ENVIRONMENT-ASSISTED SUBCRITICAL CRACK GROWTH IN CFRP EXTERNALLY BONDED CONCRETE SYSTEMS

by

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ABSTRACT

The nation’s current state of infrastructure is at an increasing rate of disrepair with new methods of cost-effective rehabilitation being considered over timely and costly reconstruction. Externally bonding carbon fiber-reinforced polymer plates to the tension side of a flexural beam has been one such method that has proven effective and reliable due to the outstanding performance and various advantages in CFRP materials. Gaps in long-term durability studies have been the major reason why this strengthening technique has not seen more success. It is fundamental and crucial that the mechanism of the bond interface between concrete and the strengthening material be thoroughly tested and understood. It is the intention of this study to further increase the knowledge and understanding of the long-term durability of the concrete/adhesive interface while being exposed to various aggressive environments in such externally bonded systems. Firstly, a systematical and comprehensive literature review regarding the use of CFRPs as external reinforcement, the effects of various aggressive environments on the materials being tested, and the application of linear elastic fracture mechanics is presented. An experimental study is conducted using the Boeing wedge test with a digital image correlation technique used to analyze the subcritical crack growth along the concrete/adhesive interface. Results show that subcritical cracking does exist along this interface and its effects can be significant. A theoretical study is conducted to predict the long term debonding behavior by deriving an analytical model based on Euler-Bernoulli beam theory and
linear elastic fracture mechanics representing the Boeing wedge test specimen as a double-
cantilever beam extending off of a Winkler elastic foundation. A finite element analysis of the
test set-up is conducted as verification of the analytical model. Results of this study can and
should be included in the long-term durability section to be used as an indicator of maintenance
in all future infrastructure rehabilitation plans made at the federal, regional, and state levels.
LIST OF ABBREVIATIONS AND SYMBOLS

a  Effective Crack Length
b  Specimen Width
D  Flexural Rigidity of Composite Beam (CFRP+epoxy)
da/dt Crack Growth/Debonding Rate
k  Modulus of Winkler Elastic Foundation
L  Total Specimen Length
l  FRP Cantilever Length
M  Moment
P  Applied Peel Load
p  Distributed Vertical Load
q  Vertical Stress of Winkler Elastic Foundation
w  Vertical Deflection, where I or II denotes the occurring section
V  Shear Force
\( \Delta \)  Max Deflection of Section I
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CHAPTER 1: INTRODUCTION

1.1 Statement of the Problem

In 2009, of the 603,307 miles of road bridges in the United States (about half of which are concrete bridges), 11.8% were judged as structurally deficient and 13.0% were additionally deemed functionally obsolete [90]. A structurally deficient bridge is one whose components may have deteriorated or have been damaged, resulting in restrictions on its use [63]. Deterioration is caused by age, excessive loading, corrosion of internal reinforcement, exposure to and interaction with the surrounding environment, lack of maintenance, and/or poor initial design [90, 110]. A functionally obsolete bridge is one that still fully functions but was built to standards that are not used today. Functionally obsolete bridges can involve inadequate lane widths, shoulder widths, or vertical clearances to serve current traffic demand, or those that may be occasionally flooded. U.S. bridges scored a rating of $C$ (i.e. mediocre) on the 2009 report card for America’s infrastructure, while the 4.04 million miles of public roadways received a rating of $D$- (i.e. very poor). Overall, America’s infrastructure received a $D$ grading (i.e. poor) with an estimated 5 year investment of $2.2$ trillion required to boost the rating to a more acceptable level [7]. Due to the high percentage of unsatisfactory rated infrastructures and the great cost estimated to bring these infrastructures to a satisfactory level, methods of rehabilitation can be considered beneficial over reconstruction. ASCE issued out “Five Key Solutions” for
raising the grade on the infrastructure report card [7]. These include

1. Increase Federal Leadership in Infrastructure
2. Promote Sustainability and Resilience
3. Develop Federal, Regional, and State Infrastructure Plans
4. Address Life-Cycle Costs and Ongoing Maintenance
5. Increase and Improve Infrastructure Investment from all Stakeholders.

With these five key solutions in mind, I propose the following work to be used in concrete infrastructure rehabilitation.

In an effort towards concrete infrastructure rehabilitation this author proposes to use externally bonded carbon fiber reinforced polymer (CFRP) plates with an epoxy structural adhesive to study the subcritical crack growth induced by both mechanical and environmental forces. The subcritical crack growth will be tested using a Boeing (static) wedge test in a series of environments: ambient temperature (73°F), ambient tap water (73°F), a de-icing salt solution (4% NaCl), an alkaline solution (pH = 13), and elevated temperature tap water at 90°F and 110°F (32°C and 43°C). The peeling force causing debonding along the concrete/adhesive interface is theoretically modeled using linear elastic fracture mechanics and verified by a finite element analysis conducted using commercially available software ANSYS. Longterm debonding models can be used as an indicator when the structure is leading towards a catastrophic event, this indication can be interpreted as an indication of when maintenance is required. Using the results found, this method should be included in future infrastructure rehabilitation plans made at the federal, regional, and state levels.
1.2 Literature Review

The use of FRPs as reinforcement has progressed a long way in recent years. In the 1960s, civil engineers came across a problem of accelerated corrosion from using road salts in colder climates and marine salts in coastal areas. The solution the engineers came up with was to use a galvanized coating on the reinforcing bars and epoxy-coated rebar [1]. When researchers first started experimenting with polymer-impregnated concrete, they ran into compatibility problems with different thermal properties between the polymer and the steel rebar. To solve this dilemma glass FRP reinforcing bars were implemented [1]. In 1978, Germany made reports of doing experimental work using FRP materials for retrofitting concrete structures [1]. The United States’ interest in fiber-based reinforcement began in the 1930s; however it wasn’t until the 1980s that FRPs were considered in retrofitting owing to the initiatives of the National Science Foundation and the Federal Highway Administration [1]. However, it was in Switzerland in 1987 that the first application of externally bonded FRP systems to strengthen concrete bridges occurred [1].

Conventional materials and technologies have proven suitable for use in civil infrastructure but lack longevity and often rapidly deteriorate emphasizing a need for newer and more advanced materials [76]. Utilizing FRP composites as reinforcement and in rehabilitation of civil infrastructure is well covered in the literature and several state-of-the-art reviews have been published [13, 42, 64, 65, 74, 104]. FRPs can be externally bonded to reinforced concrete columns, wall-slabs and beams, bridges, and sewer pipes. Additionally, FRPs can provide seismic strengthening, increased environmental durability and corrosion resistance, and more.
FRPs have also been considered to reinforce steel structures as well [35, 122, 135]. Experimental studies have shown the compressive strength of repaired RC columns using externally bonded FRPs to increase from 100% to 500% depending on the materials and thickness of the FRP layers [90], nearly 300% in shear [134], and over 148% in flexural strength [127]. FRPs have shown popularity in relining sewer pipes, particularly cured-in-place pipe (CIPP), but also in other underground conduits, enabling rehabilitation without resorting to expensive excavation and replacement [146]. Characterizations have also been conducted including the influence of FRP plate length [100], width [130], type of fiber [133, 137], fiber orientation [133], surface preparation [100, 138], resin cure cycle [137]. Deformations and crack growth have been recorded using strain gauges [23, 100], displacement sensors [123], fiber optic sensors [70], laser moire [43], speckle interferometry [111], through measuring changes in electrical resistance [107], and using piezoelectric materials [67].

There are many advantages to using externally bonded FRPs compared to the more traditional rehabilitation methods and materials. Wood [146] recently stated it best in an article published in Composites Technology: “Fiber-reinforced thermoset resins have earned a place in the ‘industrial’ applications sector, that catch-all descriptor that encompasses applications in which toughness and corrosion-, fire- and fatigue-resistance eclipse aesthetics.” CFRP’s are well known for their high durability and good chemical, environmental, and corrosion resistance [63, 91, 97, 127, 145]. These properties allow CFRP structures to have longer in-service life cycles. However, it is the CFRPs very high strength to weight ratio fashioned by the lightweight carbon fibers that allow it to be a part of the civil engineering industry. CFRP’s very high specific strength and specific stiffness make it ideal over traditional civil engineering materials such as steel and concrete. Although steel can provide a higher strength than a CFRP, most of the steel
member’s strength is utilized in self-weight and superimposed dead load. The light weight of the CFRP allows the material to dedicate nearly all of its strength to the applied loading. CFRPs are anisotropic materials meaning that their material properties vary in the different material directions allowing for a more specifically designed structure. This is a great advantage for the structural engineer who can design the CFRP component of the structure to have high strength in only the direction of applied load limiting the amount of material used and limiting the amount of material wasted. Providing high quality fibers at the most effective regions in a structural member’s cross-section is a key innovation to the use of these high-performance materials [63]. If multiple loading directions are involved, the more complicated 2D and 3D woven plates can be implemented.

The structure of the composite is one of the many aspects that make it a valuable reinforcing material. The composite is made up of two parts: the fiber (reinforcement) and the resin (matrix). Each of these components plays a very important and specific role. The fiber is predominantly responsible for carrying the load of the composite. FRPs derive their mechanical properties wholly from the fiber and the bond at the fiber/matrix interface. The region where the fiber coating and matrix have fully bonded on a molecular level is known as the interphase. The interphase is poorly understood and holds responsibility for transferring the load from the matrix to the fiber. This dependence puts high significance on the quality of this bond. The resin serves as a protector to the fibers from the environment permitting load transfer from fiber to fiber. The resin is responsible for increasing the composite’s resistance to corrosion and degradation. The efficacy of the rehabilitation scheme relies on the combined action of the entire system with emphasis on the integrity and durability of the bond between the FRP and the concrete. If the FRP debonds prematurely, or a loss of force transfer occurs due to degradation of the bond
strength, the FRP strengthened structural component may fail at load levels lower than what it was initially designed for.

Glass fibers and aramid fibers are other commonly used fibers in industrial applications in addition to carbon fibers. Aramid fibers have seen the most success in bulletproof vests and the aerospace industry; they do not possess the ideal properties for the civil engineering industry. Kevlar 49 fibers in particular exhibit creep and high degradation when exposed to environmental species. Glass fibers have seen a lot of success in the automobile industry and have been well tested for application in infrastructure rehabilitation [18, 48, 69, 73, 78, 80, 81, 86]. Although glass fibers are more economical, they tend to absorb more moisture than carbon fibers and possess less strength along with a lower modulus [30, 82, 86]. Moisture uptake can affect the integrity of the bond between the FRP and concrete as well as the integrity of the composite itself. Ingress of aqueous solutions into the composite is known to cause fiber/matrix debonding and microcracking of the resin leading to additional absorbed moisture. In a durability test conducted by Kootsookos and Mourtiz [86], the glass fiber composites experienced more damage and deterioration than carbon fiber composites when exposed to seawater. This same result was observed by Karbhari and Ghosh [82] in their durability evaluation of externally bonded CFRP and GFRP strengthening systems as well as Chen et al. [30]’s durability evaluation using CFRP and GFRP reinforcing bars. On the contrary, however, Taheri et al.’s [132] results shows that GFRP plates can effectively replace CFRP plates for rehabilitation purposes.

Externally bonding FRPs has become a popular method for strengthening deteriorated structures due to the simplicity of the method, flexibility to adapt in the field, and long-term durability [2]. FRP plates are not very thick, and can be cut into any shape providing a high
coping ability with complicated shapes and corners making minimal changes to the aesthetic qualities of existing structures. There are different techniques and externally bonded strengthening schemes based around the system’s desired performance. These various schemes include side bonding, U-jacketing, U-jacketing with anchorage, and complete wrapping. Other popular FRP strengthening methods include wet layup of FRP laminas and near surface mounted FRP strips or bars. In a study by Karbhari and Ghosh [82] the authors showed that externally bonded prefabricated carbon-epoxy strips absorb less water than CFRPs laminates applied using wet layup. This is due to the higher fiber volume fraction of the prefabricated strips and use of elevated cure regimes. The wet layup strengthening scheme overall performed better than the prefabricated strips however. This victory is due to the thinner bond-line required and greater level of penetration into concrete, for the wet layup method.

