Parametric Study of the Bond Between Fiber Reinforced Polymers and Concrete using Finite Element Analysis

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PARAMETRIC STUDY OF THE BOND BETWEEN FIBER REINFORCED POLYMERS AND CONCRETE USING FINITE ELEMENT ANALYSIS

by

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ABSTRACT
PARAMETRIC STUDY OF THE BOND BETWEEN FIBER REINFORCED POLYMERS AND CONCRETE USING FINITE ELEMENT ANALYSIS

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Ever since Fiber Reinforced Polymers (FRPs) have been used to externally reinforce concrete beams, it has been known that the strength of the bond is what controls the design. Recently, it has been proven that the primary mode of debonding failure is cohesive failure within the concrete substrate close to the FRP/concrete interface. To further enhance the research in this area, finite element analysis has been used to help study this debonding phenomenon.

This thesis is a parametric study of the FRP/concrete interface using finite element analysis software. The finite element model was calibrated using experimental data. The parameters taken into consideration were the geometric configuration of the materials (thickness), the material properties (Young’s modulus), the effects of the cohesive zone elements (maximum stress and its corresponding displacement), and finally mixed mode loading conditions.

Conclusions were then made concerning the effects of adjusting the parameters of the model. By increasing the respective thicknesses of the FRP, adhesive, and residual thickness of concrete, the maximum peel load was increased, however, debonding was also faster. In general, increasing Young’s modulus of the individual materials had the same effect, increasing the maximum peel load. Mode II loading in the mixed mode loading conditions has a severe impact on the bond behavior, reducing the maximum peel load for initial debonding.
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Chapter 1 Introduction

1.1 Background

With the turn of the century it has become evident that the world's concrete infrastructure is in desperate need of repair. In 2001, 23% of the 576,000 bridges across the United States were considered structurally deficient according to the US Department of Transportation and The Federal Highway Administration (Giurgiutiu et al. 2001). Reasons for the deficiency can be attributed to increased highway loads and frequency, age, and environmental factors. Steel plates were first used to retrofit both steel and concrete failing bridge beams. This was done by attaching the plates to the tension flange of the beam using an epoxy bond. The major drawbacks of using steel plates are their lack of corrosive resistance and heavy equipments needed to install them. This is when fiber-reinforced polymers (FRP) began to gain interest. Along with better corrosive resistance, FRPs have high specific stiffness and specific strength ratios making them a better alternative to the conventional strengthening and repair materials. An et al. (1990) were some of the first to use epoxy bonded fiber-reinforced plates in lieu of steel plates. Theoretical and experimental studies have shown that externally bonded FRP composites can be used to improve the desired performance of a structural member such as its load carrying capacity, stiffness, ductility, performance under cyclic and fatigue loading and environmental durability.

Composite overlays are thin sheets of fiber reinforced polymeric material adhesively bonded to conventional construction materials. The polymers used include epoxy, polyester, and vinylester. The fibers used can consist of glass, carbon, aramid (Kevlar), or a combination of these materials. These fibers are available in many forms,
including weave and non-woven fabrics, pre-cured plates and manufactured structural members. The composite may be applied to its substrate as a wet lay-up, pre-cured panels, or partially-cured pre-impregnated sheets. One critical issue raised by the structural engineers is the still unknown in-service durability of these new material systems. Their ability to safely perform after prolonged exposure to service loads and environmental factors must be ascertained before wide acceptance in the construction engineering community is attained (Giurgiutiu et al, 2001).

Since 2000 many efforts have been made to create standards and procedures for design involving FRPs. Many of these created procedures used to analyze the structural capacities of the FRPs and the components to which they belong are close approximations based on experimental data taken from lab specimens. The primary focus for research on the structural capabilities of the FRP composite structure lies in the bond between the FRP and the substrate it is connected to. This is because the primary failure mode of composite beams in flexure is debonding of the FRPs from the substrate.

