy. Diffusion is defined as the process by which matter is transported from one part of a system to another as a result of random molecular motions, or called as Brownian motions [Crank (1975)]. Such motions are however governed by a concentration gradient. Thus, there is a net movement of these molecules flowing from a higher concentration region to a lower concentration region as a result of the random molecular motions.

Moisture diffusion in solid is a phenomenon where water molecules flow from a wetter environment to a dryer environment. The rate of water transfer through a unit area of a section is directly proportional to the concentration gradient measured normal to that section, which can be expressed by Fick's first law of diffusion:

$$F = -D \frac{\partial C}{\partial x}$$  \hspace{1cm} (2.1)

where $F$ is the rate of transfer per unit area of section, $C$ is the concentration of water, $x$ is the spatial coordinate measured normal to the section, and $D$ is the diffusion coefficient. The negative sign in the above equation means that diffusion takes place in the direction of decreasing concentration. With respect to time, the rate of diffusion follows Fick's second law of diffusion:

$$\frac{\partial C}{\partial t} = \frac{\partial}{\partial x} \left( -F \right) = \frac{\partial}{\partial x} \left( D \frac{\partial C}{\partial x} \right)$$  \hspace{1cm} (2.2)

The above expression describes one-dimensional diffusion. Note here that $D$ may or may
not be a constant. It may vary with time during which diffusion has been taking place.

2.1 Types of Diffusion Behavior

Moisture diffusion has been traditionally classified according to the relative rates of diffusion and relaxation [Alfrey et al. (1996)]. There are three classes which are stated as follows:

✓ Case I is referred to as Fickian diffusion in which the rate of diffusion is much less than that of relaxation.

✓ Case II is referred to as the other extreme opposed to Case I in which diffusion is very rapid when compared to the relaxation.

✓ Case III is referred to as non-Fickian diffusion in which diffusion occurs at a rate comparable to the rate of relaxation. This case falls somewhere between Case I and Case II.

When the amount of moisture sorption at time \( t \) is described by \( kt^\alpha \) where \( k \) and \( \alpha \) are empirical constants which depend on geometry and material such that:

\[
\frac{M_t}{M_s} = kt^\alpha
\]  (2.3)

\( \alpha \) can then be defined in the range as follows:

✓ Case I: \( \alpha = 0.5 \)

✓ Case II: \( \alpha \geq 1 \)

✓ Case III: \( 0.5 < \alpha < 1 \)

In the above expression, \( M_t \) represents the mass of water absorbed at time \( t \) and \( M_s \) is the mass of water absorbed at saturation. Considering a concrete/epoxy system, diffusion within the constituent materials usually belongs to Case I or Case III as conducted based on the existing experimental data [Au (2005)]. Figure 2.1 shows the different types of Case III diffusion when compared with Case I diffusion. In most cases, Fickian diffusion or Pseudo-Fickian diffusion is observed [Alfrey et al. (1996)]. The term Pseudo-Fickian was first used by Rogers in
It describes the sorption-desorption curves of the same general shape as in Fickian diffusion, except that the initial linear portion persists for a shorter time. It is noted that the mass uptake curve for ideal Fickian diffusion would exhibit the following characteristics:

- Initial slope is linear
- Linear limit exceeds 60% of the saturation mass uptake
- Nonlinear portion of curve is concave in shape (decreasing slope)
- Ideal mass uptake curves should be generated regardless of coupon thickness

![Diagram of Fickian and Pseudo-Fickian diffusion curves](image)

Figure 2.1: Diffusion behavior of Case I and Case III [Crank (1975)]

### 2.2 Diffusion Coefficient

Diffusion coefficient is defined as the rate at which a diffusion takes place. A higher diffusion coefficient value corresponds to a faster rate of diffusion in a given time duration. In
order to measure the diffusion coefficient of a material, most researchers assume this parameter as a constant. When such assumption does not valid for the material, a mean value is often obtained instead. In the cases of concrete and epoxy, constant diffusion coefficients can be assumed without much loss of accuracy. Measurement of their values experimentally is also much simpler. One method of doing so is through the gravimetric sorption technique, which involves the generation of the overall moisture update curve as shown in Figure 2.2 by submerging a thin coupon of material in a water bath so that one-dimensional double-sided diffusion takes place.

One can get a relationship which describes the above moisture uptake behavior relatively accurately by solving Fick’s second law [Crank (1975)] as follows:

\[
\frac{M_t}{M_s} = \frac{4}{\sqrt{\pi}} \left( \frac{D t}{h^2} \right)^{0.5}
\]

where \(M_t\) is the mass of water sorbed at time \(t\); \(M_s\) is the mass sorbed at equilibrium state (or saturation); \(h\) is the thickness of the material coupon; \(D\) is the diffusion coefficient. Hence, \(D\) can be computed from the initial linear slope of the mass uptake curve. For Pseudo-Fickian diffusion, \(D\) can be averaged up to the point of 60% of the initial portion (i.e. \(M_t/M_s = 0.6\)). It is noted that \(M_t/M_s\) is always between 0 and 1.

2.3 Estimation of Time to Saturation

It can be observed from the mass uptake expression that at saturation, the coupon thickness, \(h\), can be correlated to the time to saturation, \(t_s\), as follows:

\[
h = 4 \sqrt{\frac{D t_s}{\pi}}
\]

At a constant moisture sorption temperature, \(T\), the diffusion coefficient of the material remains unchanged. Based on equation (2.5), it is noticed that the coupon thickness varies with the
square root of the time to saturation. Disregarding the complex coupling of simultaneous diffusion of heat and moisture, $D$ will be factored into the rate process as follows when $T$ is allowed to vary [Crank (1975)].

$$t_{s,2} = \left( \frac{D_{T_1}}{D_{T_2}} \frac{h_2}{h_1} \right)^2 t_{s,1}$$  \hspace{1cm} (2.6)

where $D_{T_1}$ and $D_{T_2}$ are the diffusion coefficients at temperature $T_1$ and $T_2$ for the same material respectively; $h_1$ and $h_2$ are the respective coupon thickness; $t_{s,1}$ and $t_{s,2}$ are the respective time to saturation.

![Typical Pseudo-Fickian Moisture Uptake Curve](image)

Figure 2.2: Typical Pseudo-Fickian Moisture Uptake Curve

### 2.4 Summary Comments

Theoretical background on moisture diffusion, the one important phenomenon central to this research, is laid out. In reality, we are interested in the moisture content at the interface of FRP-bonded concrete system for a large scale structural element, such as beam, column and slab. Three-dimensional diffusion modeling is a critical simulation activity diffusion process.
along the interface because such modeling enables us to understand the interaction between water and multi-layer material system. The theoretical background is therefore very important for such modeling. The simulation is performed by solving Fick's diffusion law, using diffusion coefficients as presented and defined in this chapter that are experimentally determined and tabulated in Chapter 4. The diffusion property characterization conducted in this thesis serves as a basis for the future large scale diffusion modeling which will link the fracture theories with physical observations of the debonding modes by means of interfacial moisture concentration as well as moisture affected material properties.
Chapter 3

Interface Fracture Mechanics

The objective of interface fracture mechanics is to seek a useful definition of interfacial fracture toughness in order to assess the behavior of the bi-material systems using this parameter. Interface fracture toughness is a material property that characterizes the resistance against fracture at a biomaterial interface. This chapter reviews the fundamental concepts of interface fracture mechanics as applied to a bi-material system. It is based on the recent works of Hutchinson and his co-workers [Rice (1998), Suo (1989), Evans et al. (1989), Hutchinson (1990), Hutchinson et al. (1992)]. This review is an altered version of a chapter in K.M. Lee’s Ph.D thesis [Lee (1993)], U. Trende’s S.M. thesis [Trende (1995)] and C. Au’s Ph.D. thesis [Au (2005)].

3.1 Bi-material Elasticity

A composite of two homogeneous materials with continuity of traction and displacement is called a bi-material. No matter how many layers are there in a layered material system, an interface can only be formed by adjoining material, which is considered as a bi-material system. In the following, only isotropic bi-material under plane strain conditions will be discussed. Consider a bi-material composite that consists of two dissimilar homogeneous isotropic materials that are bonded together with an interfacial crack as shown in Figure 3.1. The upper layer is defined as material 1 and the lower layer is defined as material 2. Let $E_1$, $\mu_1$, and $\nu_1$ be the
Young's modulus, shear modulus, and Poisson's ratio of material 1, whereas \( E_2, \mu_2, \) and \( \nu_2 \) be theses quantities of material 2.

![Diagram of an interfacial crack](image)

Figure 3.1: Geometry and conventions for an interfacial crack

Dundurs [Dundurs (1969)] has observed that a wide class of plane problems of bi-material elasticity only depends on two non-dimensional quantities which are the combinations of the elastic moduli. Those factors, further called as Dundurs parameters \( \alpha \) and \( \beta \), are defined as follows:

\[
\alpha = \frac{\overline{E}_1 - \overline{E}_2}{\overline{E}_1 + \overline{E}_2}, \quad \beta = \frac{1}{2} \frac{\mu_1 (1 - 2\nu_2) - \mu_2 (1 - 2\nu_1)}{\mu_1 (1 - 2\nu_2) + \mu_2 (1 - 2\nu_1)}
\]

where \( \overline{E} = E/(1 - \nu^2) \). The parameter \( \alpha \) describes the relative stiffness of the two materials. \( \alpha \) approaches +1 when material 1 is much stiffer than material 2 while -1 when material 2 is much stiffer than material 1. Both \( \alpha \) and \( \beta \) vanish when there is no dissimilarity of the elastic properties. \( \alpha \) and \( \beta \) also change signs when the materials layers are switched. The parameter \( \beta \) vanishes only in plane strain and when the two materials are incompressible which means that the Poisson's ratio of both materials 1 and 2 equals 0.5. Figure 3.2 shows the Dundurs
parameters of some common material combinations in a FRP-bonded concrete system, together with those in electric packaging industry where debonding is considered as a critical issue. The concrete/epoxy bi-material system is the focus of this thesis.