The near-surface mounted (NSM) technique insets FRP bars or strips in grooves cut into the concrete surface on the tension side of the structure. Tests have shown that this technique can provide higher failure loads to the concrete structure compared with externally bonding FRPs along with exhibiting a more ductile failure [19, 28]. The major drawback to this technique is the increase in time, money and skill required for installation.

In addition to using adhesives to externally bond FRPs, the more traditional method of bolting or fastening can be used. However, like Adams et al. [4] states “With good design, adhesively bonded joints are capable of bearing loads equivalent to or in excess of their mechanically fastened counterparts.” When external reinforcement is mechanically fastened stress concentrations are created around the bolt holes and perforations in the plate and adhesive. This is a huge disadvantage for mechanically fastened strengthening schemes. In an adhesive
joint, stress concentrations can form at plate ends or due to extreme irregularities in the bond line, however these stresses are typically not as high as those occurring at bolt holes.

With the increased use of composites and popularity in externally bonding them as reinforcement, expanding markets have prompted the development and use of a greater variety of industrial grade resin formulations [146]. In particular, unsaturated polyesters and various epoxies have seen a significant increase in use. The mechanical properties coupled with good adhesion properties of synthetic thermosetting resins, particularly epoxies, are what enable their success. Recently, resin manufacturers AOC developed both green and styrene-free resins (for more information refer to www.green-resins.com).

Epoxy resins have seen use in many rehabilitation applications including filling cracks in concrete, bonding fresh concrete to hardened concrete, and the two applications used in this study: bonding concrete with another material and serving as the matrix of a composite used as reinforcement. Common epoxy formulations used in civil engineering applications are based on diglycidylether of bisphenol A resins (DGEBA) that can be cross-linked with a wide variety of hardeners such as amines, organic acids, anhydrides, and catalysts [47]. The performance and ultimate lifetime of an epoxy resin’s adhesive strength and heat and chemical resistance depends on the conditions in which the resin-hardener system was cured in and the environment it is exposed to throughout its life. It is most economical and practical for epoxy resins used in civil engineering applications to be cured at ambient temperature [5]. Ambient temperature curing is often assisted by the use of curing agents. Aliphatic amines and polyamines are two common curing agents used to induce cross-linking reactions at low temperatures [5]. Polymers can be very vulnerable to temperature, especially if the exposed temperature is near the material’s glass transition temperature. The glass transition temperature, $T_g$, is the temperature, or range of
temperatures, at which polymeric resins change from a rigid, glasslike state to an elastomeric like state, with a consequently dramatic decrease in mechanical and adhesive properties [5]. The $T_g$ of the epoxy varies based on the degree of cure and water content present in the material. Typically, the epoxy resin’s $T_g$ will be ~10-20°F higher than the temperature at which it was cured, generally ranging from 95-120°F [5]. This can present a problem since often the temperatures concrete structures are exposed to reach values within or higher than this range. In fact, even if the exposed temperature is slightly beneath the range, it is likely that the temperature inside the concrete element whose surface is directly exposed to the sun all day will reach temperatures in excess of the adhesive’s $T_g$. Once $T_g$ is reached ‘free volumes’ can appear within the composite’s matrix that can be filled by intruding molecules as the polymer attempts to restore equilibrium, potentially altering the material’s mechanical properties [127]. $T_g$ is dependent on the chemical bonds within the polymer’s structure and the amount of free volume in addition to the curing temperature. It can be used to assess the effects of moisture absorption or chemical diffusion on the physical degradation that can damage the bond in the polymer structure itself.

In this project, an ambient temperature cured epoxy resin-hardener was used in bonding the CFRP to the concrete substrate. In Karbhari and Ghosh’s [82] durability evaluation, the authors concluded that ambient temperature cured resin systems may show faster deterioration than the well characterized high-temperature and controlled condition cure resin systems in long-term performance. Therefore it is recommended for practical implementation purposes to use a high-temperature and controlled cure resin system. Regardless of the curing cycle used, it is important that all resins used in these applications are effective, safe and non-toxic.
In order for a resin-hardener system to be effective, the correct amount of adhesion must be present. Adhesion is a measure of the work required to separate dissimilar materials [27]. The magnitude of this work depends on the nature of bonding across the interface. There are five primary adhesion mechanisms that can improve the quality of the bond: absorption and wetting, interdiffusion, electrostatic attraction, primary chemical bonding, and mechanical interlocking. Chemical bonding requires intimate contact of two materials. The type of chemical bonding depends on the structure of the material near the interface. Primary covalent bonds and weaker secondary hydrogen and van der Waals bonds occur at the typical polymer/inorganic interface such as the epoxy/concrete interface [27]. Djouani et al. [47] conducted a study on the molecular interactions between cement paste and various epoxy adhesives. The results showed that specific hardeners exhibited such interfacial strength with ordinary Portland cement concrete that the interface was capable of proton transfer. It was also seen that epoxy adhesive minimizes the surface energy of the cement paste suggesting efficient wetting of the mineral substrate by the adhesive. This wetting behavior has a dramatic effect on the $T_g$ of the absorbed adhesive by comparison to the adhesive formulated in the absence of the mineral support. Moreover, the $T_g$ can be decreased to even lower values by the absorption of water acting as a plasticizer [5].

The use of CFRPs in civil infrastructure has not risen to its full potential yet due to the higher initial cost and lack of confidence in long-term results (i.e. after 50 years in-service). However, when considering their long life cycle with low maintenance, economical installation process, reduction in service disruptions and favorable aesthetic features, this initial cost is justifiable. In Hollaway’s [63] review of FRP utilization in civil infrastructure, the author states that when the whole-life cost of the construction and long-term durability of the material is taken into account, the initial cost of using CFRP materials is generally cheaper than that of the traditional materials.
Furthermore, metals and metal alloys are being consumed at an “ever-increasing” rate by China and India causing prices to escalate making composites more advantageous both in performance and from an economic standpoint [146].

However, there are disadvantages to this rehabilitation method. CFRPs and concrete have a pronounced difference in elastic properties, particularly in Poisson’s ratio. Poisson’s ratio compares the difference in shrinkage and expansion of the stressed material in two material axis directions. This difference in Poisson’s ratio can affect the bond between the materials if one is expanding and contracting at a higher rate than the other. It has been shown that the higher the Poisson’s ratio mismatch between the FRP plate and the RC beam, the earlier the likelihood of onset delamination [11, 132]. Au and Büyüköztürk [11] concluded from their parametric study that from the perspective of debonding resistance, one would prefer to have a bond line that has stiffness close to that of the adjoining concrete and a thin epoxy bond line when a considerably strong elastic mismatch is present, so as to minimize fracture energy release. On the other hand, if minimizing the debonding driving force is most important, then for a given set of constituent materials one would prefer to have a thicker epoxy bond line and a thinner FRP laminate.

Thermal residual stresses can develop when two or more materials possessing different coefficients of thermal expansion (CTE) are bonded together and exposed to a temperature change. The CTE of adhesives can be orders of magnitude different from those of bulk resins and/or composites [81]. These thermal residual stresses can cause premature debonding along the FRP/concrete interfaces. Residual stresses can also result in piezoelectric application or when a single layer swells when exposed to a diluent [60].
For successful implementation, the FRP-adhesive reinforcing system must possess sufficient strength and stiffness properties in order to resist the full superimposed and self-weight loads to which the structure is exposed along with the relevant in-service and physical characteristics required to function in the aggressive and sometimes hostile environments encountered in civil infrastructure [63]. The mechanism of failure depends on the loading, geometric properties, matrix toughness, and interface fracture resistance [26]. If the FRP is externally bonded to the tension face of a flexural beam, there are six failure modes the system could follow:

1. Tensile rupture of the laminate due to excessive strain demand in the maximum moment zone;
2. Concrete crushing caused by too much internal reinforcement;
3. Shear failure in the concrete section outside the area reinforced by the composite;
4. Peel failure at the plate end generally resulting in propagation of a horizontal crack in the concrete and separation of the concrete cover;
5. Debonding or adhesive failure at the concrete/adhesive interface generally initiated at a flexural or shear crack;
6. Interfacial debonding induced by an intermediate crack.

These six failure modes are depicted in Fig. 1.1. [63, 76, 97, 128]. FRPs show a linear elastic behavior up to failure, but in general their high strength is not achieved when they are used as external reinforcement due to debonding. Debonding depends on the bond behavior at the concrete and FRP interfaces, the location of which is dictated by the cracking pattern, internal steel reinforcement percentage, presence of steel stirrups, loading scheme, and interaction between shear and normal bond stresses along the interfaces [28, 40]. Debonding is often observed at discontinuities such as plate extremities due to load transfer from concrete to FRP.
plate (end debonding). The same load transfer can cause debonding at any existing crack mouths within the retrofit span (intermediate debonding) where high concentrations of shear and peeling stresses can be found [11, 100].

In this study, a static wedge is used to test the bond durability under various environmental exposures. The static wedge creates a tensile peel load along the adhesive/concrete interface resulting in one of two failure modes: adhesive or cohesive failures along the concrete/adhesive interface. These two failure modes are also known as interface separation and material decohesion [11]. Interface separation is a debonding mode governed by the adhesion property of the adjoining constituent materials, here being the epoxy adhesive and concrete substrate. Interface separation is exactly what it implies, separation of the two materials exactly along the interface between them. Material decohesion is when delamination occurs

![Failure modes of FRP-strengthened RC beams](image)

Figure 1.1: Failure modes of FRP-strengthened RC beams
between the concrete/adhesive interface and a tiny layer of concrete (< 1 mm) is detached with the adhesive. The cohesive failure is a result of material limits being utilized and is caused by the adhesive penetrating into the concrete providing increased strength just next to the interface. Combining this region of increased strength with high shear stresses acting along the concrete/adhesive interface will result in microcracks that tend to propagate away from the interface at the same angle as the principle compression direction [110]. The interaction and merging of these inclined cracks will produce the final debonding surface inside the concrete. Delamination is a very brittle failure mode and should be avoided.

To avoid debonding and delamination, specific guidelines should be followed. Worldwide, governments and engineering associations are standardizing workable international design parameters. The American Concrete Institute has three published guidelines: ACI 440-2R-08, a guideline for the design and construction of externally bonded FRP systems; ACI 440.5-08, a specification for construction with FRP reinforcing bars; and ACI 440.6-08, a specification for carbon and glass FRP bar materials for concrete reinforcement. Europe (Eurocode), Canada (ISIS Canada), Japan (Japan Society of Civil Engineers) and several other countries have published design codes and specifications for using FRP composites in civil engineering as well.
CHAPTER 2: THEORETICAL WORK

Material defects will develop through a structure’s service life instigated by various time-dependent processes (e.g. fatigue, creep, corrosion, and wear); particularly as subsurface delaminations due to low-energy impacts and edge delaminations occur. These defects along with existing manufacturing defects (e.g. micro-voids, matrix microcracks, and ply overlaps) create stress concentrations in the material making it susceptible to cracking. The fracture behavior of materials is concerned with the initiation and growth of such cracks that may cause premature failure in a structure [97]. The fracture mechanics approach evaluates bond degradation in terms of interfacial fracture toughness typically through DCB tests, peeling tests, or shear bond tests. Fracture toughness, $G_{IC}$, is measure of a material containing a crack to resist fracture. The interfacial fracture toughness is a good indicator of the extent of bond degradation while excluding the effects of material stiffness and member dimensions. The higher an interface’s fracture toughness, the more ductile failure it will have. The compliance approach is used in this study to obtain the expression for the interfacial fracture toughness. This approach is well adopted in the literature for calculating the interfacial fracture toughness. Employing fracture mechanics to theoretically explain the interface does present difficulty in directly implementing the interfacial fracture toughness in practical structural durability designs [40].
2.1 Linear Elastic Fracture Mechanics

Linear elastic fracture mechanics is widely used in the literature for modeling interfacial debonding of an adhesive joint as a horizontal fictitious crack growing along the interface of two materials. The FRP/concrete interface region is assumed to possess some safe amount of small cracks induced during manufacturing or by intermediate cracks existing in the concrete substrate. Due to the relatively low strength of the matrix, transverse cracks can be induced in the matrix of a unidirectional composite laminate by quasi-static or fatigue tensile loads. Once transverse matrix cracks are formed, local delaminations at the interface between lamina can be induced by the high stress concentration at the crack tip. The stiffness and strength of the laminate can be significantly reduced by these cracks and local delaminations [150]. When delamination occurs, mode II cracking occurs (i.e. tangential relative displacement between two materials). Debonding can be caused by diagonal shear cracks or stress concentrations at large irregularities on the concrete surface [100]. Interfacial debonding is a complex mixed-mode problem with both tangential and normal displacements.