1.2 Research Objectives and Significance

Many experiments of different configurations have been completed over the previous two decades. All are trying to ultimately determine the structural capabilities of composite FRP/concrete beams. Lab experiments are great to determine many critical properties of FRP strengthened concrete beams with certain geometric sizes and a few changes of material properties. However, it is difficult and not economical to test beams with various combinations of different parameters. The best remedy for this is the computer. Software has become an integral part of structural engineering. When used
correctly, software can model a specimen as little as a cluster of atoms or as large as a skyscraper and predict the outcome after loads and displacements are applied to the specimen.

There are many structural applications of FRPs in engineering, however the focal point of this research was centered on the development of a numerical model using ANSYS finite element analysis software, which can be used to evaluate the mechanical behaviors of FRP bonded concrete specimens. After the model was calibrated using experimental data, it was used to conduct a parametric study on the FRP bonded concrete Modified Double Cantilever Beam (MDCB) specimen. The parameters evaluated in this research were thicknesses of FRP, adhesive, and residual thickness of concrete (RTC). Also evaluated were the effects caused by changing the modulus of elasticity of the FRP, adhesive, and concrete. In addition to studying the effects of changing the geometric and material properties, mixed mode loading effects were taken into account. The individual components of the fracture energy equation were also assessed.

1.3 Outline and Organization of the Thesis

Chapter 2 in this thesis is a literature review. The review begins with a look at where FRPs came from as well as the part they play with strengthening existing structural members. A comprehensive background for the FRP/concrete interface is then presented, identifying the primary failure modes of debonding. The concept of fracture mechanics is covered next, leading to the discussion of cohesive zone modeling. Finally, experimental test approaches and some finite element analysis models are summarized.
In Chapter 3, the calibration model is described. The background for the finite elements used in the FE models is provided. The geometric and material properties used in the calibration are then defined. How the cohesive zone properties were selected is then covered. Also presented in this chapter is the importance of modeling concrete as a nonlinear instead of linear elastic material. Finally the calibration is verified.

In Chapter 4, the results of the parametric study are presented and chapter 5 consists of the conclusions drawn from this research as well as recommendations for future work.
Chapter 2 Literature Review

Even though there were no design codes to follow, civil engineers have been incorporating FRPs into their design since the early 1960s (Bank 2006). These individuals gained their exposure to FRPs either through personal studies or through other engineered applications of FRPs. They noted the unique advantages of FRPs used to reinforce concrete, beginning with the resistant corrosive properties. Brandt Goldsworthy, widely regarded as the inventor of pultrusion for FRP, stated "the chemical inertness of this material allows its use in... concrete reinforcing and all types of structural members that are subject to corrosive action in chemical plants or other areas where corrosive conditions exist" (Bank 2006). Over the course of many years, those with experience with FRPs began to work together to create new FRP components for structures. 1993 marked the first of a series of biannual international symposia devoted to FRP reinforcement of concrete structures, The International Symposium on Fiber Reinforced Polymer Reinforcement for Concrete Structures (FRP-RCS). It was then that international research interest in the use of FRP in concrete increased dramatically. Today many new developments and research can be found in the *Journal of Composites for Construction*, established by American Society of Civil Engineers (ASCE), and many other journals and conference proceedings.

2.1 FRP Strengthening of Existing Structural Members

The use of FRPs in existing structural members is known as a retrofitting application. There are two types of this, the first of which is strengthening.
Strengthening is considered when an existing building is forced to increase its load capacity due to either updating the structure with current codes or a change in the structure’s original purpose. One of the main reasons for a building requiring strengthening is the implementation of new seismic codes and the desire to update seismic stability. The other type of retrofitting is classified as repair. Repair is needed in buildings where corrosion caused by environmental effects has damaged the steel reinforcement and concrete. Also repair is needed in cases where construction was not done correctly, and the proper amount of reinforcement has been left out. "Although these two types of applications are similar, there are important differences that are related primarily to evaluation of the existing structural capacity and the nature of the repair to be undertaken before FRP can be used" (Bank, 2006). Often strengthening is considered a part of a repair design to add a level of safety and also to account for the uncertainty of the old design.