Figure 3.2: Values of Dundurs parameters for selected bi-material combinations

3.2 Crack Tip Fields

Considering a semi-finite, traction-free interfacial crack between two homogeneous isotropic half-planes with material 1 above and material 2 below as shown in Figure 3.1, it is noted that there are two coordinate systems and no specific length or load is present in this problem [Suo (1989)]. Solutions to this problem were first presented by England, Erdogan, Rice and Sih in 1965 [England (1965), Erdogan (1965), Rice et al. (1965)]. Also, William discovered the oscillatory near tip behavior of an interfacial crack in 1959 [Williams (1959)]. The notations...
of Rice [Rice (1988)] are used in this thesis because of the advantages of reducing to the conventional notation when the elastic mismatch vanishes.

For two-dimensional problems, the normal and shear stresses of the singular field acting on the interface at a distance $r$ ahead of the crack tip are given by:

$$
\sigma_{22} + i\sigma_{12} = \frac{K}{\sqrt{2\pi r}} r^{ie}
$$

(3.2)

Eqn. (3.2) can also be written as:

$$
\sigma_{22} = \text{Re}[Kr^{ie}(2\pi)^{-1/2}], \quad \sigma_{12} = \text{Im}[Kr^{ie}(2\pi)^{-1/2}]
$$

(3.3)

where $K = K_1 + iK_2$ is the complex stress intensity factor, $i = \sqrt{-1}$ and $\varepsilon$ is the oscillation index. It can be written as a function of $\beta$ as follows:

$$
\varepsilon = \frac{1}{2\pi} \ln \left( \frac{1 - \beta}{1 + \beta} \right)
$$

(3.4)

This parameter introduces some complications which are not present in the elastic fracture mechanics of homogeneous solids. It is noted that $K_1$ and $K_2$ do not directly measure the normal and shear stress singularities at the interface ahead of the crack tip because of the term $r^{ie}$ in Eqn. (3.2). The associated horizontal and vertical crack face displacements at a distance $r$ behind the crack tip, $\delta_1 = u_1(r, \theta = \pi) - u_1(r, \theta = -\pi)$ and $\delta_2 = u_2(r, \theta = \pi) - u_2(r, \theta = -\pi)$, are given by:

$$
\delta_2 + i\delta_1 = \frac{8K}{E(1 + 2i\varepsilon)\cosh(\pi\varepsilon)} \left( \frac{r}{2\pi} \right)^{ie}
$$

(3.5)

Let us define an average stiffness $E'$ for the sake of convenience. We have:

$$
\frac{1}{E'} = \frac{1}{2} \left( \frac{1}{E_1} + \frac{1}{E_2} \right)
$$

(3.6)

The corresponding energy release rate per unit area crack extension [Malyshev et al. (1965)] is:
\[ G = \frac{1}{E \cosh^2(\pi \varepsilon)} |K|^2 \]  

(3.7)

where \(|K|^2 = K_i^2 + K_o^2\) and \(\cosh^2(\pi \varepsilon) = 1/(1 - \beta^2)\). Referring to Eqn. (3.3), the phase angle \(\psi\), which is a measure of the contribution between peel and shear mode, is defined as:

\[ \psi = \tan^{-1} \left( \frac{\text{Im}(KL^{ie})}{\text{Re}(KL^{ie})} \right) \]  

(3.8)

where \(L\) is a reference length. The measure of the proportion between peel and shear modes in the vicinity of a crack tip requires the specification of a length quantity because the ratio of the normal traction to the shear traction varies slowly with the distance measured from the crack tip when \(\beta \neq 0\). It is noted that \(\psi\) is always non-zero due to the fact that there is an elastic moduli mismatch across the interface even under pure mode I loading conditions, which means that the geometry and the loading are symmetric with respect to the crack plane. The reference length \(L\) can be chosen quite freely. One can choose a distance based on an in-plane length of the specimen geometry, such as the crack length. Also, one can choose a distance based on the material length scale, such as the size of the fracture process zone. In this thesis, the former \(L\) is sued because it is useful for discussing the mixed mode character of a bi-material interfacial crack, which is independent of individual material fracture behavior.

### 3.3 Interfacial Fracture Toughness with \(\beta = 0\)

The fracture mode at an interfacial crack between two different materials is usually mixed. The differences in the elastic moduli disrupt the symmetry even when the loading and the geometry are symmetric with respect to the crack plane. For any interfacial crack problem, the complex \(K\) will have the form:

\[ KL^{ie} = YT \sqrt{L} e^{i\psi} \]  

(3.9)
where $T$ is the applied traction loading, $L$ is a reference length, $Y$ is a dimensionless real positive number, and $\psi$ is the phase angle of the quantity $KL^{ie}$. However, $\psi$ is often called the phase angle of the stress intensity factor or loading phase angle. Both $Y$ and $\psi$ generally depend on the ratios of various applied loads and lengths, as well as $\alpha$ and $\beta$.

From the maximum value of $T$ at the onset of crack propagation, the critical interfacial fracture toughness $|K|_c$ can be calculated according to Eqn. (3.9). It is obvious that this value is also a function of the phase angle. In general, the toughness increases with increasing phase angle.

It is noted that the fracture toughness depends on the choice of $L$. In fact, there is a freedom in the choice of $L$ in the definition of $\psi$ is a consequence of a simple transformation rule from one choice to another. Let $\psi_1$ be associated with $L_1$ and $\psi_2$ associated with $L_2$. Based on Eqn. (3.8), one can show that:

$$\psi_2 = \psi_1 + \varepsilon \ln \left( \frac{L_2}{L_1} \right)$$

(3.10)

Therefore, the toughness data can be transformed easily [Rice (1988)].

The results presented before become much simpler for conditions which satisfy $\beta = 0$, referring to Eqn. (3.1). When $\beta = 0$, it implies $\varepsilon = 0$ and hence Eqn. (3.2) becomes:

$$\delta_2 + i\delta_1 = \frac{8(K_1 + iK_2)}{E^*} \left( \sqrt{\frac{r}{2\pi}} \right)$$

(3.11)

Here, the interface stress intensity factors $K_1$ and $K_2$ are analogous to their counterparts in linear elastic fracture mechanics for homogeneous and isotropic solids. Due to the difficulties in decoupling the normal and shear components for $\beta = 0$ and the fact that for most concrete/epoxy interfaces $|\beta|$ is small (about 0.1), it is convenient to set $\beta = 0$. Note that with $\varepsilon = 0$, the phase angle is given by
\[ \psi = \tan^{-1}\left( \frac{K_2}{K_1} \right) \]  

(3.12)

With \( \varepsilon = 0 \), \( \psi \) is indeed the phase angle. Here, \( \psi = 0^\circ \) corresponds to pure Mode I (peel mode) and \( \psi = 90^\circ \) corresponds to pure Mode II (shear mode).

### 3.4 Summary Comments

In this chapter, a review of interface fracture mechanics is given based on the work of Rice, Suo, Evans and Hutchinson [Rice (1998), Suo (1989), Evans et al. (1989), Hutchinson (1990), Hutchinson et al. (1992)]. The concept of fracture mechanics applied on bi-material system is directly related to my current work with concrete/epoxy system, which is also a bi-material. In Chapter 4, focus will be made on the application of such concept on a particular type of fracture specimens and in this thesis sandwich beam specimens is chosen as the basis of the experimental analysis. The sandwich beam specimens are used to characterize the fracture toughness of concrete/epoxy system under different loading scenarios. Besides, the formulation shown in this chapter will be used as a basis for designing fracture specimens of concrete as well. The assumption of \( \beta = 0 \) will be adopted throughout the analysis in this thesis because it simplifies the equation with complex variables to a neat and manageable form, but still without introducing significant errors which will be demonstrated clearly in Chapter 4.
Chapter 4

Interface Fracture Model: Sandwich Test Specimen

4.1 Universal Relation for Sandwich Beams

There are, in general, two test methods for obtaining interface fracture parameters for bi-material system, namely the Brazilian disk specimen \([\text{Hutchinson (1990), Lee (1993)}]\) and the sandwiched beam test specimen \([\text{He et al. (1990)}]\). It is reported that there is an abrupt change in failure mode of Brazilian disk specimen with increasing specimen size \([\text{Bazant (1992)}]\); while the failure mode of sandwich beam specimen does not change with size. Hence, the size effect should be considered during the experimental preparation if the Brazilian disk specimens were used. Here, it should be clarified that size scale effect is definitely the most important practical consequence of fracture mechanics. It does affect the fracture parameter, such as interface fracture energy, evaluated based on the experiment. In sandwich beam specimen, size scale effect is still present. It affects the fracture energy derived by experiment. However, based on the prior study conducted at M.I.T. related to the sandwich beam specimen, it was found that the failure mode does not change with the size \([\text{Trende (1995)}]\). As size effect is not the focus of this thesis, sandwich beam test specimens were chosen instead of Brazilian disk specimens.