In fracture mechanics, there are three general modes of deformation that may result from applied stress fields, depicted in Fig. 2.1. Fig. 2.1a shows mode I the opening component, Fig. 2.1b shows mode II the in-plane shear component, and Fig. 2.1c shows mode III the out-of-plane shear component. These three modes correspond to stress intensity factors $K_I$, $K_{II}$, and $K_{III}$ [120]. A crack growing in a bulk material induced by an applied mode I load translates to mode I fracture deformation. However, at the interface between two materials with a significant
Figure 2.1: Three general fracture modes: (a) mode I opening; (b) mode II in-plane shear; (c) mode III out-of-plane shear

difference in elastic properties an applied mode I load translates to mixed mode fracture energy consisting of an opening component and an in-plane shear component. The extent of this difference is determined by the extent of elastic mismatch. Au and Büyüköztürk [11] showed that when elastic mismatch between the epoxy and concrete layers is high, debonding at the interface is more likely to occur, given that the interfacial fracture energy remains unchanged. It is also observed that the effect of elastic mismatch would diminish when the epoxy bond line thickness is reduced for a given FRP laminate thickness. In 1969, Dundurs proposed two non-dimensional elastic mismatch parameters to describe the aforementioned situation. These two parameters, \( \alpha \) and \( \beta \),

\[
\alpha = \frac{\mu_1(k_2+1) - \mu_2(k_1+1)}{\mu_1(k_2+1)+\mu_2(k_1+1)}
\]

\[ (2.1) \]

\[
\beta = \frac{\mu_2(k_2-1) - \mu_1(k_1-1)}{\mu_1(k_2+1)+\mu_2(k_1+1)}
\]

\[ (2.2) \]

where

\[
\mu_i = \frac{E_i}{2(1+v_i)}
\]

\[ (2.3) \]
use the shear modulus of material $i$, $\mu_i$, with an elastic modulus $E_i$, a Poisson’s ratio $\nu_i$, and the bulk modulus $\kappa_i$. The bulk modulus is different under plane strain (Eq. 2.4) and plane stress

$$\kappa = 3 - 4\nu_i \quad (2.4)$$

$$\kappa = \frac{(3-\nu_i)}{(l+\nu_i)} \quad (2.5)$$

(Eq. 2.5) conditions. Because of the mismatch creating an additional mode II component even under uniquely mode I applied loads, a local mode II displacement will arise.

Pan and Leung [110] conducted a test analyzing a flexural-shear crack under combined pulling and peeling forces. The combination of both opening (mode I) and shearing (mode II) on the crack tip results in horizontal and vertical displacements between the two sides of the crack. The peeling force generates tensile stresses acting orthogonal to the interface making it easier for initial debonding to occur. This vertical displacement reduces the interlocking effect and accelerates shear softening [110].

In 2006, Au and Büyüköztürk [11] developed a LEFM solution to FRP/concrete interface debonding. The authors introduced a tri-layer interface fracture energy release rate model for five scenarios (two interface separation and three material decohesion models) of debonding between the FRP/concrete interfaces. Their model is essentially a direct application of the classical interface fracture model in bi-layered beams (Hutchinson and Suo (1990)) expanded to tri-layered beams. The authors developed solutions for the energy release rate for a typical peel model, Fig. 2.2, where $N_y$ is defined as the total peel load over the bond width (i.e. $N_y = P \cdot b$), $l$ is
the FRP cantilever length, and \( a \) is the pre-crack length. The solution for strain energy required for the FRP/epoxy layer to peel is

\[
G_{\text{peel}} = \frac{N_y^2}{b^2 E_{\text{concrete}} h_{\text{CFRP}}^3} \left( \frac{(l+a)^2 - l^2}{2 I_{1,2}} + \frac{6 l^2}{\Sigma_{13}} \right)
\]  

(2.6)

where,

\[
\Sigma_{ij} = \frac{E_i}{E_j}, \quad \bar{E}_n = \frac{E_n}{1 - \nu_n^2}, \quad \gamma = \frac{h_{\text{epoxy}}}{h_{\text{concrete}}}, \quad \eta = \frac{h_{\text{CFRP}}}{h_{\text{concrete}}};
\]  

(2.7)

\[
I_{12} = \Sigma_{13} \left[ \left( \frac{\gamma}{\eta} \right)^2 + (1 - 2 \delta) \left( \frac{\gamma}{\eta} \right) + \frac{3(1 - 2 \delta)^2 + 1}{12} \right] + \Sigma_{23} \left[ \delta \left( \frac{\gamma}{\eta} \right) \left( \delta - \frac{\gamma}{\eta} \right) + \frac{1}{3} \left( \frac{\gamma}{\eta} \right)^3 \right]
\]

\[
\delta = \frac{\Sigma_{12} + 2 \Sigma_{12} \frac{\gamma}{\eta} + \left( \frac{\gamma}{\eta} \right)^2}{2 \left( \Sigma_{12} + \frac{\gamma}{\eta} \right)}
\]  

(2.8)

A major drawback existed in their expression of energy release rate \( G_{\text{peel}} \), that is mode mixity of the interface debond caused by differences in elastic properties is not available. The expression for strain energy release rate, \( G \), used in this study is derived in the following section.
Another approach to the energy release rate at the crack tip is using traditional equation for a cantilever beam with a fixed end at the crack tip. Compared to the CFRP/epoxy layer, the concrete substrate is much stiffer. Therefore, the concrete block can be modeled as a rigid body and the driving energy release rate at the crack tip can be expressed as:

\[ G = \frac{9D\Delta^2}{2ba^3} \]  

(2.9)

where \( D \) is the bending stiffness of the CFRP/epoxy composite layer; \( a \) is the effective crack length; and \( \Delta \) is the thickness of the wedge.

2.2 Analytical Model

The Boeing wedge test specimen can be modeled as shown in Fig. 2.3. The beam is divided into two sections by the axis, a cracked portion and an uncracked portion. The cracked section \((a < x < 0)\) consists of the CFRP laminate and epoxy layer, modeled as a composite double cantilever beam (DCB) subjected to a peeling force \( P \), and total deflection of the beam is taken as the thickness of the wedge, \( \Delta \). The uncracked section \((0 < x < l-a)\) is analyzed as a
beam continuously supported on a Winkler elastic foundation. The reaction forces of the elastic foundation are taken to be linearly proportional to the beam deflection at any point. This assumption leads to open ended solutions; however, an idealization closely approximating many real situations is obtained [139]. The Winkler model is the simplest analytical model for a continuous elastic foundation and will be employed here. The Winkler elastic foundation can be used in many civil engineering applications including railroad tracks with the cross ties, ballast, and subgrade representing the elastic foundation, concrete footings on an earth foundation, a roadway surface resting on an earth foundation, long steel pipes resting on earth or a series of elastic springs, ship hulls, or a bridge deck or floor structure consisting of a network of closely spaced bars.

In the present model, the following assumptions were made:

1. The FRP composite and epoxy adhesive are linear elastic up to failure;
2. The thickness and the width of the FRP plate, epoxy adhesive layer, and concrete beam are constant along the bonded length;
3. The interface is subjected to only tensile forces acting orthogonal to the interface (i.e. mixed mode deformation is ignored);
4. The interface between the FRP and concrete is assumed to be of infinitesimal thickness.

Section I is modeled as an Euler-Bernoulli beam under general assumptions and boundary conditions. Cutting a segment out of section I, as shown in Fig. 2.4, and summing the moments about the axis origin generates the following relationship

\[ + \sum M_{0,0} = 0: \ M - P(a - x) \]  \hspace{1cm} (2.10)
For a beam with constant flexural rigidity $D$, the moment, $M$ can be expressed as follows in the governing equation for bending for section I

$$M = D \frac{d^2 w_I}{dx^2} = P(a - x)$$ (2.11)

taking the integral of Eq. 2.10 twice yields the deflection, $w_I$, for the cracked section of the beam:

$$w_I = \frac{P}{D} \left[ \frac{ax^2}{2} - \frac{x^3}{6} \right] + C_1 x + C_2$$ (2.12)

For section II of the beam, we can consider a semi-infinite composite beam supported continuously along its length by a foundation, itself assumed to experience elastic deformation, shown in Fig. 2.5. The force $q$ per unit length, resisting the displacement of the beam, is equal to $kw_{II}$. Where $w_{II}$ is the beam deflection with the positive direction pointing upward, as shown in Fig. 2.5, and $k$ is a constant representing the modulus of the foundation with dimensions of force per unit length of beam per unit of deflection. The beam has a constant cross-sectional area supported by an elastic foundation, with the left end at the origin, the $x$-axis passing through the
centroid of the beam, extending indefinitely in the negative $x$-direction, and the $z$-axis as the principal axis of the cross section.

Figure 2.5: Semi-infinite prismatic beam continuously supported on an elastic foundation

If no distributed loads are acting on the beam ($p = 0$), a segment of section II can be described as in Fig. 2.6. Summing the forces in the positive $z$ direction yields the following relationship

$$\uparrow \Sigma F_z = 0: V - (V - dV) - q * dx$$

(2.13)

By applying the relationship \( \frac{dV}{dx} = -EI \frac{d^4w_{II}}{dx^4} \) for a beam with constant flexural rigidity and $q = kw_{II}$, Eq. 2.13 resolves to

$$EI \frac{d^4w_{II}}{dx^4} + kw_{II} = 0$$

(2.14)

where the deflection $w_{II}$, subjected to reaction $q$, for a condition of small slope, must satisfy Eq. 2.14. The general solution to Eq. 2.14 will suffice and is expressed as

$$w_{II} = e^{\beta x}[D_1 \cos \beta x + D_2 \sin \beta x] + e^{-\beta x}[D_3 \cos \beta x + D_4 \sin \beta x]$$

(2.15)
Because the beam is infinite on one end, as \( x \to \infty \), \( w \to \infty \), and the two constants of integration \( D_3 \) and \( D_4 \) equate to zero. Therefore, the first portion of Eq. 2.15 can be removed and the general solution reduces to

\[
w_{II} = e^{\beta x} \left[ D_1 \cos \beta x + D_2 \sin \beta x \right]
\]  

(2.16)

where,

\[
\beta = \left( \frac{k}{4D} \right)^{1/4}
\]  

(2.17)

There are three known boundary conditions, Eq. 2.18-20, all located at the far left tip of the DCB \((x = a)\). These are the total deflection of the cantilever is equal to the thickness of the wedge, \( \Delta \), the curvature is equal to zero, and the shearing force is equal to the peel load, \( P \).

\[
w_I \big|_{x=a} = \Delta
\]  

(2.18)

\[
\frac{d^2 w_I}{dx^2} \big|_{x=a} = 0
\]  

(2.19)

\[-D \frac{d^3 w_I}{dx^3} \big|_{x=a} = P
\]  

(2.20)

Considering continuity at the crack tip \((x = 0)\) four relationships exist between sections I and II.
\[ w_I(0) = w_{II}(0) \quad (2.21) \]
\[ w_I'(0) = w_{II}'(0) \quad (2.22) \]
\[ w_I''(0) = w_{II}''(0) \quad (2.23) \]
\[ w_I'''(0) = w_{II}'''(0) \quad (2.24) \]