Of the methods that exist to attach FRP composite materials, two are primarily used. One employs adhering pre-manufactured rigid FRP strips using an epoxy. The other is known as hand layup. The hand layup process is very similar to that of a physician creating a cast on an arm; flexible dry fiber fabrics are "glued" on the structures surface using liquid polymers. Recently, pre-manufactured strips have been used in near surface mounting. This process involves a thin, narrow strip being inserted and bonded adhesively into a machined groove on the surface of the concrete member.

The initial research was primarily conducted on flexural strengthening; however it was not long before studies involving concrete confinement in columns using FRPs began. Wraps, as they are called, address numerous deficiencies present in concrete
columns subjected to seismic loading. This field developed out of the experience gained with retrofitting concrete with steel jackets.

The study of flexural design and capacity first originated in Switzerland under the direction of Urs Meier in the late 1980s (Bank 2006). Since then, many independent studies have been conducted in order to find the most effective way to use FRPs in construction and retrofitting applications.

2.2 FRP- Concrete Interface

When considering externally strengthened or repaired concrete slabs, beams and walls, the most crucial part is the connection between the concrete and FRP. Organic adhesives, or epoxy, are used to make the connection between the concrete and FRP. The bond is the means for the transfer of forces between the concrete and the FRP composite. If the connection between FRP and concrete fails, the structural advantages gained by using the FRP are lost. Therefore, the bonding property between concrete and FRP dictates the performance of repaired and strengthened concrete. Previous studies have shown that the bond between FRP and concrete has outstanding short-term performance. The failure of FRP retrofitted concrete normally occurs in the concrete substrate, and not the bond itself.

The following failure modes occur under perfect bonding, meaning the bond is maintained until failure happens; FRP Rupture: The FRP strip fracture is seen in specimens with relatively low ratios of internal steel and external FRP, and normally occurs before the yield of internal reinforcing bars; Concrete Crushing: Concrete crushes in the compression zone before or after the yielding of the internal reinforcing steel while
the FRP composite is intact; *Shear failure of reinforced concrete (RC) beam*: Shear failure occurs in the concrete beam before the flexural capacity of the FRP-strengthened member is reached (Bank 2006).

There are five different types of debonding failure, or loss of composite action between FRP strip and RC beam which takes place at the bond interface region. Debonding failure does not allow the FRP to reach its full strength. The five types are depicted in Figure 2.1 and described in the following (Ouyang 2008).

1) Cohesive failure in the surface concrete along a weakened layer or along its interface with the embedded reinforcing steel;
2) Adhesion failure at the bond interface between concrete and adhesive;
3) Cohesive failure in the adhesive layer;
4) Adhesion failure at the FRP/adhesive interface;
5) Delamination failure in the FRP layer;

![Figure 2.1 Possible debonding locations (Ouyang 2008).](image)

In general, cohesive failure in the concrete and adhesion failure at the FRP/adhesive interface dominate debonding failure while the other three are encountered
rarely as long as the materials used are meet the required standards as well as the application process is done correctly.

The debonding in FRP strengthened members can take place within the material or at the interfaces of materials that form the strengthening system, favoring a propagation path that requires the least amount of energy. Catastrophic failure of the structure can occur when a sudden loss of bond through delamination happens. Good adhesion between the composite overlay and the concrete substrate is critical for preventing early failure in externally FRP reinforced beams.

When a composite beam of reinforced concrete with externally bonded FRP is subjected to flexural loading, the concrete near the adhesive layer experiences high tensile and interface shear stresses. It is because of these high stresses that debonding of the FRP plate from the concrete occurs. The weakest part of the bond is the concrete layer adjacent to the bond interface. If the interfacial stresses cannot be sustained by the concrete, then debonding of the FRP can occur.