Let us consider a generic case of a interfacial crack as shown in Figure 4.1. A universal relation between the interface stress intensity factors, \(K_I\) and \(K_{II}\), and the applied stress intensity factors, \(K_I\) and \(K_{II}\), can be formulated when the thickness of the aggregate layer \(h\) is small compared to the crack length \(a\) \([\text{Suo et al. (1989)}]\). For a homogenous material under in-plane
loading, the interface fracture parameters can be characterized by the Mode I and Mode II stress intensity factors, $K_i$ and $K_{II}$. The traction at a distance $r$ ahead of the crack tip is given by:

$$\sigma_{22} + i\sigma_{12} = \frac{K^\infty}{\sqrt{2\pi r}}$$

where $K^\infty = K_I + iK_{II}$. The corresponding energy release rate is:

$$G = \frac{1}{E_1} \left( K_I^2 + K_{II}^2 \right)$$

where $E_1 = E_i/(1 - \nu_i^2)$. It appears from energy arguments that Eqn. (3.7) and Eqn. (4.2) must equal to each other. Hence,

$$|Kh^ic| = p |K^\infty|$$

where $p = \sqrt{(1 - \alpha)/(1 - \beta^2)}$ and $|Kh^ic| = |K|$. Based on Eqn. (4.3), the universal relation between the interface stress intensities and the applied stress intensities can be obtained as a complex notation:

$$Kh^ic = p K^\infty e^{i\omega(\alpha, \beta)}$$

Or it can be written as a sum of real and imaginary part as follows:

$$K_1 + iK_2 = p(K_I + iK_{II})h^{-ic} e^{i\omega(\alpha, \beta)}$$

in which $\omega(\alpha, \beta)$ is a real function. The value of $\omega$ can be found from Table 4.1 when $\alpha$ and $\beta$ are known [Suo et al. (1989)]. The parameter $\omega$ can be regarded as the shift in phase of the interface stress intensity factors with respect to the applied stress intensity factors. It should be noted that $\omega$ is a non-dimensional function of Dundurs' parameters $\alpha$ and $\beta$.

Meanwhile, it is defined that:

$$Kh^ic = |K| e^{i\omega^*}, \quad K^\infty = |K^\infty| e^{i\phi}$$

where $\omega^* = \tan^{-1}[\text{Im}(Kh^{ic})/\text{Re}(Kh^{ic})]$ is the phase angle and $\phi = \tan^{-1}(K_{II}/K_I)$ and
This definition, together with Eqn. (4.4) yields the following result:

\[ \psi^* = \phi + \omega \]  

(4.7)

If one sets \( L_1 = h \) and \( L_2 = L \) in Eqn. (3.10) and substitutes Eqn. (4.7) into Eq. (3.10), the loading phase angle can be shown as:

\[ \psi = \psi^* + \varepsilon \ln \left( \frac{L_2}{L_1} \right) = \phi + \omega + \varepsilon \ln \left( \frac{L}{h} \right) \]  

(4.8)

It is noted that \( \psi \) corresponds to \( \psi_1 \) and \( \psi^* \) corresponds to \( \psi_2 \). With a choice of \( L \) as a fixed length, one can calculate the loading angle \( \psi \) from Eqn. (4.8). The interface stress intensity, \( K_1 \) and \( K_2 \), can be calculated from the applied stress intensity factors, \( K_i \) and \( K_{ii} \), by using the universal relation given in Eqn. (4.5). Because \( \beta \) is usually small when compared to \( \alpha \) and the dependence of \( \omega \) on \( \beta \) is weak, one can make an assumption that \( \beta = 0 \) and Eqn. (4.5) becomes:

\[ K_1 + iK_2 = \sqrt{1 - \alpha} (K_i + iK_{ii}) e^{i\omega(\alpha)} \]  

(4.9)

in which \( \omega \) is a function of \( \alpha \) only.

Table 4.1: \( \omega \)-values as a function of Dundurs' parameters \( \alpha \) and \( \beta \) [Suo et al. (1989)]

<table>
<thead>
<tr>
<th>( \beta )</th>
<th>( \alpha )</th>
<th>-0.8</th>
<th>-0.6</th>
<th>-0.4</th>
<th>-0.2</th>
<th>0.0</th>
<th>0.2</th>
<th>0.4</th>
<th>0.6</th>
<th>0.8</th>
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<tbody>
<tr>
<td>-0.4</td>
<td>2.2</td>
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<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>-0.3</td>
<td>3.0</td>
<td>4.0</td>
<td>3.3</td>
<td>1.4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-0.2</td>
<td>3.6</td>
<td>4.1</td>
<td>3.4</td>
<td>2.0</td>
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<td>-3.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-0.1</td>
<td>4.0</td>
<td>4.1</td>
<td>3.3</td>
<td>2.0</td>
<td>0.1</td>
<td>-2.3</td>
<td>-5.5</td>
<td>-10.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.0</td>
<td>4.4</td>
<td>3.8</td>
<td>2.9</td>
<td>1.6</td>
<td>0.0</td>
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<td>-4.7</td>
<td>-8.4</td>
<td>-14.3</td>
<td></td>
</tr>
<tr>
<td>0.1</td>
<td></td>
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<td>1.1</td>
<td>-0.5</td>
<td>-2.3</td>
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</tr>
<tr>
<td>0.2</td>
<td></td>
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<td>-3.0</td>
<td>-4.9</td>
<td>-7.3</td>
<td>-10.5</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>0.3</td>
<td></td>
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<td></td>
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<tr>
<td>0.4</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>-11.1</td>
</tr>
</tbody>
</table>
It should be mentioned that Table 4.1 is used to find the corresponding value of $\omega$ when $\alpha$ and $\beta$ are known. In the concrete/epoxy system, once the material property of both concrete and epoxy are characterized experimentally, Eqn. (3.1) can be used to evaluate $\alpha$ with the assumption $\beta = 0$.

4.2 Four-point Bending Specimen for Testing in Mode I

We consider the four-point bending specimen with a sandwiched epoxy layer shown in Figure 4.1. Proper techniques are required to sandwich an epoxy layer into the concrete and ensure that the crack stays along one of the interfaces. The apparent stress intensity factor $K_i$ can be obtained from the stress analysis of cracks handbook [Tada et al. (1985)] as:

$$K_i = f_i \sigma_r \sqrt{\pi a} \quad (4.10)$$

where $\sigma_r = 6M/\pi b^2$ in which $M$ is the applied moment, $a$ is the crack length, $b$ is the width, $d$ is the height of the specimen. $f_i$ is a correction factor for four-point pure bending which can be expressed in terms of the relative crack length $(a/d)$:

$$f_i = 1.122 - 1.4 \left( \frac{a}{d} \right) + 7.33 \left( \frac{a}{d} \right)^2 - 13.08 \left( \frac{a}{d} \right)^3 + 14.0 \left( \frac{a}{d} \right)^4 \quad (4.11)$$

The interface stress intensity factors can be obtained by substituting Eqn. (4.10) into Eqn. (4.5). With the assumption that $\beta = 0$, we have:

$$K_1 = \sqrt{1 - \alpha} \left( f_i \sigma \cos \omega \right) \sqrt{\pi a}, \quad K_2 = \sqrt{1 - \alpha} \left( f_i \sigma \sin \omega \right) \sqrt{\pi a} \quad (4.12)$$
Based on Eqn. (4.2), the fracture energy release rate can be calculated as

$$ G = \frac{K^2}{E_1} = \frac{f_i^2 \sigma^2 \pi a}{E_1} $$

(4.13)

Since for mode I, $\phi$ is zero. However, because of mismatch material the phase angle is not equal to zero. The phase angle can be calculated using Eqn. (4.8). After substituting $\phi = 0$, we have:

$$ \psi = \omega + \varepsilon \ln \left( \frac{L}{h} \right) $$

(4.14)

The calculated shift is in the range of 0 to 15 degrees which is small and so the specimen can be considered to be essentially in mode I with phase angle ($\psi$) equals zero.

### 4.3 Four-point Shear Specimen for Testing in Mixed Mode

Next, the four-point shear specimen shown in Figure 4.2 is considered. This specimen has been rigorously analyzed for mixed mode fracture testing [He et al. (1990)]. For four-point shear specimen, the apparent stress intensity factors related to the loads and specimen
geometry are given by

\[ K_I = \frac{M}{b^d^{3/2}} f_b\left(\frac{a}{d}\right) \]  
(4.15)

\[ K_{II} = \frac{Q}{b^d^{1/2}} f_s\left(\frac{a}{d}\right) \]  
(4.16)

where \( f_b \) and \( f_s \) are correction factors depending on the ratio \( a/d \) (shown in Table 4.2). \( M \) and \( Q \) and the applied moment and shear force at the crack location respectively, and \( b_1, b_2 \) and \( d \) are geometric quantities defined in Figure 4.2.

Table 4.2: Stress intensity coefficients

<table>
<thead>
<tr>
<th>( \frac{a}{d} )</th>
<th>( f_b\left(\frac{a}{d}\right) )</th>
<th>( f_s\left(\frac{a}{d}\right) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20</td>
<td>4.97</td>
<td>0.496</td>
</tr>
<tr>
<td>0.25</td>
<td>5.67</td>
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</tr>
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<td>6.45</td>
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<td>0.35</td>
<td>7.32</td>
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<td>8.35</td>
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<td>0.60</td>
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<tr>
<td>0.65</td>
<td>19.17</td>
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</tbody>
</table>

The interface stress intensity factors can be obtained from Eqn. (4.5) with the assumption \( \beta = 0 \):

\[ K_1 = \sqrt{1 - \alpha (K_I \cos \omega - K_{II} \sin \omega)}, \quad K_2 = \sqrt{1 - \alpha (K_{II} \cos \omega + K_I \sin \omega)} \]  
(4.12)

The corresponding energy release rate can be calculated as:

\[ G = \frac{K_I^2 + K_{II}^2}{E_1} \]  
(4.17)
Figure 4.2: Sandwiched four-point shear specimen.