Before plugging in the boundary conditions and continuity conditions in order to solve for the constants of integration \( C_1, C_2, D_3, \) and \( D_4, \) first both expressions for deflection, \( w_I \) and \( w_{II}, \) must be taken out to their third derivatives

\[ w_I = \frac{P}{D} \left[ \frac{ax^2}{2} - \frac{x^3}{3} \right] + C_1 x + C_2 \quad (2.25) \]
\[ w_I' = \frac{P}{D} \left[ a x - \frac{x^2}{2} \right] + C_1 \quad (2.26) \]
\[ w_I'' = \frac{P}{D} (a - x) \quad (2.27) \]
\[ w_I''' = -\frac{P}{D} \quad (2.28) \]
\[ w_{II} = e^{\beta x} \left[ D_1 \cos \beta x + D_2 \sin \beta x \right] \quad (2.29) \]
\[ w_{II}' = \beta e^{\beta x} \left[ (D_1 + D_2) \cos \beta x + (D_2 - D_1) \sin \beta x \right] \quad (2.30) \]
\[ w_{II}'' = 2\beta^2 e^{\beta x} \left[ D_2 \cos \beta x - D_1 \sin \beta x \right] \quad (2.31) \]
\[ w_{II}''' = 2\beta^3 e^{\beta x} \left[ (D_2 - D_1) \cos \beta x - (D_1 + D_2) \sin \beta x \right] \quad (2.32) \]

Once this is accomplished, Eq. 2.27 can be set equal to Eq. 2.31 at \( x = 0 \)

\[ \frac{P a}{D} = 2\beta^2 D_2 \quad (2.33) \]
and the first constant of integration, $D_2$ can be solved for

$$\Rightarrow D_2 = \frac{Pa}{2\beta^2D} \quad (2.34)$$

Eq. 2.28 is set equal to Eq. 2.32 at the crack tip

$$-\frac{P}{D} = 2\beta^3(D_2 - D_1) \quad (2.35)$$

plugging in the value for $D_2$, the second constant of integration $D_1$ is obtained.

$$\Rightarrow D_1 = \frac{P}{2\beta^3D}(a + \frac{1}{\beta}) \quad (2.36)$$

Now, Eq. 2.25 is set equal to Eq. 2.29,

$$C_2 = D_1 \quad (2.37)$$

and the value for $D_3$ is plugged in, and $C_2$ is obtained

$$\Rightarrow C_2 = \frac{P}{2\beta^2D}(a + \frac{1}{\beta}) \quad (2.38)$$

Finally, the last relationship is used, setting Eq. 2.26 equal to Eq. 2.30 at the crack tip,

$$C_1 = \beta(D_1 + D_2) \quad (2.39)$$

plugging in $D_1$ and $D_2$, and solving for the final constant of integration, $C_1$

$$\Rightarrow C_1 = \frac{P}{2\beta D}(\frac{1}{\beta} + 2a) \quad (2.40)$$

Now that all four constants of integration have been obtained, the expressions for the deflection
of sections I and II become

\[ w_I = \frac{P}{D} \left[ -\frac{x^3}{6} + \frac{ax^2}{2} + \frac{ax}{\beta} + \frac{x}{2\beta^2} + \frac{a}{2\beta^3} \right] \] \hspace{1cm} (2.41)

\[ w_{II} = \frac{P}{2\beta^2 D} e^{\beta x} \left( a + \frac{1}{\beta} \right) \cos \beta x + a \sin \beta x \] \hspace{1cm} (2.42)

The applied tensile force \( P \) can be solved for by plugging the boundary condition described in Eq. 2.18 into Eq. 2.41

\[ P = \frac{\Delta D}{\left[ \frac{a^3}{3} + \frac{a^2}{\beta} + \frac{a}{\beta^2} + \frac{1}{2\beta^3} \right]} \] \hspace{1cm} (2.43)

Employing the compliance approach, defined as the ratio of opening displacement at the crack mouth, \( \Delta \), to applied load \( P \), that is

\[ C = \frac{\Delta}{P} = \frac{1}{D} \left[ \frac{a^3}{3} + \frac{a^2}{\beta} + \frac{a}{\beta^2} + \frac{1}{2\beta^3} \right] \] \hspace{1cm} (2.44)

the mode I energy release rate \( G_I \) is obtained, as by Carlsson and Pipes (1987) by differentiating the compliance with respect to crack length

\[ G = \frac{p^2}{2b} \frac{dC}{da} \] \hspace{1cm} (2.45)

Differentiating Eq. 2.44 and plugging into Eq. 2.45, the final equation for the mode I strain energy release rate

\[ G = \frac{p^2}{2bd} \left[ a^2 + \frac{2a}{\beta} + \frac{1}{\beta^2} \right] \] \hspace{1cm} (2.46)
and the interfacial fracture toughness

\[
G_{IC} = \frac{P^2}{2bD} \left[ a_c^2 + \frac{2a_c}{\beta} + \frac{1}{\beta^2} \right]
\]  \hspace{1cm} (2.47)

can be developed by setting \((a = a_c)\) so that the energy release rate is calculated at the critical

[crack length. The flexural rigidity, \(D\), of the cracked section, is obtained through transposing the

epoxy layer so as to use the value of \(E_{CFRP}\). The full solution is not obtained until the modulus of

the foundation, \(k\), is computed. This is accomplished using the relationship mentioned above

between the deflection of section II, \(w_{II}\), and the force resisting deformation of the beam, \(q\),

\[
q = k \cdot w_{II}
\]  \hspace{1cm} (2.48)

with the strain in the epoxy layer caused by the deflection of section II, \(w_{II}\),

\[
\varepsilon = \frac{w_{II}}{h_{epoxy}}
\]  \hspace{1cm} (2.49)

Here the force per unit length \(q\), acts over the width of the CFRP plate, \(b\), creating a uniformly

distributed stress on the layer equal to the modulus of elasticity of the epoxy layer, \(E_{epoxy}\),

multiplied by the strain in the epoxy layer, \(\varepsilon_{epoxy}\),

\[
q \frac{b}{E_{epoxy}} = \varepsilon_{epoxy} \cdot E_{epoxy}
\]  \hspace{1cm} (2.50)

Plugging Eq. 2.48 and Eq. 2.49 into Eq. 2.50, the modulus of the foundation becomes

\[
k = b \frac{E_{epoxy}}{h_{epoxy}}
\]  \hspace{1cm} (2.51)

This is a simplified expression that may overestimate the modulus of the elastic foundation. This

expected overestimation is caused by the fact that the thickness of the adhesive layer is large, and

greater than the thickness of the CFRP.
Wang and Zhang [143] proposed a novel three-parameter elastic foundation model to analyze the interface stresses on adhesively bonded joints. Their three-parameter model is a direct extension of the classical two-parameter elastic foundation model, and was inspired by the major drawback that the previous two-parameter models exhibited: not satisfying the zero shear stress at the free edges of the adhesive layer. In the new model, the peel stresses along the two adherend/adhesive interfaces of the adhesive joint are assumed to be different rather than the same. In this way, there are three different interface stresses existing in the adhesive layer: shear stress in adhesive layer, peel stress along the top adherend/adhesive interface, and peel stress along the bottom adhesive/adherend interface. The adhesive layer is modeled as two linear-normal spring layers interconnected by a shear layer. The model developed by these authors has an increased accuracy over the Winkler elastic foundation and is worth noting, although it was not used here.

2.3 Finite Element Analysis

Figure 2.7: Meshed deformed shape of finite element model of the CFRP-strengthened concrete specimen

Finite element analysis (FEA) provides a common way to calculate the energy release rate, $G$, and its mode mixity of an interface crack under general conditions. The commercially
available finite element software ANSYS 12.0 is used in this study to perform the FEA. The FE model analyzed here is composed of eight-node, quadrilateral higher order, and plane stress elements as shown in Fig. 2.7. This element has compatible displacement interpolation functions and good ability to fit curved boundary. The material properties used in the FEA are listed in Table 2.1.

<table>
<thead>
<tr>
<th>Material</th>
<th>Young’s Modulus</th>
<th>Poisson’s Ratio</th>
<th>Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFRP</td>
<td>131 GPa</td>
<td>0.28</td>
<td>1.4 mm</td>
</tr>
<tr>
<td>Epoxy</td>
<td>1.2 GPa</td>
<td>0.44</td>
<td>1.5 mm</td>
</tr>
<tr>
<td>Concrete</td>
<td>25 GPa</td>
<td>0.18</td>
<td>50.8 mm</td>
</tr>
</tbody>
</table>

Table 2.1: Material properties used in the FEA

Table 2.1. The length of the CFRP-strengthened concrete beam is 203.2 mm (8 in). A unit tensile load was applied at the top left corner and boundary conditions along the vertical outsides of the concrete beam.

2.4 Comparison and Evaluation

To verify the accuracy of the proposed model above, the analytical energy release rate and FEA energy release rate expressions are compared here. The same material properties and dimensions were used in both solutions. In order to obtain the compliance of the FEA, the analysis was run at various crack lengths ranging from $a = 8$ mm to 100 mm, measuring the displacement in the positive $z$ direction at the crack mouth, $\Delta$, for each value of $a$. A third order polynomial was fit to the curve to fill in all values of $a$. The energy release rate was then determined using the same expression as in Eq. 2.45, differentiating the fitted polynomial with respect to crack length, $a$. 
Fig. 2.8 compares the energy release rates versus increasing crack length for the previously existing model for a fixed cantilever beam, the analytical model derived in this study, and the finite element analysis. The curves in Fig. 2.8 compare excellently. Fig. 2.9 shows the energy release rate for the existing model and current model normalized by the FEM versus the increasing crack length normalized by the thickness of the CFRP plate. The present study’s curve in Fig. 2.9 is less comparable at lower crack lengths but levels out to an 11% error whereas the previously existing model’s curve is even less comparable at lower crack lengths but levels out to a 14% error. This resulting error is likely due to the estimation that the epoxy layer partially acted as an elastic foundation making it much softer in the analytical model than in real life. The present model is more accurate than the previous existing model for a fixed cantilever beam likely due to the fact that rotation is fully resisted at the crack tip in the existing model however some rotation is allowed in the present study’s model. Although the developed expression for energy release rate and interfacial fracture toughness were simplified models that neglect shear stress, the results are still fairly reliable when considering the length of the beam in practical application.
Figure 2.8: Energy release rate versus crack length

Figure 2.9: Energy release rates versus crack length per adhesive thickness
CHAPTER 3: EXPERIMENTAL PROGRAM

There exist various separate environmental conditions in which civil infrastructure may be exposed to: moisture/solution, alkaline environment, thermal cycling (i.e. temperature cycling and freeze-thaw), ultraviolet radiation, fire, and other chemical exposure such as de-icing salts. Synergistic effects are the effects resulting from the combination of multiple environmental conditions with or without the presence of a load, and are known to intensify individual effects. The effects of all said environments need to be evaluated for the fiber, interphase, and matrix of the composite, and for the bonding material and bonded substrate as well as for the system as a whole in order for successful implementation and understanding of long-term durability. The effects on subcritical debonding along the concrete/adhesive interface subjected to a pure aqueous environment, two aqueous solutions (alkaline and de-icing salt), and the synergistic effects of an aqueous environment coupled with an elevated temperature will be tested and reported here.

3.1 Environmentally Assisted Subcritical Cracking

Environmentally assisted subcritical cracking describes slow crack growth in an adhesive joint subjected to an aggressive environment. Subcritical cracking occurring under static loading is triggered by specific environmental conditions that encourage time-dependent crack growth processes. These processes include aggressive environments, residual stresses,
thermomechanical cycling, and mechanical or vibrational loading [85, 135]. The mechanism
causing subcritical crack growth involves the synergistic interaction of strained atomic bonds and
an environmental species [85]. Fig. 3.1 depicts the interaction of water with a strained bond at
the crack tip driven through forces below the critical fracture energy.