Regions of high stress concentration at the interface include the ends of the FRP reinforcement and areas around the flexural and shear cracks. It is in these regions where FRP delamination starts. Therefore codes and guidelines concentrate on two debonding failure modes: plate end debonding and mid-span debonding. For plate end debonding, failure originates near the plate end and propagates in the concrete from one of two places. Failure can occur at either the cut-off point of the plate, which is called plate end shear failure, or at the last crack, which is referred to as anchorage failure at last crack. The second failure mode starts at a shear or flexural crack and then propagates from such a crack towards the plate end. These failure modes are shown in Figure 2.2.
To prevent debonding from occurring, various methods, such as U-anchors, L-shaped plates and anchor bolts, have been developed. However, these are only used to secure the plate at the end of the beam. Flexural/shear cracks caused by external loading can create debonding within the mid-span. The FRP/concrete interface is subjected to normal and shear stresses. This means that there is present a mixed mode of loading.

A crack at the interface can experience three different loading types, tension, in-plane shear, or torsion (out-of-plane shear). These are more commonly referred to as mode I, mode II, and mode III respectively, which are shown in Figure 2.3. The most important modes of loading that must be considered when dealing with FRP/concrete bond strength are modes I and II. As discussed in the previous paragraph, the flexural/shear cracks introducing debonding are subjected to mixed-mode I and II loading at the FRP/concrete interface. Debonding propagation is dependent on loading conditions, material properties (strength and stiffness), and the fracture properties at the debonding crack tip.
2.3 Fracture Mechanics Background

Debonding is essentially a crack propagation promoted by local stress intensities. This concept has raised interest among many researchers to take a fracture mechanics approach to the problem and develop predictive models that utilize elastic and fracture material properties. Several models based on fracture mechanics have recently been developed to study the phenomenon of crack propagation in concrete, and FRP debonding from concrete substrate. As discussed by Ceriolo (1998), one notable advantage of the fracture mechanics method is that stiffness and size effects are included in the analysis. Wittmann (2002) predicted cracking in concrete under different boundary conditions due to shrinkage using a fracture mechanics method. Karbhari and Engineer (1997) used a peel test to measure the mixed mode interface fracture energy between concrete and FRP. Neubauer and Rostasy (2001) performed bond strength tests to determine the FRP concrete interface fracture energy. Giurgiutiu et al. (2001) used fracture mechanics to develop an energy release rate concept which was used to analyze
debonding of a composite layer modeled as an elastic cantilever. Wan et al. (2004) investigated the FRP/concrete interface behavior under the mixed mode loads and measured the critical crack tip opening displacement (CTOD) using computer vision technology during their modified double cantilever beam (MDCB) tests. Buyukozturk (2004) employed a fracture energy based criterion to predict debonding failures in reinforced concrete beams and study the effects of the environment and cyclic loading on bond strength. Dai et al. (2005) developed an analytical method for defining the nonlinear bond stress-slip models of FRP sheet/concrete interfaces through a pullout bond test. Two parameters, the interfacial fracture energy and the interfacial ductility index, are necessary in their model to study the effects of all interfacial components. A non-linear FRP-concrete interface law was developed by Ferracuti et al. (2006). Ouyang and Wan (2008) used MDCB tests and fracture mechanics to study the effects of moisture effects on the FRP/concrete bond interface.

Fracture mechanics begins with an idea from an early 20th century British aeronautical engineer. Alan Arnold Griffith came up with concept of fracture energy. He realized "that the weakening of material by a crack could be treated as an equilibrium problem in which the reduction in strain energy of a body containing a crack, when the crack propagates, could be equated to the increase in surface energy due to the increase in surface area." (Ceriolo 1998) Griffith's equation, $\sigma^2 = \frac{2\gamma E}{\pi a}$, relates the applied stress ($\sigma$), Young's modulus (E), crack length (a), and surface energy connected with traction-free crack surfaces ($2\gamma$). Later in 1957, George R. Irwin, introduced the line crack. This idea expanded on Griffith's approach which lacked the inclusion of the friction between crack surfaces.
The stress intensity factor approach (Irwin 1957), states that the stress field around a sharp crack in a linear-elastic material can be uniquely described by a parameter called the stress intensity factor, \( K \). Furthermore, fracture occurs when the value of \( K \) exceeds a critical value, \( K_c \), where \( K_c \) referred to as the fracture toughness, is a material property. Irwin (1957) gave the following stress function solutions for regions close to the crack tip:

\[
\sigma_{ij} = \left( \frac{K}{\sqrt{2\pi r}} \right) f_{ij}(\theta) \quad (2.1)
\]

where \( \sigma_{ij} \) are the components of the stress tensor at a point, and \( r \) and \( \theta \) are the polar coordinates, \( f_{ij} \) are the overall stress components, and \( K \) is the stress intensity factor. \( K \) is a measure of the magnitude of the stress intensity near the crack and is a function of the applied load and geometry of the structure. It can be seen however that as \( r \) approaches zero, \( \sigma \) approaches infinity. Therefore \( K \) was suggested by Irwin to be used as the fracture criterion. In order to address the interface of two materials, two parameters, \( \beta \) and \( \alpha \), were developed to help express the strain.

\[
\beta = \frac{\mu_1(1-2v_2)-\mu_2(1-2v_1)}{\mu_1(1-2v_2)+\mu_2(1-2v_1)} \quad (2.2)
\]

\[
\alpha = \frac{(E_1-E_2)}{(E_1+E_2)} \quad (2.3)
\]

\[
E = \frac{2\mu}{1-\nu} \quad (2.4)
\]

and

\[
\varepsilon = \left( \frac{1}{2\pi} \right) \ln \left( \frac{1-\beta}{1+\beta} \right) \quad (2.5)
\]

where \( E, \mu, \) and \( \nu \) are Young’s modulus, the shear modulus, and Poisson’s ratio respectively. The subscripts refer to the materials 1 and 2. A relationship between mode
I and II loading has been developed to define the combined interfacial stress intensity factor;

\[ K_{ic} = \sqrt{K_{1c}^2 + K_{2c}^2} \]  

(2.6)

where,

\[ K_{1c} = \sigma_0 \left[ \frac{\sqrt{2\pi}(\cos(\varepsilon \ln (2a)) + 2\varepsilon \sin((\varepsilon \ln(2a)))}{\cosh(\pi \varepsilon)} \right] \sqrt{a} \]  

(2.7)

\[ K_{2c} = \sigma_0 \left[ \frac{-\sqrt{2\pi}(\sin(\varepsilon \ln (2a)) - 2\varepsilon \cos((\varepsilon \ln(2a)))}{\cosh(\pi \varepsilon)} \right] \sqrt{a} \]  

(2.8)

with 'a' being defined as the crack length. The above relationships are very cumbersome and most often not used in actual engineering applications. Luckily, there is another approach.

The energy criterion is based on the research of Griffith. It states that when energy released by the growth of the crack is sufficient to supply the energy required to create new free surfaces, fracture occurs. This energy criterion involves a parameter usually referred to as the fracture energy or the critical strain energy release rate, \( G_f \).

The physical meaning of the strain energy release rate, \( G \), is the amount of energy that would be released if the crack advances a unit length. When this value is greater than the critical strain energy release rate, \( G_f \), crack growth would occur. \( G \) is derived through a series of advancements on the law of conservation of energy.

Considering static loading, the energy balance can be expressed as follows (Anderson 2004)

\[ W_d = U + U_p + \Gamma \]  

(2.9)
where $W_d$ is the potential energy of the externally applied load, $U$ is the elastic strain energy, $U_p$ is the plastic strain energy, and $\Gamma$ is the macroscopic work of fracture, which is the energy consumed in increasing the crack area. The energy criterion is represented by,

$$\frac{\delta(W_d-U)}{\delta A} \geq \frac{\delta \Gamma}{\delta A} = \gamma_s \frac{\delta A}{\delta a}$$

(2.10)

where $\gamma_s$ is the surface free energy and $\delta A$ is the increase in surface area associated with an increment of crack growth of $\delta a$.