To determine the loading angle \((\phi)\) for the four-point shear test, the following equation can be used: \(\phi = \tan^{-1} \left( \frac{f_s d f_b c}{f_s d f_b c} \right)\). However, the phase angle \((\psi)\) also depends on the material combinations. In this study, the value of \(\alpha\) is not a constant because \(\alpha\) depends on the Young’s Modulus of concrete \((E_c)\) which changes with the moisture content. As a result, \(\omega\) also changes with the moisture content. However, by considering all cases with different duration of moisture conditioning, it is noticed that the value of \(\alpha\) and \(\omega\) did not vary much and the mean shift of 11 degrees was taken into account.

### 4.4 Summary Comments

In this chapter, a background of sandwich beam specimen is described. The sandwich beam specimen is tailor-made for measuring the fracture toughness of bi-material. There are two main reasons why this specimen is adopted for the characterization of fracture property in this thesis. Firstly, the failure mode of sandwich beam specimen does not depend on size. Since size scale effect is not the focus in this thesis, sandwich beam specimen is chosen rather
than the other types of specimen, such as Brazilian disk specimen, in which the failure mode is related to the size of specimen. Secondly, the fabrication of sandwich beam specimens is easier compared to that of Brazilian disk specimen. Hence, a better quality control of the specimen duration the fabrication can be achieved. Meanwhile, by using the sandwich beam specimen, it is easy to change the loading angle by varying $b_1$ and $b_2$ as shown in Figure 4.2.

The result in this thesis will be useful in future when the relationship between fracture energy and phase angle becomes our concerns. There are two limitations in using sandwich beam specimen in this thesis. Firstly, the sandwich beam specimens cannot truly represent a real problem. It is because, in practice, concrete/epoxy system usually does not exist alone, but rather comes with a multi-layer system, such as the FRP-bonded concrete system which consists of concrete, epoxy and FRP. As a result, the failure mode of sandwich beam specimen can hardly be found in reality. Secondly, it is noticed that the loading angle is very sensitive to the value of $c$ (Figure 4.2). Such sensitivity will lead to the loss of accuracy in measuring the relationship between fracture energy and phase angle. Because of this limitation, investigation was only made for one phase angle under the mix mode situation.
Chapter 5

Diffusion Characterization

In order to evaluate the interface fracture toughness of a concrete/epoxy system and its variation under moisture ingress, mechanical properties of the constituent materials as a function of moisture content should be determined, as shown in the interface fracture formulations developed in Chapter 3. Here, only the diffusion properties of epoxy are discussed. The diffusion characterization of concrete can be referred to the thesis of C. Au [Au (2005)] because the same concrete mix was used in this study as well.

It is known that diffusion is a transient phenomenon which means that the diffusion property may not be a constant throughout the process. Here, I am using steady-state properties. In fact, when I measured the moisture content of epoxy, the time between successive data collecting points is long enough such that steady-state assumption can be made.

Materials being characterized in this research include a normal grade concrete and one type of epoxy adhesive system. These materials are made and cured according to the available ASTM standards and manufacturer stipulations as will be described in details below. The materials discussed in this chapter are identical to those used in the mechanical characterization, as well as the sandwich beam specimens, which are to be discussed in the following chapters.

5.1 Material Description

Normal strength concrete is used in this research which was composed of water, cement, sand and gravel with a mass ratio of 0.5:1.0:2.4:1.6. It is the same type of concrete described in
the thesis of C. Au [Au (2005)]. Aggregates were carefully sieved and remixed to comply with the ASTM C33 grading standard. Maximum gravel size was set at 9.5 mm in order to meet the requirement of ASTM C192 that the largest aggregate size should be at least three times smaller than the minimum specimen dimension. Meanwhile, all aggregates which passed through sieve No.100 were considered to be too fine and were discarded in order to maintain a good workability with a designed water-cement ratio. Mixing, casting and curing process were complied with the stipulation set forth in ASTM C192. It should be mentioned that a 28-day curing process was carried out.

One type of epoxy obtained from a local U.S manufacturer (MBT) was used in this study. The commercial name of this epoxy is “Concreseive Paste LPL”. It was a 2-component, 100% solids, moisture-tolerant structural epoxy paste adhesive. Component A is black in color, while component B is white in color. Mix ratio A:B was 2.841:1 by weight. Mixing was performed by a low-speed paddle for 10 minutes, until a uniform grey color was observed. Curing required 7 days in ambient environment (room condition).

5.2 Conditioning and Test Environments

All epoxy samples were divided into two constant temperature moisture conditioning groups at 23°C and 50°C. Except those specimens tested for initial dry properties, all samples were continuously conditioned in water bath in order to ensure 100% relative humidity environment. They were then taken out for weighing and testing in standard laboratory conditions at selected time intervals. All epoxy specimens were assumed to be dry initially because the mixing and curing process did not involve any water. It should be mentioned that de-ionized water was chosen for conditioning all specimens, including the specimens used in this chapter, as well as chapters 6 and 7 in this thesis, as a matter of convenience.
5.3 Moisture Sorption Study

Moisture sorption study was conducted on thin circular coupons of epoxy for determining the diffusion properties, in particular, the diffusion coefficients and moisture saturation values (in terms of percentage mass uptake). Ten coupons were tested under one constant temperature to provide averaging of the moisture sorption quantification. The aspect ratio (diameter to thickness) of these coupons was chosen such that it can resemble a one-dimensional diffusion behavior as discussed in Chapter 2. The diameter of the coupons was more than one order of magnitude larger than the thickness. As suggested by ASTM D570, the epoxy coupons were molded in circular shape with a thickness of 3.2mm and a diameter of 50mm.

All epoxy coupons were first weighted for their dry weights. Then, they were immersed in constant temperature water baths at 23°C and 50°C respectively in order to make sure that there was continuous moisture diffusion. The coupons were taken out for subsequent weighing at selected time intervals. It should be noted that the time intervals between consecutive weighing need not be uniform. In fact, the diffusion rate is fast at the beginning and it drops afterwards and eventually approaches zero when reached saturation stage. Hence, closer time interval at the beginning is required in order to capture the faster moisture uptake behavior. Therefore, measurement were made on Day 0, 0.5, 1, 2, 3, 5, 7, 14, 28, 56, 84, 140 and 280 for all moisture coupons. During the process of weighing, all coupons were dried with filter papers, then immediately weighted, and returned to the original water baths as soon as possible. All weighing were performed on an AccuLab Al-204 electronic balance which had a precision up to 0.0001g.

5.4 Moisture Sorption Behavior

Moisture sorption uptake curve for epoxy is shown in Figure 5.1, plotting percentage weight gain $M_t$ versus square root of time. In Figure 5.1, it is shown that the moisture coupons did not
saturate even after 280 days of continuous moisture conditioning with 100% relative humidity. Because of this, normalized plots could not be made accurately on these coupons with respect to the moisture saturation content $M_s$. The computation of diffusion coefficients, $D$, could be performed as outlined in Chapter 2. Table 5.1 summarizes the estimated diffusion coefficients, $D$, and saturated mass uptake, $M_s$ of the epoxy under two different temperatures.

![Mass Uptake Curves for Epoxy "Concreseive LPL"](image)

**Figure 5.1: Mass Uptake Curves for Epoxy “Concreseive LPL”**

<table>
<thead>
<tr>
<th>Table 5.1: Moisture diffusion properties of epoxy “Concreseive Pastes LPL”</th>
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<tr>
<td>Material</td>
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<tr>
<td>Epoxy</td>
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</table>

It is observed that higher conditioning temperature has generally led to faster moisture
diffusion, as indicated by the higher diffusion coefficients and moisture saturation values. Also, the time of saturation for both conditioning temperature did not demonstrate a saturation state even after 280 days of continuous submergence in water baths. Based on this observation, together with the fact of the linear increasing curve shape, it is suggested that this epoxy has a high potential of more moisture uptake.

5.5 Summary Comments

The eventual aim of this project is to improve the durability of FRP-bonded concrete system in real life problem. Hence, a large-scaled structural element should be studied rather than just focus on a small-scaled laboratory specimen. However, the purpose of this thesis serves as a basis for future numerical modeling which requires correct mechanical and diffusion properties of constituent materials in the FRP-bonded concrete system. In this chapter, the diffusion properties of epoxy “Concreseive LPL” are characterized by means of the moisture sorption curve. Together with the existing diffusion properties of concrete and FRP [Au (2005)], the moisture diffusion phenomenon of a large-scaled FRP-bond concrete system can then be simulated in finite element program, such as ABAQUS. With the moisture simulation of a real scale structural element, the corresponding structural behavior of a large-scaled structural element can be modeled as well with those parameters obtained in Chapters 6 and 7. Ultimately, the numerical model can be validated by an appropriate experiment using the materials in which the mechanical and diffusion properties have been characterized.
Mechanical tests were performed on concrete and epoxy to determine their properties before and after moisture conditioning. In order to estimate the interface fracture toughness using the formulations presented in Chapters 3 and 4, only the elastic moduli need to be determined. It is assumed that $\nu$ doesn’t vary with moisture content and adopt 0.22 for concrete. However, it is beneficial to extensively characterize their mechanical properties, which cover the tensile strength of epoxy, the compressive strength and the fracture toughness of concrete. The comprehensive investigation on these mechanical properties might increase the overall understanding of the moisture effect on individual material, as well as the interface between the concerned materials. Figure 6.1 illustrates the overall mechanical test program.