![Diagram of subcritical crack growth](image)

Figure 3.1: Subcritical crack growth onset by the interaction of
water with a strained bond at the crack tip

The stability of subcritical cracks is maintained by the fact that they are essentially
displacement controlled at the crack tip. A sharp crack tip is a common factor in all stable crack
growth processes. The crack tip’s sharpness enables high stresses and strains necessary for local
material failure to be sustained. Severe blunting of a critical crack dramatically reduces its
effectiveness in propagating a failure [135]. Local discontinuous crack advance can occur but
only within a strict limit. In the case of epoxy adhesives, stick-slip behavior has been attributed
to plastic crack blunting at the crack tip [148]. After crack arrest, a plastic zone is formed at the
crack tip. The crack grows subcritically through this plastic zone. Once the crack tip reaches the
outside of the newly formed plastic zone the crack tip advances at a critical rate. Microcracking
ahead of the tip is an essential feature of unstable critical crack growth. Crack growth will occur
if the equilibrium energy balance is disrupted by the strain energy exceeding the surface creation.
At a critical crack length, $a_C$, there is an equilibrium point that Griffith defined as the critical strain energy release rate $G_C$. The term subcritical refers to classification at applied strain energy release rate, $G$ (or stress intensity factor, $K$) values below the critical strain energy release rate, $G_C$ (or $K_{IC}$). Subcritical crack growth behavior is often described by the relationship of debond growth rate, $da/dt$, versus the driving energy release rate at the crack tip, $G$. An illustration of this curve is shown in Fig. 3.2. This curve can generally be separated into four sections, each representing a specific debond growth mechanism. Starting from the far right side in Region III where the curve is nearly vertical, as $G \to G_C$, the fracture processes dominate the crack growth rate without regard of environmental species. Once $G$ reaches $G_C$, debonding acts in a critical manner.
As $G$ deceases, Region II is encountered. This region of the curve is known as the transport controlled region consisting of slower crack growth at a somewhat constant growth rate with the diffusion of the environmental species to the crack tip being the rate-limiting step. Frequently, this region shows little to no dependence on the applied $G$.

In Region I, the crack growth is governed by the rate of reaction of environmental species with the strained crack tip bonds. At this section along the curve, the crack tip has opened up large enough and the crack growth due to mechanical loads has slowed enough for the environmental attack mechanism to readily occur. As a result, the crack growth rate in this section is dependent on both the reaction rate of the environmental species and the applied mechanical load, $G$. In general, the reaction between the environmental species, e.g. water, and a crack tip bond species, $A$, is described as,

$$nH_2O + A \rightarrow A^*$$  \hspace{1cm} (3.1)

where $A^*$ is the resulting activated complex [85]. The number of molecules of water required to break a single bond, $A$, of the material is assumed equal to the order, $n$, of the reaction with respect to water. For subcritical crack growth in Region I, existing studies on adhesive joints suggest that $G$ and $da/dt$ can be fitted into a power-law expression [85]:

$$\frac{da}{dt} = KG^n\exp(-Q/RT)$$  \hspace{1cm} (3.2)

where, $K$ is a constant dependent on the materials and relative humidity; $n$ is the crack growth exponent; $Q$ is the apparent activation energy of the interface required for thermally activated subcritical debonding; $R$ is the gas constant; $T$ is the absolute temperature in $^0K$. Eq. 3.2 is valid for moisture-assisted subcritical debond growth as well as creep-induced and cyclic fatigue load-
induced subcritical debond growth. The crack growth exponent $n$ is an important kinetic parameter of environment-assisted debonding because its value reflects the reaction mechanism at the crack tip [84]. If the viscoelasticity process occurring at the crack tip controls the debonding, $n$ should satisfy:

$$\tan \frac{n\pi}{2} = \tan \delta_r$$  \hspace{1cm} (3.3)

where $\tan \delta_r$ is the viscoelastic loss factor of the interface. If the dominating mechanism of debonding is moisture corrosion, the value of $n$ should be much higher than that given by Eq. 3.3.

Often a situation arises wherein the strain at the crack tip is not large enough to overcome the activation energy of the reaction with the exposed environmental species resulting in a sharp transition out of the reaction region. This performance is known as threshold. When the driving energy release rate, $G$, is less than the debond-growth threshold value, $G_{th}$ debond-growth/cracking is assumed dormant. The existence and magnitude of $G_{th}$ may depend on the energy barrier associated with the debond tip reaction process or steric hindrance of the environmental species [125]. This threshold may be achieved when the environmental species is unable to reach the crack tip. There have been cases in which an absence of a threshold existed [27,125] caused by an anomalous region of persistent debonding below $\sim 10^{-8} \text{ m s}^{-1}$. Once $G \approx G_{th}$, growth rates may continue to follow the reaction rate behavior in region I, exhibit a change in slope, or simply not exhibit a threshold at all, Fig. 3.3 [27]. The absence of a debond-growth threshold has very important implications for interface reliability, resulting in continued growth of interfacial defects even at very low loads.
As previously stated, in civil infrastructure the environments likely to be experienced by in-service structures include water, humidity, rain water, acid rain, sea-water, alkaline solutions, de-icing salts, temperature fluxes, exposure to freeze-thaw cycles, ultra-violet radiation, and fire. The criteria for choosing the best rehabilitation technique should begin with the system’s durability. The durability of a material or structure can be defined as its “ability to resist cracking, oxidation, chemical degradation, delamination, wear, and/or the effects of foreign objects’ damage for a specified period of time, under the appropriate load conditions and under specified environmental conditions” [50, 63, 75, 127]. Externally bonded FRP composites, in addition to providing reinforcement, can serve as a protective layer to the strengthened concrete as well as the internal reinforcement. Within a FRP composite, the polymer matrix is primarily responsible for providing environmental and damage protection to the fibers, toughness to the composite, and transferring the load between fibers. In externally bonded systems, the
composite matrix material and the adhesive used in external bonding are simultaneously subjected to the surrounding environmental (in this study both of these are a type of epoxy adhesive and therefore a thermosetting polymer). Thermosetting polymers are known to be relatively brittle and experience irreversible chemical changes at elevated temperatures on the mechanical, physical, electrical, and thermal properties [97]. Epoxies in particular have a good balance of physical, mechanical and thermal properties, normal curing times, and less shrinkage during cure [97]. Epoxy resins have three curing stages: “A” stage – the uncured, soluble stage; “B” stage – partially cured (Prepreg), the vitrified stage prior to gel point; “C” stage – fully cured, insoluble [97]. In the final cure stage, epoxy resins have a cross-linked molecular structure consisting of covalent bonds. A high degree of cross-linking leads to a decrease in the permeability with a consequential decrease in the diffusion process, thus it is necessary to fully cure the adhesive [63]. Under-cured epoxy has an increase susceptibility to moisture due to the lack of cross-linking. Epoxy resin’s molecular structure consists of three member rings with two carbon and one oxygen atom shown in Fig. 3.4 [97]; this single oxygen atom puts the adhesive bond at risk to hydrolysis. Adams et al. [4] and Kook et al. [85] report that distilled water is one of the most aggressive environments that an epoxide can be exposed to. Water can enter a resin by diffusion or by capillary action through cracks and crazes [51]. The OH groups in the adhesive’s molecular structure causes it to become polar attracting water molecules and thus
forming hydrogen bonds. The onset of hydrolysis can introduce detrimental effects on the system’s integrity. Other than hydrolysis, there are several other weakening mechanisms that can occur once water has entered the joint including plasticization, saponification and crazing [4, 8, 63, 81, 86]. When plasticization occurs, it typically lowers the $T_g$ of the cured resin, but is usually reversible. Saponification resistance is an important factor in wet adhesion. The onset of corrosion generates hydroxide ions at the cathode, raising the pH levels as high as 14. Ester groups in the backbone of a binder can be saponified, degrading the polymer near the interface and reducing the wet adhesion [145]. Furthermore, moisture can cause the adhesive to swell or warp putting stress on the joint. When epoxy adhesives are permanently immersed in water or exposed to de-icing salt solutions, the adhesive can crack or craze allowing moisture to permeate through the molecular structure and degrade the bonds between fiber and resin, and between the FRP plate and concrete structure [8, 55, 63, 81]. When the moisture permeates through, the adhesive (or resin) experiences a chemical change, reducing the material’s mechanical properties through a slow and irreversible variation in the molecular structure leading to the material’s aging. A study by Gomez and Casto [55] showed that water absorption can reduce the strength and stiffness by as much as 30%. In this project only the effects of moisture exposure are studied, however water diffusion in the strengthening scheme is an important factor in determining the long-term durability of the reinforced structure.

Temperature can also be very detrimental to the integrity of composites, especially if those temperatures are near or above the glass transition temperature, $T_g$, of the adhesive or resin being used. If the $T_g$ of the resin or adhesive is reached, an increase in viscoelastic response is seen with thermal softening of the polymer weakening the cross-linked structure, decreasing the strength and stiffness, and sometimes increased susceptibility to moisture absorption [81, 127].
Temperature fluxes in general are harmful to the FRP along with the externally bonded system as a whole due to thermal incompatibility. Coefficients of thermal expansion of carbon fibers (glass and aramid fibers as well) differ significantly from those of concrete and those of resins and adhesives. This difference can cause internal stresses to develop at the fiber/matrix interphase as well as at the concrete/FRP interfaces. Subzero temperature exposure, or freeze/thaw cycling, can be deleterious to resin and adhesive as well. When the polymer is exposed to very low temperatures, matrix hardening and micro-cracking occurs resulting in fiber/matrix bond degradation [81]. An increase in strength and stiffness can be seen at lower temperatures caused by embrittlement of the resin or adhesive. Further damage may be caused if there is water absorbed within the system while exposed to freezing temperatures. When the water expands upon freezing a loss in strength, rigidity, and toughness can be seen [55]. The overall durability of the system is effected and vulnerable to interfacial failure and delamination. However, Gomez and Casto [55] reported that in general it was found that very little moisture is absorbed during the freeze/thaw cycling.

The presence of salt during freeze/thaw cycles produces salt deposits can induce swelling and drying, accelerating the degradation process [81]. When the FRP rehabilitated structure is exposed to marine environments or cold regions where de-icing salts are used, exposure to chlorides can be severe. The diffusion of light atomic weight free ions, i.e. OH⁻ and Cl⁻, and H₂O molecules into the composite’s matrix triggers deterioration of the FRP [127]. Cracks and voids in the matrix accelerate the rate of absorption increasing the risk of hydrolysis and polymerization. It has been shown that salt solutions can also cause blistering between the concrete and adhesive due to osmotic effects [41]. Testing the effects of a de-icing salt solution
can help to understand and quantify the synergistic effects of osmotic effects and moisture on the durability of the FRP/concrete interfaces.

Furthermore, degradation can occur from the concrete pore water solution. This pore water solution is known to have a pH level as high as 13.5 [81]. Alkaline environments can have deleterious effects on the integrity of the whole system. The onset of corrosion raises the pH level as high as 14 causing degradation in the polymer near the interface and reducing the wet adhesion [144]. It is for this reason that the effects on the durability of the concrete interface will be tested in a sodium-hydroxide solution with a pH = 13.

The final two environments, UV exposure and fire, are not tested in this study even though the externally bonded system’s response to both of these is important for an accurate long-term durability understanding and design. Ultraviolet radiation has been reported to only affect the top few microns on the surface of a material or structure [81]. When this occurs to an FRP composite, degradation at the surface can occur to the resin resulting in flaws affecting the mechanical properties. These flaws can result in stress concentrations when combined with other environmental species. If the externally bonded FRP strengthened concrete structure catches fire and heats up the FRP or adhesive past their $T_g$’s traditional effects will take over including a loss of modulus. If the temperature is kept below that at which chemical degradation occurs, the loss in modulus can be reversed; otherwise a permanent loss in strength can occur [81].

It is important to note that the concrete also experiences chemical changes throughout its lifetime caused by simultaneous reactions of the various compounds composing Portland cement concrete. The hydration of Portland cement is a chemical reaction leading to the formation of hydrates from the anhydrous cement powder and water [98]. This complex process has
thermodynamic, kinetic, and structural features depending on both chemical and physical parameters [47]. Furthermore, when concrete is exposed to freeze/thaw conditions it is subject to continuous expansion and contraction resulting in microcracking and deterioration. Once this occurs, aggressive chemicals such as chlorides and salts can migrate into the concrete causing cracking and corrode the internal steel reinforcement. Shrinkage, thermal and humidity gradients can cause microcracking as well.