The equation for the energy criterion for a quasi-static crack propagation in a lamina of thickness, $b$, is;

$$\frac{1}{b} \left| \frac{\delta(W_d-U)}{\delta a} \right| \geq 2\gamma_s$$

(2.11)

or

$$G \geq G_f$$

(2.12)

The surface free energy must be multiplied by two because there are two faces created when a crack forms. The equation for $G$ can also be written as;

$$G = \frac{-\delta \Pi}{\delta A}$$

(2.13)

where $\Pi = U - W_d$ is the energy potential of the system. Figure 2.4 represents the potential energy for (a) linear elastic behaviors and (b) nonlinear elastic behavior. Notice the area above the line represents the energy potential of the system.
2.4 Theoretical Background of Cohesive Zone Model (CZM)

Dugdale (1960) was the first to introduce the cohesive zone concept. He used the cohesive zone to model stress behavior near the crack. Dugdale assumed that the stress acts uniformly across the plastic zone. He stated that this stress was the same as the yield value of the material. Barenblatt (1962) used a similar approach as that of Dugdale. However, his model assumed that there was a variable stress present across the cohesive zone as a function of the cohesive crack length.

The cohesive zone model (CZM) was first introduced to concrete fracture by Arne in the late 1970s under the name of fictitious crack model (Hillerborg et al. 1976). The developed model was implemented with a finite element method to study the fracture behavior of an unreinforced concrete beam in flexure.
The fundamental idea of the CZM in FRP bonded concrete can be described in Figure 2.6. Compared with the single-parameter fracture approach of linear elastic fracture mechanics, which disregards anything that occurs within the damage zone, the CZM takes the behavior of the fracture processing zone into consideration and provides a way to characterize and model the failure process.
Before reaching the maximum stress, the stress increases with the increase of separation. Right after the maximum stress, the damaged cohesive zone is assumed to occur. Its formation is due to the micro cracking in the fracture process zone which softens the materials at this location. The stress-separation, $\sigma-\delta$, relation at the fracture process zone is used to describe this softening relation (Ouyang 2007).

![Figure 2.7 Exponential form of separation-stress law (Rose et al. 1983).](image)

The constitutive relationship that is developed using the CZM, is between the traction, $\sigma$, which acts on the interface corresponding with interfacial separation, $\delta$. Many studies have been carried out to investigate the $\sigma-\delta$ law. The stress-separation law which Ouyang (2007) used, and what was also used in this thesis, was based on the fundamental calculations of Rose et al. (1983). Rose et al. (1983) suggested a universal exponential form of the normal traction versus normal separation relation, as seen in Figure 2.7. In this figure, $\sigma_{\text{max}}$ is the maximum normal traction at the interface, $\overline{\delta_n}$ is the normal characteristic separation length where the maximum normal traction is located,
and $G_f$ is the fracture energy of separation. As the interface separates, the magnitude of $\sigma_n$ increases to a maximum value and then falls to zero as complete separation occurs. Through this relationship, the fracture energy can be derived as (Ouyang 2007):

$$G_f = e \sigma_{\text{max}} \delta_n$$

(2.14)

where $e = 2.71828$.

2.5 Debonding Behaviors Studies using Experimental Tests

As stated before, numerous experimental tests have been created to evaluate the bond between FRP and concrete. Some test critical interfacial stress and the others test critical interfacial fracture energy for debonding failure. For testing fracture energy, although each test is unique in its own way, they all are based on the same principle. By combining measurements of crack length and critical load at which the crack propagates, a value for the critical energy release rate, $G_f$, can be found. Following are brief introductions of some of these methods.

*Uniaxial Tension Test by Mullins et al. (1998)*

In this test, a metal disk is epoxied to the FRP surface, which is bonded to a concrete substrate. After the epoxy has been cured, the force required to pull the metal disk and the bonded material from the concrete surface is measured (Mullins et al., 1998).
Figure 2.8 Uniaxial tension test (Wan 2002).