6.1 Concrete

Concrete is a traditional construction material and the effect of moisture on their mechanical properties has been investigated by a number of research groups [Bazant and Thonguthai (1978), Bazant and Prat (1988), Ross et al. (1996), Neville (1997), Konvalinka (2002), Au (2005)]. It has been observed that the mechanical properties of concrete degrade with increasing moisture content in general. In particular, mode I fracture toughness of concrete could reduce by as much as 60% after fully saturated with moisture and tested under water [Bazant and Prat (1988)]. The reduction in compressive strength of concrete could range from 30% to 40% on
720-day cured concrete, which depends on the level of moisture saturation [Konvalinka (2002)]. Based on my test result, such large variation in compressive strength of concrete has been recorded as well.

The explanation of the reduction in fracture toughness can be made from a viewpoint of internal pore water pressure [Bazant and Prat (1988)]. It is believed that the internal pore water pressure is developed in wet concrete under external loading due to the limited pore space. Migration of water in a pore is not allowed when adjacent pores are also filled with water. Due to the capillary actions, the hindered adsorbed water produces a very high disjoining pressure between the contacting cement pastes. This pressure is expected to be responsible for intensifying the stress intensities at the micro-crack tips and leads to earlier crack propagation under external loading. As a result, the fracture resistance of wet concrete becomes lower, as compared to that of dry concrete.

On the other hand, the degradation of mechanical properties in concrete under moisture can be approached from a viewpoint of volumetric change of the hardened cement pastes [Konvalinka (2002)]. It is believed that there is an increase in volume of hardened cement pastes under moisture conditioning. The average distance between the surfaces of the hardened cement gel becomes larger which leads to a decrease in secondary bonds between the surfaces. As a result, there is a decrease in strength for wet specimens.

In order to validate these hypothesis on a macroscopic scale and determine the mechanical properties of concrete with increasing moisture content, compressive test, mode I and mode II fracture tests have been performed in accordance to ASTM C39, Bazant's mode I concrete fracture test setup [Bazant and Prat (1988)] and Reinhardt's mode II concrete fracture test setup [Reinhardt and Xu (1998)].
Concrete cylinders with dimensions of 50mm (diameter) × 100mm (height) were tested under uniaxial compression. The maximum aggregate size is 10mm. All specimens were continuously moisture conditioned and taken out at selected time intervals for testing in standard laboratory condition. The selected time intervals are 0 (dry), 2, 4, 6, 8 and 10 weeks. There were three identical specimens for each moisture and temperature condition. The compressive tests were performed on a 60-kN Baldwin machine. Axial strains of the compressive cylinders were measured by means of a pair of clip-on extensometers mounted on opposite sides of the curved surface. The compressive tests were displacement controlled. To make sure that there is no free water inside the concrete specimen, all the specimens were dried in oven at 50°C for 3 days after curing. The initial compressive strength (f’c) and Young’s modulus (E_c) are 35.3MPa and 30.7GPa respectively.

Figures 6.2 and 6.3 plot respectively the variations of compressive strength and Young’s
Modulus with increasing moisture uptake. Trend lines are also given as a result of regression analysis. In general, it can be observed that degradation took place in both material properties. However, the effect of different conditioning temperatures does not seem to be significant. The strength reduction can be up to 40% as demonstrated in Figure 6.2. The Young's modulus of concrete, on the other hand, was not affected by much even after 10 weeks of 100 RH moisture conditioning. Also, the data scattering seems to be less substantial when compared with the case of compressive strength. The more substantial scattering of data found in compressive strength maybe be explained by the fact that redundant crack paths are available when concrete is subjected to compression, while the Young's modulus is measured at an infant stage without many crack formations. The results found in this part agree with the work from other researchers as mentioned previously.

Figure 6.2: Concrete compressive strength vs. moisture uptake
6.1.2 Mode I Fracture Test

Concrete fracture beam specimens measured 100mm x 37.5mm x 37.5mm with a through thickness center notch that was 6.25mm deep x 1.6mm wide. Span length of the fracture beam specimens was 93.75mm which was 2.5 times the beam thickness. All the mode I fracture specimens were continuously moisture conditioned and taken out at selected time intervals, which were the same as those of compressive test, for testing in standard laboratory conditions. Again, there were three identical specimens for each moisture and temperature condition.

Mode I fracture tests were performed on an Instron universal test machine Model 1331 (Figure 6.4). The tests were displacement controlled with a rate of 0.1mm/min. Figure 6.5 shows the experimental test setup of the three-point bending test of concrete beam specimen.

The mode I stress intensity factor can be obtained from the stress analysis of cracks handbook [Tada et al. (1985)] as:
\[ K_I = \sigma \sqrt{\pi a} F\left(\frac{a}{b}\right) \]  \hspace{1cm} (6.1)

where \( \sigma = 6M/b^2 \) (stress per unit thickness) in which \( M \) is the applied moment, \( a \) and \( b \) are the crack length and the height of specimen respectively as shown in Figure 6.6, and \( F(a/b) \) is a configuration correction factor which its relationship with \( a/b \) shown in Figure 6.7.

Figure 6.4: Instron universal test machine Model 1331

Figure 6.8 plots the variations of mode I fracture toughness with increasing moisture uptake. It can be observed that fracture toughness reduction ranged from 12% to 15%. Again, the effect of different conditioning temperatures seems to be insignificant. The less substantial scattering of data found in mode I fracture test maybe be explained by the fact that the crack propagation locus under mode I fractures is relatively clear while redundant crack paths are available when concrete is subjected to compression.
Figure 6.5: Three-point bending test setup

Figure 6.6: Schematic diagram of a three-point bending setup [Tada et al. (1985)]
Figure 6.7: Relationship between \( F \) and \( a/b \) [Tada et al. (1985)]

Figure 6.8: Concrete mode I fracture toughness vs. moisture uptake
6.1.3 Mode II Fracture Test

A specimen geometry and loading arrangement has been proposed and has already been applied to wood [Xu et al. (1996)]. It has also been proven that it is applicable to generate a shear fracture and to determine the mode II fracture toughness of concrete [Reinhardt et al. (1997)]. The specimen geometry is a rectangular or square plate with two notches at opposite side. Figure 6.9 shows the total depth $2h$, the ligament length $2a$, and the width $2w$. The thickness of the plate may vary. If depth and width were infinite, mode II stress intensity factor can be obtained from the stress analysis of cracks handbook [Tada et al. (1985)] as:

$$K_{II} = \frac{\sigma}{4} \sqrt{\pi a}$$  \hspace{1cm} (6.2)

For finite specimen size, it has been shown that Eqn. (6.2) applied if $h \geq 2a$ and $w \geq \pi a$. If $h \geq 2a$ and $w \leq \pi a$, the geometry can be regarded as an infinite strip and mode II stress intensity factor can be expressed as follows [Xu et al. (1996)]:

$$K_{II} = \frac{\sigma}{4} \sqrt{w}$$  \hspace{1cm} (6.3)

![Figure 6.9: Double-edged notch plate geometry](image)
The condition $h \geq 2a$ ensures the uniform distribution of stress on the loaded end of the strip. The loading should be the most appropriate to the concrete. Compressive force was chosen here because the compressive strength of concrete is high and shear fracture is possible before concrete crushing.

The dimensions of the concrete mode II fracture specimen are $200\text{mm (width, } 2w) \times 200\text{mm (height, } 2h) \times 100\text{mm (thickness)}$ with a notch depth $50\text{mm}$. With such dimensions, Eqn. (6.3) can be applied rigorously for the evaluation of mode II fracture toughness of concrete. All specimens were continuously moisture conditioned and taken out at selected time intervals, which were the same as those of compressive tests and mode I fracture tests, for testing in standard laboratory conditions.

The tests of the double-edge notched plate specimens were carried out in the open-loop hydraulic 200-kip Baldwin machine (Figure 6.10). Steel plates with smooth surfaces were put under and on top of one half of the specimens. The whole arrangement consisting steel plates and specimen was positioned very carefully between the loading platens of the testing machine in order to avoid eccentricity. Figure 6.11 shows the experimental testing arrangement. The load was applied with a constant cross-head displacement rate of $0.3\text{mm/min}$.

The specimens were carefully examined, especially near the notch. At a certain loading, a shear crack developed at the tip of the notch. Sometimes the crack propagated along the ligament; sometimes inclined cracks formed in the loaded part of the specimen. Based on the experimental result, there was not a great difference (about 30%) between the load which caused shear cracking and load which caused compression failure. Figure 6.12 shows an example in which shear crack started at the notch tip and merged into distributed cracks in the loaded part.

There were three specimens measuring the mode II fracture toughness for each moisture and temperature condition. Figure 6.13 shows a plot of load against displacement between
loading plate when the specimen was dry. Figure 6.14 shows a plot of load against displacement between loading plate when the specimen was moisture-conditioned for 10 weeks. Here, it should be mentioned that the displacement in Figure 6.13 measured the position of the loading head. In both Figures 6.13 and 6.14, it is noticed that there is a pronounced discontinuity before the peak load comes. It is due to the crack initiation. After releasing some load, the load increases again up to a maximum value which causes final compression fracture in the loaded part. All specimens contained the point of discontinuity in the corresponding load-displacement curve were used for the computation of the critical stress and hence the mode II fracture toughness of concrete $K_{IIc}$.