There are certain measures the engineer should take to avoid or limit the amount of environmentally induced cracking; these include careful material selection, heat treatment, manufacture and jointing technology, designing below threshold levels of stress, and applied protection or coatings [138]. Due to the effect of moisture on the $T_g$, composites and adhesives must be fully cured with the $T_g$ significantly higher than the maximum service temperature, a minimum of 30°F about the maximum service temperature is recommended by Karbhari et al. [81]. Additionally, silanes and organotitantes are two popular agents that have proven successful to coat fibers for a barrier against moisture and should be used as well.

3.2 Drawbacks in Existing Durability Studies

The use of FRPs in civil infrastructure has only just blossomed in the past fifteen years throughout which extensive experimental, theoretical, and analytical studies have been conducted on the mechanical properties along with the short-term and long-term environmental durability. Field-level repairs are being conducted including the development of design methods and efficient construction techniques. Even still, there are unresolved issues regarding long-term durability as well as damage and failure mechanisms in aggressive environments. This gap in
understanding has become a major barrier to their wide-acceptance in civil infrastructure. In fact, panelists of the Workshop on Research in FRP Composites in Concrete Construction identified the durability studies of FRP strengthening techniques to be research priorities [112]. The current strengthening guidelines, such as ACI 440, do not consider the long-term durability of the FRP/concrete interfaces. Rather, existing studies and design guidelines adopted a “knock-down factor” method to account for the long-term durability of the structures [36, 81]. The knock-down factors (i.e. the ratio of conditioned to pristine properties) are usually determined through accelerated aging tests. The major drawback of this method is that the real mechanism of deterioration is masked. The accelerated aging test can be inaccurate or even misleading [36, 81, 112], and no new information can be produced by this method to be used to improve the durability and long-term performance of the structure. Accelerated aging tests use an accelerating factor to age the material or system at a high rate, temperature is generally used. The use of very high temperature for accelerated aging can lead to over-degradation of the matrix or adhesive causing an underestimation of the durability FRP materials. However, the use of short test followed by extrapolating results can lead to erroneous results as well [81].

Of the limited number of studies conducted on the durability of FRP reinforced concrete structures, the main focuses were on the durability of the FRP or the resin [8, 15, 16, 17, 18, 22, 32, 44, 45, 48, 50, 55, 69, 73, 75, 78, 79, 80, 92, 95, 101, 119, 124] or the durability of the whole FRP-concrete strengthening system [29, 49, 54, 56, 58, 91, 94, 106, 115]. Very limited studies have been dedicated to the durability of the FRP/concrete interfaces [6, 12, 21, 24, 25, 40, 41, 59, 61, 68, 77, 93, 96, 105, 130, 140, 149]. In these studies, testing specimens were first conditioned in typical civil infrastructure environments such as various aqueous solutions [32, 73, 75, 78, 79, 80], freeze-thaw cycling [41, 58, 92, 93, 130], wet-dry and temperature cycling [40-41], cyclic
loads [49, 54, 61, 115, 148], and sustained loads [44, 48, 68, 93]. Synergetic effects of different environmental conditions were also considered in a few of the studies [8, 22, 68, 93, 101]. After environmental conditioning, specimens were loaded to failure to measure the deteriorated mechanical properties. Two general observations can be obtained from those studies: a) varying degree of deteriorations of structures are induced by environmental conditioning; b) structural failure due to interface debond occurs more frequently in conditioned structures than in unconditioned structures. This is in agreement with the extensive studies on adhesive joints [14], which indicated that moist in-service failures are caused by environmental degradation of the interface of adhesive-adherend. This suggests that the durability of the FRP/concrete interfaces play a critical role in the durability of FRP strengthened structures.

The behavior of the FRP/concrete interfaces in aggressive environments is mainly examined through experimental studies [6, 12, 21, 24, 25, 40, 41, 59, 61, 68, 77, 93, 96, 105, 130, 140, 149]. Grace and Grace [57] examined the effect on the FRP/concrete interfaces of various factors, and identified that moisture could do the most damage to the FRP/concrete interface. Based on this study and a few others [12, 24, 25, 109, 140], two conclusions can be drawn: substantial loss of the fracture toughness of the interface can be induced by moisture and the debonding locus shifts from within the concrete cover in its dry state to along the adhesive/concrete interface in its wet state.

An inherent problem with all of these studies is that only the loads occurred at the time of catastrophic failure are measured. As pointed out by RILEM [53], debonding is a “gradual process where slow growth in crack occurs at the interface.” The most distinct feature of these slow cracks is that they grow with a very slow rate at an energy release rate $G$ only a fraction of the critical energy release rate $G_C$ if reactive environmental species exist. This slow crack
growth is a long-term process of synergistic action of environments and mechanical loads. The catastrophic interface debond (critical crack) is only the ending point of this process. For any structure which requires long-term stability, a resistance to this slow crack growth would be needed. To understand the degradation mechanism of the interface and gain the ability to accurately predict the long-term durability ultimately requires quantifying and appropriate analysis of the slow debond growth process. Currently, it seems that FRP research groups are unaware of this significant role of slow debond growth, which could be a dominant mechanism for the failure of the FRP/concrete interface in service loads and aggressive environments. No study on this has ever been conducted or reported.

To date, no study has been published on the environment-assisted subcritical debond of the FRP/concrete interfaces. Meshgin et al. [102] found the FRP/concrete interfaces debonded 65 days after the specimen was subjected to a sustained load. This “delayed failure” is the result of subcritical debonding. The author thought this failure was “unexpected” and suggested further study should be conducted on this “delayed failure” due to creep since it is not considered in the current design guideline. The delayed failure due to creep was also investigated by Diab and Wu [46]. In their study, the authors observed debond grew slowly along the FRP/concrete interface until full debond occurred. An interesting phenomenon observed by Diab and Wu [46] is that the debond locus can shift from within the concrete to the epoxy/concrete interface if the debond speed is slow enough.

Furthermore, existing studies on the strength and durability of the FRP/concrete interfaces in aggressive environments only focus on Region III (in reference to Fig. 3.2). Region I and II are totally ignored. Since no environmental species can reach the crack tip in Region III, existing studies have to adopt a two-step approach. In the first step, the testing specimens are conditioned
in designed accelerated environments so that environmental species can reach the interface through diffusion and capillary action. In the second step, the residual strengths of the conditioned specimen are measured at catastrophic failure (i.e. Region III). This approach suffers a few obvious drawbacks:

1) The results of critical debonding testing can be misleading to be used to evaluate the long-term durability of the interface. This is because catastrophic failure-based testing can lead to a different failure mode from that of real applications. As demonstrated in many studies [46], interface debond may shift from adhesive failure at slow growth rate under service loads to cohesive failure at high growth rates under catastrophic failure.

2) Interaction between the environmental species and mechanical loads is lost. As a result, besides the ultimate strength, little information of the degradation mechanism of the interface under environmental species attack can be obtained from the testing.

3) Testing results are specimen-dependent. An accurate relationship between the concentration of environmental species depends on geometries and material properties of the specimen. Some important properties related to transportation, such as diffusion coefficient of the adhesive/concrete interface, are difficult to measure. As a result, the concentration of environmental species within the FRP/concrete interfaces is difficult to control and predict during testing, which makes it difficult to reproduce and compare testing results from different authors.

4) Environmental conditioning process usually requires a significant duration.

Thus far researchers have yet to study the long term effects of subcritical crack growth induced by various environmental species, coupled with a mechanical force, along the
concrete/adhesive interface. This interface represents the weakest region in the externally bonded FRP-concrete strengthening system and therefore is important to understand.

3.3 Research Significance and Objectives

To address the huge research gap in the long-term durability of the FRP-concrete interface and overcome the drawbacks in existing approach, a systematical experimental study will be conducted on the environment-assisted debond growth along the FRP-concrete interface under service loads in this chapter. The focus of this study will be on subcritical crack growth (region I) because our knowledge on it is virtually nonexistent and it provides a better way to study the long-term durability of the FRP-concrete interface for the following reasons:

1) The long-term durability of the FRP-concrete interface can be better characterized by subcritical crack testing because it closely simulates the failure occurring in the real-life application or service-life of the FRP-concrete interface. The change of debond locus occurred in catastrophic failure test can be avoided.

2) Interaction with environmental species is allowed in subcritical crack testing because of slow crack growth rate. This can help us to understand the degradation mechanism, chemical kinetics, and other phenomena of the interface debonding.

3) It takes much shorter time because there the time-consuming process of conditioning specimens is unnecessary.

4) The transportation of environmental species is separated from the reaction at the crack tip in subcritical crack testing. Therefore, degradation rate of the interface induced by environmental species can be accurately quantified. Testing results are specimen-independent. A mechanism-based model can be developed to predict the degradation of
the interface in aggressive environments. In existing studies, the transport of environmental species is always mixed with the reaction at the crack tip.

5) Ambiguity associated with bond strength due to competitive effects of concrete curing, long-term concrete strength gaining, epoxy creep, epoxy curing, can be reduced. This is because that the debond locus is within the epoxy-concrete interphase zone and that long-term environmental conditioning is not needed.

6) A realistic approach to predict service life of the interface can be developed based on the subcritical crack testing results.

7) Subcritical crack testing provides a far more useful indicator of interface quality than the ultimate bond strength of fracture toughness because the durability of the interface is generally more important than ultimate bond strength under harsh and changing environmental conditions. New knowledge to improve the durability of the interface can be obtained.

There are many environments an FRP strengthened concrete structure may be exposed to which can induce adverse effects on the FRP/concrete interfaces. Tap water at 70°F, 90°F, and 110°F (21°C, 32°C, and 43°C), deicing salt, and alkaline solution are considered in this study to maintain its reachability. The concrete, adhesive, and CFRP play important roles in the interface debond, however, the emphasis of this study is on gaining fundamental knowledge, data, and mechanisms of the environmentally-assisted subcritical debond of the FRP/concrete interfaces. Therefore, the material’s properties and geometries are fixed in this study. Their effects along with the effects of a silane coupling agent will be investigated in the future.
3.4 Materials and Test Specimens

A series of Boeing (static) wedge tests were conducted in various environments to characterize the subcritical crack growth along the concrete/adhesive interface. The static wedge test specimens were prepared with the epoxy layer constrained between the CFRP plate and the concrete substrate. The test specimen size and structure are shown schematically in Fig. 3.5. For testing, the specimens were restrained in the steel frame shown in Figs. 3.7 and 3.8 so that a steel wedge can be inserted into the interface between the adhesive layer and concrete substrate. Referring to Fig. 3.5, $L_a$ is the total crack length measured from the edge of the specimen to the crack tip; $a$ is the effective crack length measured from the contact point between the wedge and the FRP/epoxy layer to the crack tip. The total width of the specimen is kept constant for the CFRP, epoxy layer, and concrete beam at 50.8 mm (2 in). The steel wedges were fabricated according to ASTM standard D3762-98 [38], as shown in Fig. 3.9. Details on the concrete substrate, CFRP plate and structural adhesive, as well as the bonding process and testing procedures are described in the following.
3.4.1 Concrete Substrate

The materials used in fabrication of the concrete beams in this research project were donated locally by Bama Concrete in Tuscaloosa, Alabama. The concrete beams in this study are made from ordinary Portland cement with a maximum aggregate size of 9.525 mm (3/8 in). The beams were cast in steel molds, shown in Fig 3.6, with the dimensions of the concrete beams being 50.8 x 50.8 x 203.2 mm (2 x 2 x 8 in.). Concrete specimens were mixed, casted and cured following ASTM C33 and ASTM C192. Beams were manufactured in groups of 20 with 3 standard (6 x 12 in) testing cylinders poured as well. Before testing, the concrete specimens were moisture cured for 28 days then dried in the laboratory for an additional two weeks. The average compressive strength of the concrete used in this project is 27.6 MPa (4000 psi). The modulus of elasticity of the concrete is estimated using the following ACI equation in English units:

$$E_c = 4700*(f'_c)^{0.5}$$  \hspace{1cm} (3.4)

where $E_c$ is the modulus of elasticity of the concrete, and $f'_c$ is the specified compressive strength of the concrete both in MPa.
3.4.2 CFRP Plates and Structural Adhesive