The bond stress was calculated by the equation,

\[ f_{bt} = \frac{P}{A} \]  \hspace{1cm} (2.15)

where \( f_{bt} \) is the uniaxial tensile bond stress, \( P \) is the applied tensile force, and \( A \) is the area of the metal disk. Mullins et al. (1998) concluded that tensile failure will occur in the concrete substrate due to the tensile bond strength of the epoxy being greater than the tensile strength of the concrete.

**Bond Strength Based on Crack Spacing (Neubauer and Rostasy 2001)**

For this study, Neubauer and Rostasy (2001) used experimental data from four point bending tests to develop a consistent bond model and design a concept for bond integrity of bonded CFRP-plates.
Using fracture mechanics, the force at crack B that initiates plate debonding can be expressed as:

\[
F_{IB,DB} = T_{\text{max}} \cdot \frac{\tanh(\omega s_{cr})}{1 - \frac{1}{\cosh(\omega s_{cr})}\left(k_P + 1\right)}
\]  \hspace{1cm} (2.16)

where \( T_{\text{max}} \) is the maximum ultimate bond force of a pure bond specimen and is calculated by:

\[
T_{\text{max},k} = 0.5 * b_1 * k_p * \sqrt{E_1 t_1 f_{ctm}}
\]  \hspace{1cm} (2.18)

where \( b_1 \) is the width, \( E_1 \) is the Young's modulus, and \( t_1 \) is the thickness of the FRP plate. Also \( f_{ctm} \) is the mean tensile strength of the concrete substrate. The variable \( k_p \) is known.
as the geometry factor, accounting for the ratio of plate width to concrete width ranging from 1-1.3. Furthermore $\omega$ is defined as;

$$\omega = \sqrt{\frac{\mu_{fcm}}{E_1 t_1}}$$

(2.19)

and

$$K_F = \frac{\Delta F_t}{F_{La}}$$

(2.20)

This is one of the first attempts to predict the failure load of the bond using strictly geometric and material properties of the system.

**Peel Test (Karbhari and Engineer 1996)**

Karbhari and Engineer (1996) developed a peel test for investigation of the bond between composites and concrete. A spring loaded linear variable differential transformer recorded the actuator movement and the peel force was directly measured by a load cell. The biggest advantage of the peel test is that this technique can test the interface failure under a mixed mode of loading, thereby representing the failure mode under service conditions. The peel test was unique because it was one of the first tests to incorporate mixed mode loading. The delamination of the FRP can be characterized by $G$, the interfacial energy release rate, and $\psi$, the loading phase angle. $\psi$ is the ratio of the shear to normal stress at the crack tip.
The following equations were derived from this test:

\[ G = \frac{P(1+\varepsilon - \cos(\alpha))}{w} - U_t \]  
(2.21)

\[ U_t = \frac{1}{2E} \left( \frac{P}{wt} \right)^2 \]  
(2.22)

\[ \varepsilon = \frac{P}{wtE} \]  
(2.23)

\[ \psi = \tan^{-1} \left[ \frac{1-(1+\varepsilon)\cos(\alpha)}{(1+\varepsilon)\sin(\alpha)} \right] \]  
(2.24)

Where \( G \) is the interfacial fracture energy, \( P \) is the peel force applied to the peel specimen, \( \alpha \) is the angle from the direction of the peel force to the substrate, \( \varepsilon \) is the strain in the peel arm, \( t \) is the thickness of the peel strip, \( w \) is the width of the peel strip, \( E \) is the elastic modulus of the strip, and \( U_t \) is the strain energy stored in the peel arm. Using \( \psi \), \( G \) can be decomposed into Mode I and II interfacial fracture energy \( G_I \) and \( G_{II} \) as
\[ G = G_I + G_{II} \]  

(2.25)

where \[ \tan^2\psi = \frac{G_{II}}{G_I} \]  

(2.26)