The mode II fracture toughness of concrete was plotted against the moisture content in Figure 6.15. Mode II fracture toughness was approximately 27% and 48% for 23°C and 50°C temperature groups respectively. It is observed that there is a similar reduction trend in the mode II fracture toughness as well when compared to mode I fracture toughness. The less
substantial scattering of data found in mode II fracture test maybe be explained by the fact that the crack propagation locus under mode II fractures are quite clear, while there are redundant crack paths when concrete is subjected to compression.

Figure 6.11: Test setup for mode II fracture toughness of concrete

Figure 6.12: Crack pattern of double-edged notch specimen
Figure 6.13: Load vs. Displacement of specimen under 23°C and dry conditioning

Figure 6.14: Load vs. Displacement of specimen under 23°C and 10 week moisture conditioning
6.2 Epoxy

Tensile properties of epoxy were characterized using Type I dumbbell shaped samples in accordance to ASTM D638 with an Instron universal test machine Model 1331 (Figure 6.4) mounted with a 50kN load cell and a pair of self-tightening grips. Crosshead speed was set at 1mm/min. A clip-on Instron extensometer with 50mm gage length was used to measure the strain within the specimen. The initial properties (dry) for epoxy “Concreptive Paste LPL” were determined and summarized in Table 6.1.

<table>
<thead>
<tr>
<th>Tensile Strength (MPa)</th>
<th>13.2</th>
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<tbody>
<tr>
<td>Young’s Modulus (GPa)</td>
<td>3.6</td>
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</table>
The test data given by the manufacturer are 13.8MPa for tensile strength and 2.8GPa for Young's Modulus. The agreement of these two sets of data confirmed the accuracy of the measured initial material properties of epoxy that was used in this project. Figures 6.16 and 6.17 show the variations of tensile strength and Young's modulus with increasing moisture uptake respectively. There is a significant increase in the tensile strength after 2 weeks of moisture conditioning under room temperature. It may be due to the fact that additional curing of the epoxy occurred. After the increase of the first 2 weeks, the tensile strength generally degraded with increasing moisture uptake under 23°C. However, the degradation of tensile strength under 50°C was very minimal. It is expected that the moisture degradation effect was eventually offset by the high temperature post-curing effect. The Young's modulus degraded significantly with increasing moisture uptake under 23°C. However, under 50°C, the degradation of the Young's modulus was even more serious. It is probably because the conditioning temperature is very close to the glass transition temperature ($T_g$) of the epoxy. Hence, the stiffness of the epoxy decreased nearly by an order when compared to the initial tensile properties of epoxy under 23°C.
Figure 6.16: Epoxy tensile strength vs. moisture uptake

Figure 6.17: Epoxy Young's modulus vs. moisture uptake
6.3 Summary Comments

The important mechanical properties of concrete and epoxy were characterized in this chapter. Most of the findings agree with the existing data. It is primarily found that mode I and mode II fracture toughness of concrete decrease at a similar rate under moisture conditioning. For the compressive strength of concrete, the experimental result validated again the fluctuation of concrete compressive strength when subjected to moisture ingress. Regarding the epoxy, it is intuitively surprising that Young's modulus reduces while tensile strength does not change much. It can be explained in terms of molecular level. Epoxy is a thermosetting plastics which consist a series of polymer chains and connected with each order through cross-linking (secondary bonds). The Young's Modulus of epoxy is related to the secondary bonding between polymer chains, which does not related to the strong covalent bonding within the polymer chains. However, tensile strength relates to the breaking of the polymer chain which implies the breaking of the strong covalent bonds within the polymer chains. It can be induced that moisture and high temperature degrade the secondary bonding in the epoxy, while the strong covalent bonds within the polymer chains are inert to moisture and temperature variation. It is observed that the mechanical properties of concrete and epoxy both deteriorate with increasing moisture content and temperature. The degradation is significant in certain mechanical properties, such as the concrete compressive strength. However, it will be seen that the deterioration of the concrete/epoxy bonded system is much more severe when subjected to moisture and temperature effect. Hence, such findings further consolidate the importance of this thesis in a sense that durability of individual material can not be used to predict the durability of a bonded system.
Chapter 7

Measurement of Fracture Toughness of Concrete/Epoxy Interface

This chapter gives the procedures for the calculation of fracture toughness of concrete/epoxy interface. It summarizes the test data from tests on mode I and mixed mode sandwiched beam specimens.

In the first part, test results are presented for sandwiched beam specimen under mode I loading. The data obtained in this part are the most variable because the shift of failure mode under moisture from concrete delamination to interface separate, which leads to premature failure of concrete/epoxy system, was observed here. Sandwiched beam specimens under mixed mode loading are presented in the second part. The result from both mode I and mixed mode fracture toughness measured the strength of the interface under peel and shear forces. The result presented here can be compared with the result from peel and shear tests on FRP-bonded concrete [Au (2005)]

7.1 Interface Fracture Parameters

In order to calculate the interface fracture toughness at the interface, we need the material properties of individual material. First of all, we need the Young’s modulus of concrete and epoxy, as well as their Poisson’s ratio ($\nu$), in order to calculate the Dundurs parameters $\alpha$ and $\beta$. 
Here, we assumed $\nu$ to be 0.22 for concrete and 0.35 for epoxy. It should be noticed that $\alpha$ and $\beta$ are not constants because Young’s modulus of both concrete and epoxy degrade with increasing moisture content. It is worth to mention that the value of $\beta$ was still about 0.1 throughout the concerned moisture duration. It implies that the assumption which considers $\beta = 0$ is still valid. After calculating the value of $\alpha$ and $\beta$, the procedure shown in Chapter 4 can be used for evaluating the fracture toughness of concrete/epoxy interface under different type of loadings.

7.2 Details of Sandwiched Beam Specimens

To generate the fracture toughness for both mode I and mixed mode conditions, the two types of sandwich beam specimens presented in Chapter 4 were tested. In order to fabricate the sandwiched beam specimen, two concrete blocks were casted. They have the same cross sectional dimension with 76.2mm (height) x 38.1mm (thickness). However, their lengths were different. One was 152.4mm long; while the other was 228.6mm long. These concrete blocks were all properly cured and then dried in order to ensure the concrete face, which would be stuck with epoxy, was dry when it was bonded with epoxy.

The cross sectional dimensions of both sandwiched specimen were 76.2mm (height) x 38.1mm (thickness). The thickness of the epoxy layer, $h$, was 2.54 mm for both specimens. The thickness of this layer is chosen at least one order of magnitude smaller than the dimension of the specimen such that linear elastic fracture mechanics can be applied and the energy release rate of the specimen can be evaluated based on the Young’s modulus of concrete only. The relative crack size $(a/d)$ was 0.5 for all the specimens. In this study, $l = 228.6$ mm and $s = 114.3$ mm (see Figure 4.1); while $b_1 = 139.7$ mm in, $b_2 = 88.9$ mm, $c = 5$ mm (see Figure 4.2). Concrete with an average 28-day compressive strength of 37.9 MPa and one type of epoxy were used. The epoxy used here is “Concreasive Paste LPL”. To make sure a sharp precrack, a
notch plate made of thin plastic with the thickness of 0.1 mm was attached to one side of the epoxy layer.

The nomenclature for each of the sandwiched beam specimens is in the form of its variables (written in capital letters) and their respective content (designated as $x$), i.e. $M_x-P_x-T_x$. The meaning denoted by each symbol is explained as follows:

- $M_x$ represents the time duration of moisture conditioning in terms of number of weeks. For instance, $M_4$ means the time duration of moisture conditioning is 4 weeks.
- $P_x$ represents the phase angle of the tested specimen in terms of degree. For example, $P_60$ means the phase angle is 60°.
- $T_x$ represents the temperature during moisture conditioning in terms of Celsius. For example, $T_{23}$ means the temperature during moisture conditioning is 23°C.

### 7.3 Moisture Conditioning of Specimens

All specimens were cured for 28 days before conditioning. After 28 days curing, all the specimens were dried for 30 days in order to ensure that the moisture content of all specimens was 0% before any conditioning. After these preparations, all the specimens were treated under three different moisture durations, namely, 0-week, 2-week and 4 week; and two temperatures, namely, 23°C and 50°C. For each moisture and temperature condition, three specimens were tested. Dundur's parameters $\alpha$ and $\beta$, oscillation index $\varepsilon$, and shift angle in a sandwich specimen $\omega$ for the biomaterial combination are then computed based on the material properties corresponding to certain moisture level. Here, the Poisson's ratios of concrete and epoxy were taken as 0.22 and 0.35 respectively for the calculation process with an assumption that they are independent of moisture effect. It is noticed that $\beta$ is quite small in which it is smaller than 0.2 in all cases. Hence, in order to simplify the calculation, $\beta$ is taken as zero and hence all the equations stated in Chapter 4 can be used.
7.4 Test Setup

Four-point bending and shear tests on the sandwiched beam specimens were performed using an Instron Model 1331 machine with a 50kN load cell as shown in Figures 7.1 and 7.2. The point load deflection was measured by an extensometer which was mounted on a reference frame in order to prevent measuring inelastic deformation of the specimen. All the tests were carried out under displacement control with a rate of 0.00167 mm/min. The phase angle \((\psi)\) of the bending specimen was treated 0\(^{\circ}\), while the phase angle \((\psi)\) of the shear specimen was adjusted to be 60\(^{\circ}\) in order to achieve the mixed-mode loading condition.