Commercially available Aslan 400 CFRP plates were used to strengthen the concrete beams. The CFRP composite was prepared by 4,830 MPa, 227.5 GPa (700 ksi, 33 Msi) carbon fiber, approximately 70% by weight in an epoxy resin matrix as reported by the manufacturer. The CFRP plates were manufactured in a controlled and consistent environment with guaranteed physical and mechanical properties. To allow better bonding with structural adhesive, one side of the FRP plates was sanded by the manufacturer. The mechanical properties given by the manufacturer are given in Table 3.1. For the CFRP plates used in this experimental study,

<table>
<thead>
<tr>
<th>Laminate Width</th>
<th>Laminate Thickness</th>
<th>Cross Sectional Area</th>
<th>Tensile Strength</th>
<th>Tensile Modulus of Elasticity</th>
<th>Ultimate Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>50.8 mm</td>
<td>1.397 mm</td>
<td>71.10 mm²</td>
<td>2413 MPa</td>
<td>131,000 MPa</td>
<td>0.0187 %</td>
</tr>
</tbody>
</table>

Table 3.1: CFRP mechanical properties

<table>
<thead>
<tr>
<th>Epoxy layer Width</th>
<th>Epoxy layer Thickness</th>
<th>Cross Sectional Area</th>
<th>Tensile Strength</th>
<th>Tensile Modulus</th>
<th>Tensile Elongation</th>
</tr>
</thead>
<tbody>
<tr>
<td>50.8 mm</td>
<td>1.397 mm</td>
<td>71.0 mm²</td>
<td>22.7 MPa</td>
<td>1,200 MPa</td>
<td>1.88 %</td>
</tr>
</tbody>
</table>

Table 3.2: Epoxy mechanical properties

the recommended structural adhesive is Tyfo TC, which is a two-component adhesive based on a combination of epoxy resins and special filler. Tyfo TC is designed to bond the CFRP plates to the concrete substrates. The mechanical properties provided by the adhesive’s technical data
sheet are shown in Table 3.2; all values are assumed after curing for seven days at room temperature.

3.5 Methodology

The following section goes through the external bonding process, the experimental testing setup and research plan, along with the significance and applicability of the Boeing wedge test for this study.

3.5.1 Bonding CFRP Plates to the Concrete Substrate

The strength and effectiveness of externally bonded systems wholly depend on the quality of the bond between the structure and external reinforcement, thus the surface preparation of these materials plays a vital role. For the case of externally strengthening civil infrastructures in-service, there may be access limitations resulting in less than optimal bonding conditions. Karbhari and Zhao note that in this situation, mechanical interlocking is the dominant adhesion mechanism. Mechanical interlocking works through interlocking the adhesive into irregular rifts on the substrate surface to create a stronger interfacial layer. Sand blasting and grit blasting are often used in the literature to accomplish this, followed by air blasting and cleaning to remove all loose surface particles. It is important to not allow too much irregularity on the concrete substrate. Nonplanarity of the interface can significantly affect the measured subcritical debond energy [85]. If the concrete surface is extremely rough and uneven showing large irregularities, radial stresses can form at irregularities resulting in local microcracking [82].
In order to achieve the proper bonding performance, in this experiment the structural adhesive was mixed and applied onto the CFRP plates and concrete substrate following the strict bonding procedures described below:

1) Sandblast the bonding surface of the concrete substrate carefully to remove the loose particles, expose coarse and fine aggregates, and create a roughened surface.

2) Clean the sandblasted surface of the concrete substrate with high pressure air. This surface should be dry and free of contaminations to ensure good bond between the structural adhesive and the concrete substrate.

3) Apply a layer of primer onto the concrete substrate to create a uniform base for the epoxy material and seal the concrete surface. This water-based epoxy primer is mixed using two equal parts of primer and hardener. The primer is applied onto the concrete surface using a short nap roller and cured for 7 days.

4) Wipe the sanded side of the CFRP plate using white cloth and acetone to remove any excess residue from the CFRP plate until the cloth remains white after wiping.

5) Wrap the bonding surface of the concrete at one end of the concrete substrate with a Teflon tape to create a 3.175 mm (0.125 in) pre-crack along the interface of the concrete substrate and epoxy layer.

6) Measure 100 parts of component A to 23.3 parts of component B by weight and pour component B into the bucket of component A. Mix thoroughly for five minutes with a low speed mixer until component B is thoroughly dispersed.

7) Place four plastic spacers at the four corners of the concrete surface to ensure the adhesive layer has a uniform thickness.
8) Apply the structure adhesive onto the bonding surfaces with a spatula to a nominal thickness of 1.5875 mm (1/16 in) on both the concrete specimen and the CFRP plate. To minimize the presence of air pockets inside the adhesive when the CFRP plate was to be bonded to the concrete surface, the epoxy was smoothed in a V shape along the concrete surface, see Fig. 3.7.

9) Place the CFRP plate with adhesive onto the concrete specimens with the structural adhesive and press the CFRP plate with a hard rubber roller until the adhesive is forced out on both sides of the plates. Remove excess adhesive and make the glue line thickness equal to 1.5875 mm (1/16 in).

10) Apply proper pressure to the bonded specimens to ensure the maximum bonding strength is achieved. Cure the bonded specimens for two weeks.

11) Polish two sides of the specimens carefully to make sure no adhesive hinders the formation of the cracks.
3.5.2 Experimental Set-Up

The testing system used in the study consists of a stainless steel fixture for the CFRP-concrete specimen and a digital camera for monitoring the crack growth as shown in Figs. 3.8 and 3.9. The static wedge tests were conducted in six altered environmental exposures in an effort to measure the differences in crack growth rate and energy release rate. A dry, ambient condition was tested to set a standard as the control group. To test the effects of moisture and temperature, a submerged condition in ambient temperature 70°F (21°C) tap water, submerged in 90°F (32°C) tap water, and submerged in 110°F (43°C) tap water were compared. These ranges of temperatures cover the high temperatures the FRP strengthened structure will be exposed to. Allowing for three temperature settings provides knowledge on the extent that temperature will affect the interface, with the 110°F submerged testing approaching the T_g of the epoxy 160°F (71°C). The tests were also conducted in a calcium chlorides salt solution to simulate the deleterious effect of the de-icing salt agents used on highways during cold weather on the durability of the interface. A salt concentration of 4% will be used in this study. At this concentration, the maximum interface deterioration is reached [41]. It has been shown that salt solutions can cause blistering between the concrete/adhesive interface due to osmotic effects. Furthermore, the FRP/concrete interfaces can also be deteriorated by naturally occurring alkaline solutions due to the presence of concrete pore water. These solutions have high pH values (as high as 13.5), which can attack the adhesive. To evaluate their effect on the durability of the interface, the subcritical debonding test will be conducted in a sodium-hydroxide solution with pH =13.
The ambient air condition was the first test run to create a control group. The specimens were first placed in a stainless steel fixture, and then placed on a stable, flat surface beneath the cameras. The wedge in Fig. 3.10 was inserted between the concrete substrate and adhesive layer to form an initial crack length of about 62 mm. Accuracy of insertion along the interface was
insured by applying a thin Teflon film during external bonding to represent the presence of a precrack. The Boeing wedge tests were initially recorded for 5 hours of exposure. After analyzing the existing data, it was observed that very low values of crack growth rate could be recorded if testing was allowed to continue for up to 5 more hours. High resolution digital cameras were used to record the crack tip advance in real time. This technique is known as digital image correlation (DIC). The high-resolution images were taken of the crack tip zone every 10 seconds for the first hour, every 20 seconds for the next four hours, and every minute for the last 5 hours using the DIC software system. The length of the crack can be identified from these images recorded using the DIC system. To investigate the environmental effects, a

![Figure 3.10: Wedge dimensions](image)

transparent glass vessel was used to enclose the whole specimen and the load fixture during the test shown in Fig. 3.8. After filling this vessel with water or other aggressive chemical solutions, the whole specimen was submerged in these fluids. In this way, the specimen experienced both mechanical forces and the environmental forces.
3.5.3 Boeing Wedge Test

Boeing developed a test in the 1970s to assess the durability of different surface treatments on aluminum alloys exposed to various environments. The Boeing wedge test was developed in this effort as a cheap but reliable method with quality assurance [4]. The wedge test is commonly used among researchers for measuring the durability of adhesive systems [60]. The Boeing wedge test, or static wedge test, was chosen for this study so that the crack growth of the loaded specimen could be recorded while exposed to the various environments in a practical and easily reproducible way. Direct measurement of the crack extension makes it more convenient to assess the fracture energy at the concrete/adhesive interface. Figs. 3.8 and 3.9 display the simplicity in the test set up and loading fixture without the requirement of an expensive load cell.

A wedge is inserted at the concrete/adhesive interface creating an initial crack length. The crack growth is then “driven” by the restitution of stored, elastic, strain energy stored in the adherends displaced by the static wedge at a distance $\Delta$ [72]. The only difficulty in the test lies in the accurate measurement of the continuous crack growth. In this study, the classic “optical” method was employed through the use of high resolution cameras set to capture the crack growth every 10, 20, or 60 seconds throughout the duration of the test. Other research groups have found success in using strain gauges [23], displacement sensors [123], laser moire [43.], speckle interferometry [111], measuring changes in electrical resistance [107], and by employing piezoelectric techniques [67] to obtain crack growth. It has been demonstrated in the literature [42, 123, 131] that the actual shape of the crack tip is not straight or pointed, but rather a “thumbnail” shape with the length of curvature being approximately half the thickness of the wedge, as seen in Fig. 3.11. This shape does not have significant effect when the width of the
beam is 50.8 mm, however when we consider the width of an actual bridge and CFRP, this additional depth of the crack will be more significant.

![Diagram of a thumbnail-shaped crack tip](image)

**Figure 3.11: Thumbnail-shaped crack tip**

### 3.6 Experimental Results and Discussion

The effects of six environmental conditions on the debond growth rate and energy release rate of concrete specimens externally bonded with a CFRP strip using the Boeing wedge test are reported here. These environments include a dry, ambient condition, a submerged tap water at room temperature, an alkaline solution, and a de-icing salt solution with an applied static load throughout the durations of the tests. The synergistic effects of an applied static load in an aqueous environment coupled with an elevated temperature were tested as well at two temperatures: $33^\circ$C ($90^\circ$F) and $43^\circ$C ($110^\circ$F). The crack tips were identified through high resolution images taken during the tests that allowed accuracy up to 0.01 mm. Three identical specimens were tested in each environment; the specimen with closest initial crack lengths for each condition is reported here (comparison of individual results is found in the Appendix). The
energy release rate at the crack tip for the results of the experimental testing is calculated using the analytical model derived in the previous chapter

\[
G_I = \frac{p^2}{2bd} \left[ a^2 + \frac{2a}{\beta} + \frac{1}{\beta^2} \right]
\]  

(3.5)

Figs. 3.12 and 3.13 shows the growth in interface debond with time and the energy release rate, \( G \), with time, respectively, for the dry, ambient condition. This test is taken as a basis of comparison for the other testing conditions. Fig. 3.12 shows the interface debond starting at an initial crack length of 60.8 mm, exhibiting faster growth in the beginning, but as time progresses the debond growth decreases substantially leveling out to a very slow growth with a total growth less than 0.5 mm. Fig. 3.13 shows the energy release rate at the crack tip steadily decreasing

![Figure 3.12: Crack length versus time for the dry, ambient condition](image-url)
Figure 3.13: Energy release rate versus time for the dry, ambient condition

Figure 3.14: Crack growth rate versus energy release rate for the dry, ambient condition
Figure 3.15: Fracture surface of specimen in dry, ambient condition

![Fracture surface of specimen in dry, ambient condition](image)

a. Concrete substrate; b. epoxy surface on top of the CFRP

The highest values of $G$ are at the beginning of the test when faster cracking is occurring. Fig. 3.14 shows the crack growth rate versus the energy release rate. All three regions are visible in this plot and it is clear that subcritical cracking does exist along the epoxy/concrete interface due to the values of $G$ well below $G_C$. This exposure condition required the highest amount of energy dissipation at the crack tip ($G \approx 0.087 - 0.0878 \text{ } N/mm$). Figure 3.15 shows the fracture surface along the epoxy/concrete interface. A fairly significant portion of concrete was removed with the CFRP/epoxy layer around the edges of the specimen. This represents a weakness in the strength of the concrete, i.e. a cohesive failure.