The tests were stopped once the load dropped beyond 50\% of the peak load. Using the loading procedure outlined above, the peak load was reached within 10 minutes of the starting of the test. Finally, it must be mentioned that during the tests, load and point load deflection were both continuously recorded.
7.5 Results and Discussions

7.5.1 Mode I Fracture Toughness of Tested Specimens

Typical failure modes of the bending beam specimens with and without moisture conditioning are shown in Figure 7.3 and 7.4. Figure 7.4 shows the failure mode of the dry sandwiched bending specimen at 23°C. Figure 7.5 shows the failure mode of the wet sandwiched bending specimen with 4-week moisture duration at 50°C. Dry bending beam specimens exhibited failure in concrete itself, rendering a cohesive type of failure. The pre-crack, which was placed at the concrete/epoxy interface, kinked into the concrete upon reaching the peak load. The specimen failed by concrete delamination. This phenomenon was observed for both temperature groups. Failure surface felt powdery to the touch and small grains of sand could clearly been seen.
Figure 7.3: Concrete delamination of bending beam specimen under dry condition

Figure 7.4: Interface separation of bending beam specimen after 4-week moisture conditioning
Wet bending beam specimens, on the other hand, exhibited the distinctive concrete/epoxy interface separation. For the specimens under moisture conditioning of 2 weeks and 4 weeks, a relatively small amount of loose concrete particles adhered to the epoxy layer. However, for the specimen under moisture conditioning over 4 weeks, a clear separation between concrete and epoxy was observed. It was surprising to see that the crack did not kink into concrete substrate at all during the whole testing process but remained propagating along the moist concrete/epoxy interface. It is observed that the shift of fracture failure mode from concrete delamination to interface separation is accompanied by the substantial decrease of the mode I interface fracture toughness. Table 7.1 presents the results obtained from the four-point bending tests conducted in this study.

The load versus load-line displacement curve for sandwiched bending specimens was very brittle. Little microcracking was detected, and cracks advanced very rapidly. The variation of mode I fracture toughness when subjected to different moisture durations and temperatures are shown in Figure 7.5. The fracture toughness at each condition, which is shown on these graphs, was the average among three tested specimens. It is observed that there was a decrease in the interface fracture toughness and an asymptotic behavior can be achieved with increasing moisture ingress. In particular, there was a substantial decrease in fracture energy release rate with a complex phenomenon which involves a distinctive dry-to-wet debonding mode shift from material decohesion (concrete delamination) to interface separation. Such a great deterioration occurred after 4-week moisture conditioning at 23°C while it occurred after 2-week moisture condition at 50°C. It reveals that the combined effect of moisture and high temperature produce a more severe deterioration.

The experimental results agree with those from peel test of FRP-bonded concrete specimen [Au (2005)]. Although the numerical values of the interface fracture toughness are not the same, the decreasing trend of the interface fracture toughness with an asymptotic behavior can be observed in both studies. The results in this part further ensure that the premature failure of
concrete/epoxy occurs with increasing moisture ingress, which is illustrated by the shift of failure mode from material decohesion to clear interface separation.

Figure 7.5: Mode I interface fracture toughness variation
Table 7.1: Mode I interface fracture toughness

<table>
<thead>
<tr>
<th>Unit</th>
<th>$K_I$ (MPa·m$^{1/2}$)</th>
<th>$J_{mode I}$ (J/m$^2$)</th>
<th>Debonding Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>M0-P0-T23</td>
<td>0.725</td>
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<td>Concrete Delamination</td>
</tr>
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<td>0.692</td>
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<td>0.699</td>
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</tr>
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</tr>
<tr>
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<td>Concrete Delamination</td>
</tr>
<tr>
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<td>0.616</td>
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</tr>
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<td>0.346</td>
<td>3.93</td>
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7.5.2 Mixed Mode Fracture Toughness of Tested Specimens

Typical failure mode of the sandwiched shear beam specimens is shown in Figure 7.6. Unlike the sandwiched bending beam specimen, the failure did not change from dry to wet condition. The failure of sandwiched shear specimen was very brittle. Little microcracking was detected, and cracks advanced very rapidly. The variation of mixed mode fracture toughness when subjected to different moisture durations and temperatures are shown in Figure 7.7. The fracture toughness at each condition, which is shown on the graph, was the average among three tested specimens. It is observed that there was a decreasing trend in mixed mode fracture toughness with increasing moisture duration, however, with a slower rate when compared with the variation of mode I interface fracture toughness.

![Figure 7.6: Typical failure mode of sandwiched shear specimen](image)

According to previous studies on shear fracture specimen of FRP-bonded concrete, the mode II interface fracture toughness decreases gradually and the shear fracture degradation can be maximized after 8 weeks moisture conditioning [Au (2005)]. Here, mixed mode interface...
fracture toughness is a combination of mode I and mode II fracture toughness and hence by combining the variations of mode I and mode II fracture toughness with moisture content, which can be obtained from peel and shear specimens, it is expected that the mixed mode interface fracture toughness will decrease in a more gentle way when compared with the mode I interface fracture toughness.

Figure 7.7: Mixed mode interface fracture toughness variation
Table 7.2: Mixed mode interface fracture toughness

<table>
<thead>
<tr>
<th>Unit</th>
<th>$K_I$ (MPa √m)</th>
<th>$K_{II}$ (MPa √m)</th>
<th>$\Gamma_{\text{mixed mode}}$ (J/m$^2$)</th>
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7.6 Problems in the Measurement of Interface Fracture Parameters

It is difficult to obtain reliable data for interface fracture. The results depend on a large number of parameters. Some of them, such as the compressive strength of concrete, show scattered results. Since there is still no generally accepted test for interface fracture properties, researchers will have different results because of different specimens and test procedures.

In the four-point shear test, it was observed that a small change in $S_o$ would result in a large variation in the phase angle. Hence, in order to minimize the scale error, only one value of $S_o$ was adopted in the four-point shear test.

Also, the calculation of the interface fracture toughness are based on the linear elastic fracture, as well as the assumption of $\beta = 0$. Errors were introduced once these assumptions were made.

7.7 Significance of the Result

The experimental results imply that premature failure will occur in concrete/epoxy system when increasing moisture ingress. In reality, such system can be easily found in FRP-bonded concrete structures, which are usually related to the strengthening and retrofitting of existing reinforced concrete structures including buildings and bridges. It has been recognized that debonding could significantly affect the capacity of the retrofitted system to a great extent and is considered as an undesirable premature failure mode [Kaiser (1989), Gunes (2004)]. The detrimental effect of moisture found in this study were inline with other limited numbers of independent studies conducted on FRP retrofitted concrete beams [Chajes et al. (1995), Toutanji and Gomez (1997), Mukhopadhyaya et al. (1998), Karbhari et al. (1998), Au and Büyüköztürk (2006)]. In particular, focus was made at concrete/epoxy interface which is expected to be the weakest region in FRP-bonded concrete system when subjected to moisture ingress [Au and Büyüköztürk (2006)]. In this study, the fracture properties of concrete/epoxy interface were
quantified in relation to moisture uptake, which has not been done by previous researchers. Such findings are especially important in the sense that the computational modeling of the FRP-bonded concrete system becomes feasible with the interface fracture properties found in this study.

In order to prevent such premature failure, the moisture content at the concrete/epoxy interface should be controlled. Water proof material can be sprayed on the FRP surface in order to reduce the moisture diffusion in the FRP-bonded concrete system. Any observable separations at the concrete/epoxy interface may indicate the deterioration of the whole system.
Chapter 8

Discussion of Interface Fracture

In the prior chapters, testing on mechanical properties of concrete and epoxy has been conducted in order to compute the interface fracture toughness of sandwich beam specimens which are subjected to accelerated moisture conditioning for selected time intervals. As discussed before, the result in this thesis will be used as a basis for the future numerical simulation of a large-scaled FRP-bonded concrete system for the investigation of the structural behavior under moisture effect. As inputs in the computation, diffusion and mechanical properties of the constituent materials with respect to moisture uptake are physically determined through the experiments described in this thesis. With the knowledge of the computed toughness values and their trends with respect to increasing moisture ingress, observed debonding modes and their shift behavior from dry to wet, as well as the simulated diffusion behavior and quantification, this chapter analyzes the debonding problem synergistically by combining and correlating with kink criterion implementation, to develop better understanding of various mechanistic debonding behavior affected by moisture. Discussions are made mainly from an interface fracture mechanics perspective.

8.1 Effect on Moisture on Debonding Resistance

Upon closer examination of the tested specimens, two facts were found. The first fact is that in all mode I fracture cases, regardless moisture conditioning duration beyond 4 weeks, concrete/epoxy interface separation was observed consistently as the debonding mode in the
region where the first crack advance took place. The second fact is that the moisture concentration on the crack face contained water, as shown in Figure 7.4 with a dark color at the crack face of the concrete block. When the moisture concentration was low which can be observed in Figure 7.3 that a normal color of concrete was present at the crack face, a different debonding mode was likely to occur. For instance, when it was dry (i.e. 0%), concrete delamination initiated instead. Combination of these facts has led to the thinking of a possible threshold of moisture concentration beyond which no significant further change in mechanism and hence fracture toughness could occur. This argument, if true, should be supported by further evidence in the case of shear fracture.

In the case of mode I fracture, the shift in debonding mode from concrete delamination to concrete/epoxy interface separation has revealed the fact that when dry, the concrete/epoxy interface fracture toughness was higher than that for concrete delamination, which was approximately 15 J/m², although it would be technically difficult to measure the dry interface toughness values. Nevertheless, that value has dropped to close to 4 J/m² after moisture conditioning for 4 weeks or more and thus signified a substantial loss of adhesion between epoxy and concrete as a consequence of moisture ingress, provided that the original epoxy and concrete still remain in the interfacial region without formation of any new interphase material (more discussion on this later using crack kinking arguments).

Mixed mode fracture toughness of all wet specimens showed some degradation over time between the 2-week and 10-week test periods. In particular, high temperature conditioning group has started to degrade severely after 4 weeks. The fracture toughness has degraded by approximately half after 8 weeks under 23°C. In the mixed mode fracture case, however, the duration for such asymptotic behavior was much longer because the entire bond contributed to the debonding resistance, as opposed to the few centimeters near the edge region in mode I fracture.
8.2 Effect on Moisture on Locus of Fracture

The locus of fracture for all dry cases stayed within the material layers. The parent pre-crack at the concrete/epoxy interface that was generated by a thin Teflon film kinked into the material layers either above or below that interface. For the epoxy that used in this thesis, the parent crack kinked downward into the concrete substrate below the concrete/epoxy interface and subsequent crack propagation stayed close and parallel to that interface, leading to a thin layer of concrete adhered to the debonded strip. The locus of fracture for all wet cases, on the other hand, occurred apparently at the concrete/epoxy interface, staying at the same level as the pre-crack. In other words, no kinking was observed.

To study the shift in crack kinking behavior, the kink criterion (He and Hutchinson 1988) is employed. The kink criterion states that when

\[ \frac{\Gamma_i}{\Gamma_s} > \frac{G_i}{G'_\text{max}} \]  

is satisfied, the parent crack that lies at the interface of two adjoining materials will tend to kink into the substrate in consideration. Here, \( \Gamma_i \) is the interface fracture toughness, \( \Gamma_s \), the substrate fracture toughness in mode I, \( G_i \), the interface fracture energy release rate (of the parent crack), and \( G'_\text{max} \), the maximum fracture energy release rate for the kink crack at any putative kink angle. One should note that the ratio is less than unity because \( G_i \) is always less than \( G'_\text{max} \), due to the definition of \( G_i \).

When dry, the epoxy exhibited a concrete delamination mode of debonding and the corresponding mode I fracture toughness was roughly 14J/m². Since debonding did not occur at the interface, the actual toughness values of the concrete/epoxy interface were higher than those values. Taking the Mode I fracture toughness values of concrete (oven-dried) as presented in Chapter 6, it is noticed that \( \Gamma_i/\Gamma_s \) is greater than 1. Recalling the fact that \( G_i/G'_\text{max} \), has a value less than unity, it is clearly noted the expression (8.1) was satisfied. The kink
criterion thus predicts that the parent pre-crack would kink into the concrete substrate. This prediction is inline with the observed failure mode where concrete delamination indeed took place in both loading configurations.

Although the kink criterion is capable of predicting the initial crack propagation tendency, it fails to provide further insight regarding the subsequent crack front behavior. In other words, the kink criterion explains why the pre-crack kinked into the concrete substrate, but it does not indicate why the crack continued to propagate close and parallel to the concrete/epoxy interface, but not continue to penetrate deeper into the supposedly brittle concrete block, resulting in a thin layer of concrete adhered to the debonded strip.

When wet, all mode I fracture specimens exhibited a consistent interface separation mode at the epoxy/concrete interface when the moisture duration is equal to or longer than 4 weeks. The corresponding mode I fracture toughness was roughly 4J/m². Since debonding occurred at the interface, the measured values were the actual toughness values of the concrete/epoxy interface. Taking the Mode I fracture toughness values of concrete (oven-dried) as presented in Chapter 6, it is noticed that $\frac{I/I_c}$ is smaller than 1 which implies the expression (8.1) was not satisfied. The kink criterion thus predicts that the parent pre-crack would not kink into the concrete substrate. Since the epoxy materials had a Mode I fracture toughness in the very high range, based on the existing lab data, crack kinking was unlikely to occur into the epoxy layer. As a result, the crack should propagate along the interface as it could not kink into either of the materials. This prediction is inline with the observed failure mode where a clear interface separation took place.

However, in all mixed mode fracture specimens, the standard criterion for crack kinking out of an interface cannot be used to predict the kinking angle. No matter how the interfacial fracture toughness reduced in the concrete/epoxy bi-material, the kinking direction was always to be approximately perpendicular to the direction of maximum tensile stress in the sandwich beam specimens. It is observed that kinking always ended at the bearing of the main load, hence the
kinking angle is determined by the loading geometry. Further studies on kink criterion under mixed mode condition should be carried. It is suspected that the kinking angle should be governed by both the kinking criterion and the loading geometry.
Chapter 9

Summary, Conclusions and Future Work

9.1 Summary

A fundamental study on the interface fracture of concrete/epoxy system under the effect of moisture was performed. Before the investigation of the fracture toughness at the concrete/epoxy epoxy, diffusion and mechanical characterization of constituent materials were carried out. Diffusion properties of the epoxy used were found through moisture sorption study. It is determined that the elevated temperature will increase the rate of moisture diffusion, which is represented by the moisture diffusion. The diffusion coefficient and saturation moisture content of the epoxy used, together with the diffusion properties of concrete which have already been found by C.Au [Au (2005)], will be used in future three dimensional moisture diffusion simulation of defected FRP-bonded concrete system using the finite element program, ABAQUS. Both concrete and epoxy are also tested for their mechanical property under the influence of accelerated moisture ingress. Acceleration is facilitated through continuous conditioning under water bath at elevated temperature. Mechanical tests on moisture conditioned material samples reveal that moisture is generally detrimental to their strength, stiffness and fracture toughness. In the case of concrete, both mode I and mixed mode fracture toughness are significantly reduced with a strong statistical correlation found in the experiments.

Equipped with the all necessary properties of the constituent materials, an analysis based on the energy release rate concepts was used to compute the interface fracture parameters from
tests on sandwiched beam specimens. The beam specimens consisted of a layer of epoxy embedded in concrete and were tested using either a symmetrical four-point bending setup for mode I fracture or an asymmetric four-point shear setup for mixed mode fracture. These tests were performed using two different loading angles, a measure of the contribution between mode I and mode II, at about $0^\circ$ and $60^\circ$. Results showed that interface fracture toughness tends to decrease and an asymptotic behavior can be achieved with increasing moisture ingress. The asymptote is formed between 2-4 weeks for sandwiched bending beam specimen while that for sandwiched shear specimen appears to be more gradual and does not show any asymptotic signs even after 10 weeks of moisture conditioning. The substantial decrease of mode I interface fracture toughness is accompanied by the shift of fracture failure mode from concrete delamination to interface separation.

Very often, the moisture effect on concrete and epoxy can be understood as the degradation of mechanical properties, such as tensile and compressive strength, Poisson's ratio, Young's modulus and fracture toughness, with a reduction amount up to 40% at most. However, the moisture effect on the interface property may be much more severe in a sense that the fracture toughness can be reduced by more than 70% for prolong moisture. The high uncertainty of the bond deterioration implies that the moisture effect on the interface property is very complicated. Generalization on the moisture effect of the interface property is difficult to be made because it depends on the type of epoxy to a certain extent. In some cases, interface toughening can be resulted instead of degradation [Au (2005)]. In order to study this problem in a more comprehensive way, numerical simulation using the concept of molecule dynamic is necessary so that we can have a better understanding on the interactions between water molecules and the interface in a micro-scale point of view.
9.2 Conclusions

The following conclusions are deduced from the experimental results:

- Moisture is generally detrimental to the mechanical properties of concrete including the compressive strength, Young's Modulus, mode I fracture toughness and mix II fracture toughness.
- Moisture is also generally detrimental to the mechanical properties of epoxy. However, the effect of moisture on the tensile strength of epoxy may be insignificant when the conditioning temperature is close to the glass transition temperature of the epoxy.
- Deterioration of the concrete/epoxy interface under moisture is observed through investigation of the fracture toughness of such interface.
- Mode-I fracture toughness decreases substantially when involves a distinctive dry-to-wet debonding mode shift from material decohesion (concrete delamination) to interface separation.
- Mixed mode fracture toughness decrease gradually with increasing moisture duration.
- The combined effect of high temperature and prolonged moisture duration severely deteriorate the concrete/epoxy interface.

9.3 Recommendations for Future Work

The future work should involve the following studies:

- Develop an appropriate composite beam model of FRP-bonded concrete system and study the influence of the change of interface properties due to the effect of moisture on the global structural behavior.
- Investigate the real FRP-bonded concrete system with altered interface characteristics through moisture ingress, correlate the result with those from the composite beam model study, and make recommendations for the design of FRP-bonded concrete system.
Simulate the moisture diffusion in concrete/epoxy system in a micro-scale by means of the concept of molecular dynamics (MD).

Perform the numerical simulation of the crack propagation along the interface with the consideration of kink criteria.

Study the size effect in the concrete/epoxy interface fracture by sandwiched beam specimens.

Investigate the effect of initial defects at the concrete/epoxy interface on the FRP-bonded concrete beam, quantify the defect criticality through the view point of interface fracture concept.
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