When the presence of moisture is introduced, the total growth in crack length significantly increases. Looking at Fig. 3.16, the debond growth is not as continuous as in the ambient condition, but rather the curve appears as a succession of rapid growth and near arrest phases known as stick-slip crack growth. This stick-slip behavior has been attributed to plastic crack blunting at the crack tip, controlled by the yield behavior [149]. After crack arrest, a plastic zone is formed at the crack tip. The crack grows subcritically through this plastic zone. The same trend is seen in Fig. 3.17 as in Fig. 3.13, with the highest values of $G$ in the beginning
decreasing as time progresses. Fig. 3.18 shows the crack growth rate versus energy release rate while submerged in 70°F tap water. Regions I and II are visible in this figure and therefore we
know subcritical cracking exists in this environmental exposure as well. The presence of water weakens the bonds along the epoxy/concrete interface requiring less energy for crack extension ($G \approx 0.068 \text{–} 0.081 \text{ N/mm}$) in the $70^\circ\text{F}$ tap water than in the dry ambient condition ($G \approx 0.087 \text{–} 0.0878 \text{ N/mm}$). The values of debond extension nearly reach $1E-06 \text{ mm/sec}$, not quite as low as
in the dry, ambient condition. The energy required to extend the crack forward \((G \approx 0.068 - 0.081 \, N/mm)\) is slightly lower than in the dry condition. Figure 3.19 shows the fracture surface of the epoxy/concrete. Along the edges of the epoxy a shallow ring of the concrete substrate was removed indicating a cohesive failure.

Figure 3.20 shows the average extension is crack length with time for the specimens submerged in a de-icing salt solution. Stick-slip crack growth is clearly seen from the curve. Figure 3.21 shows the energy release rate with time for this exposure. This curve effectively shows that with each segment of stick-slip crack growth in Figure 3.20, the amount of energy required for debonding is reduced as well. Figure 3.22 provides the crack growth rate versus the energy release rate. Region II is the only clear region visible here with values of crack extension rate hitting as low as 1E-05 \(mm/sec\) with a possible descent into Region I. The values for \(G\) seen

![Figure 3.20: Crack length versus time submerged in the deicing salt solution](image-url)
Figure 3.21: Energy release rate versus time submerged in the de-icing salt solution

Figure 3.22: Crack growth rate versus energy release rate in the de-icing salt solution
Figure 3.23: Fracture surface of specimen submerged in the de-icing salt solution

a. Concrete substrate; b. epoxy surface on top of the CFRP

here \((G \approx 0.063 - 0.083 \, N/mm)\) touch just below the range for 70°F tap water condition. The fracture surface is shown in Figure 3.23 with a pretty clean adhesive failure visible with the exception of tiny bits of concrete pulled off along the edges of the adhesive layer.

As the environmental exposure increases in aggressiveness, it is seen that the crack growth versus time curve becomes near linear. Fig. 3.24 shows two increments of a slight stick-slip starting after one hour of exposure. Region II is very visible in Fig. 3.26 with most of the test occurring in this region. There may be descent into Region I, but it is not completely clear due to the lack of data points. The energy required to extend the debond ranged from \(G \approx 0.053 - 0.07 \, N/mm\) exhibiting the lowest values thus far. The fracture surface shown in Fig. 3.27 is similar to the fracture surface in Fig. 3.19 with a visible shallow concrete ring along the edges of the epoxy surface indicating the same cohesive failure, with the center being closer to an adhesive failure. However the concrete is much whiter in color caused by the reaction of the cement paste with the alkaline exposure.
Figure 3.24: Crack length versus time submerged in the alkaline solution

Figure 3.25: Energy release rate versus time submerged in the alkaline solution
Figs. 3.28 shows a steep increase in debond length for the first hour after exposure. After which, the debond tends to level out at a much slower rate. Fig. 3.29 shows the decrease in energy release rate required to extend the debond with time. A maximum $G = 0.083 \, N/mm$ is required in the 90°F submerged tap water submerged condition to initiate debond, but the range
($G \approx 0.05 - 0.083 \text{ N/mm}$) is almost identical to the alkaline exposure ($G \approx 0.053 - 0.07 \text{ N/mm}$) however as the exposure time increases, the heat from the elevated water induces thermal softening of the adhesive required 0.013 $\text{N/mm}$ less energy than in the alkaline exposure.

Figure 3.28: Crack length versus time submerged in 90°F tap water

Figure 3.29: Energy release rate versus time submerged in 90°F tap water
Fig. 3.29 shows the inverse trend of Fig. 3.28 with the required energy release rate with time. Region II is the only distinguishable region for the 90°F submerged tap water condition (Fig. 3.30). This region consists of slightly faster crack growth that is governed by the rate of reaction with the elevated water temperature at 90°F. Fig. 3.31 shows the fracture surface of the
specimen submerged in 90°F tap water. This picture shows a fairly clean adhesive failure along the epoxy/concrete interface possibly due to the thermal softening of the adhesive.

Fig. 3.32 shows a faster growth during the first two hours followed by a much slower, stable growth. At this temperature, the water is approaching the $T_g$ (160°F) of the adhesive. The smooth steady data is representative of thermal softening in the epoxy. The values of energy release rate ($G \approx 0.04 - 0.085 \text{ N/mm}$) are in the range of all other exposures, but reaching the lowest values seen. Regions I and II are visible in Fig. 3.34, with the bulk of the data in Region II. The crack growth rate hits values as low as $1E-05 \text{ mm/sec}$. Fig. 3.35 shows the fracture surface along the epoxy/concrete interface. The failure appears to be adhesive most likely caused by the viscoelastic deformation (i.e. elastic softening) of the epoxy caused by the elevated temperature.

Figure 3.32: Crack length versus time submerged in 110°F tap water
Figure 3.33: Energy release rate versus time submerged in 110°F tap water

Figure 3.34: Crack growth rate versus energy release rate submerged in 110°F tap water
The average results for all exposures are compared in Figs. 3.36 – 3.38. The aggressiveness of each environment tested as expected with the 110°F submerged tap water being the most deleterious, the alkaline solution being the second worst, and the 90°F submerged tap water being the third most aggressive environment on the concrete/adhesive interface. Subcritical cracking is seen along the concrete/adhesive interface in all environmental exposures, with $G \approx 0.085 \frac{N}{mm}$ required to initiate debonding in all but the 110°F submerged tap water condition. Regions I and II are seen in nearly all environmental conditions and are compared in Fig. 3.38. For the most aggressive environment, as little as 0.04 $\frac{N}{mm}$ of energy is required. Table 3.3 gives the average total crack extension for each environmental condition along with the average initial crack length. The 110°F elevated water condition created the highest average total extension in debond (13.2 mm), while the dry, ambient condition only caused an average of $< 0.5 \text{ mm}$ total growth. ASTM D3762, (the Standard Test Method for Adhesive-Bonded Surface Durability of Aluminum (Wedge Test)), requires that all initial crack lengths be identical, however this was only able to be controlled within 1.5 mm for the individual tests.
Figure 3.36: Crack length versus time for all environmental exposures

Figure 3.37: Energy release rate versus time for all environmental exposures
Figure 3.38: Crack growth rate versus energy release rate for all environmental exposures

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<td>Average total crack extension</td>
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Table 3.3: Results of Boeing wedge tests
CHAPTER 4: CONCLUSION

America’s current state of infrastructure is not functioning at a satisfactory level and the time for implementing plans of repair or reconstruction has come. Bringing the nation’s infrastructure up to a satisfactory level will take several years and over $2 trillion. Externally bonding CFRP plates to the flexural side of a concrete bridge or beam is an easy, effective, and economical method for providing additional strength to the member to insure its integrity for a full service life. Premature debonding of the CFRP plate is the most threatening failure mode of the strengthening scheme. Its likeliness to occur is especially increased by the presence of moisture and/or elevated temperatures nearing the glass transition temperature of the adhesive. Based on the results presented here, we know that subcritical crack growth does exist along the concrete/adhesive interface. The extent of this slow growth varies based on the environmental exposure. Elevated temperature and alkaline aqueous solutions are the most deleterious to the concrete/adhesive interface requiring the least amount of energy to extend the debond. Measures can and should be taken to minimize the effect of the exposed environment such as proper surface treatment, application of primers and specialized coatings, use of prefabricated FRP plates, and adhesives with high Tg’s. Threshold values were obtained in nearly all environmental exposures with the extension in debond assumed to be dormant after slowing to about $1E-06 \text{ mm/sec}$. Long-term debonding behavior can be predicted within practical accuracy using developed analytical and numerical models that can be used to predict when maintenance of the structure is required.
The preceding work is presented in an effort towards the successful and economical rehabilitation of concrete infrastructure, specifically bridges and flexural beams. The method of externally bonding a CFRP to the tension side of the concrete member for additional reinforcement using an epoxy structural adhesive and properly prepared concrete surface is recommended. The basis of this recommendation comes from an experimental study on the environmentally-assisted subcritical crack growth induced along the concrete/adhesive interface with a theoretical model for predicting debonding behavior after an extended time. The long-term debonding model was verified with a finite element analysis using the compliance approach for determining the expression for interfacial fracture toughness. The results of this study along with all other existing knowledge on the method of external strengthening concrete beams using CFRP plates should be used to validate the method into active practice and design codes.
REFERENCES


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113. —. "Workshop on Research in FRP Composites in Concrete Construction." NSF, 2005. 35.


APPENDIX

Appendix A: Individual specimen experimental results for each environmental exposure

Specimen results for dry, ambient condition:

Crack growth versus time for all three test specimens in the dry, ambient condition
Energy release rate versus time for all three test specimens in the dry, ambient condition

Crack growth rate versus energy release rate for all three test specimens in the dry, ambient condition
Specimen results for the submerged, ambient (70°F) tap water condition:

- Crack growth versus time for all three test specimens in the submerged 70°F tap water

- Energy release rate versus time for all three test specimens in the submerged 70°F tap water
Crack growth rate versus energy release rate for all three test specimens in the submerged 70°F tap water.
Specimen results after being submerged in a salt solution using commercially available de-icing salt:

Crack growth versus time for all three test specimens in the submerged de-icing salt solution

Energy release rate versus time for all three test specimens in the submerged de-icing salt solution
Crack growth rate versus energy release rate for all three test specimens in the submerged de-icing salt solution.
Specimen results after being submerged in an alkaline solution using NaOH to create a pH = 13:

Crack growth versus time for all three test specimens in the submerged alkaline solution

Energy release rate versus time for all three test specimens in the submerged alkaline solution
Crack growth rate versus energy release rate for all three test specimens in the submerged alkaline solution
Specimen results after being submerged in elevated (90°F) tap water:

![Crack growth versus time for all three test specimens in the submerged 90°F tap water](image1)

![Energy release rate versus time for all three test specimens in the submerged 90°F tap water](image2)
Crack growth rate versus energy release rate for all three test specimens in the submerged 90°F tap water
Specimen results after being exposed to elevated temperature (110°F) tap water:

Crack growth versus time for all three test specimens in the submerged 110°F tap water

Energy release rate versus time for all three test specimens in the submerged 110°F tap water
Crack growth rate versus energy release rate for all three test specimens in the submerged 110°F tap water
Appendix B: Finite Element Model and Results

EVALUATE MATERIAL PROPERTIES FOR MATERIALS 1 TO 3 IN INCREMENTS OF 1

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FEA input material properties

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PARAMETER STATUS- (9 PARAMETERS DEFINED)
(including 4 internal parameters)

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FEA input parameters

FEM numbered and meshed lines
FEM numbered areas

FEA meshed specimen pre-post-processing