**Double Cantilever Beam (DCB) Test**

The Double Cantilever Beam (DCB) test shown in Figure 2.11 was not a new test developed for FRP/substrate interfaces. It originated as an established method for determining Mode I fracture toughness and energy release rate of adhesive bonds between symmetric metallic plates, defined by ASTM D 3433-93. ASTM D 5528-94a modified the procedure so it would be applicable to composite materials. The load, \( P \), load-point deflection, \( u \), and crack length, \( a \), are measured and used to calculate the energy release rate. The main problem with the DCB method is that it requires both arms to be identical or almost identical. This prevents DCB method from being used directly for FRP bonded concrete specimens. However, Giurgiu et al. (2001) extended the principles and practices of the ASTM standards of 3433-93 and 5528-94a to create the Modified Double Cantilever Beam (MDCB) method.

![Figure 2.11 Double Cantilever Beam Specimen.](image)
**Modified Double Cantilever Beam (MDCB) Test (Wan et al. 2004)**

Similar to the peel test, the MDCB also contains mixed mode loading. Unlike the DCB test, the specimens used in the MDCB test do not consist of two identical beams. Instead, one concrete beam with FRP overlay is used. A schematic for the test set up of a MDCB is shown in Figure 2.12.

![Figure 2.12 Schematic for MDCB method (Wan et al. 2004).](image)

The test results of crack tip opening displacement (CTOD) indicated that the crack growth was predominantly controlled by displacement in the mode I loading (opening) direction, with a relatively small Mode II component (in-plane shear) (Wan et al. 2004). Adapted from the DCB test, the MDCB must account for a rotational center ahead of the crack tip. Therefore $\Delta r$, an end correction length, is added to the measured crack length, $a_{exp}$, meaning

$$E_{I} = \frac{P_{I} \Delta}{a_{exp} + \Delta r}$$

(2.27)

The strain energy release rate under Mode I loading for a MDCB can be expressed as
Giurgiuțiu et al. (2001) applied the compliance method to calculate the effective flexural stiffness $E_{11}I$ and an experimental value for the end correction length, $\Delta r$. Mode I compliance is defined as:

$$C = \frac{\Delta_l}{P_l}$$  \hspace{1cm} (2.29)

where $\Delta_l$ is the displacement of the load point in the direction of Mode I loading. $C$ is then used in conjunction with $a_{\exp}$ to create the following graph of a least squares fit of the cubic root compliance, where

$$C^{1/3} = m a_{\exp} + n$$  \hspace{1cm} (2.30)

![Figure 2.13 C^{1/3} vs. a_{\exp} (Wan 2002).](image)

The slope is defined as

$$m = \left(\frac{1}{3E_{11}}\right)^{1/3}$$  \hspace{1cm} (2.31)

hence

$$E_{11}I = \frac{1}{3m^3}$$  \hspace{1cm} (2.32)

the end correction length is now defined as
Due to geometric nonlinearity, the crack length must be reduced to the effective crack length (Wan 2002);

\[ a_e = a - \Delta_s \]  
(2.34)

where \( \Delta_s \) is the shortening of the crack length due to the effect of large deflections.

\[ \Delta_s = \frac{1}{15} \left( \frac{P la^2}{E_{11} I} \right)^2 a \]  
(2.35)

These equations can be substituted into Equation 2.28 yielding;

\[ G_I = \frac{3m^2 P_I^2 (a_{exp} + \frac{n}{m} - \Delta_s)^2}{2B} \]  
(2.36)

This is only the fracture strain energy generated by Mode I loading. If you recall, from the peel test discussion, \( G = G_I + G_{II} \) the strain energy generated by Mode II loading is represented by load \( P_{II} \) and displacement in the direction of Mode II, \( \Delta_{II} \);

\[ \Delta_{II} = \frac{P_{II} a}{E_{II} A_c} \]  
(2.37)

where \( A_c \) is the cross-sectional area of the FRP overlay. The energy release rate for Mode II loading can be expressed as:

\[ G_{II} = \frac{P_{II}^2}{2E_{11}hB^2} \]  
(2.38)

or \[ G_{II} = \frac{P_{II}\Delta_{II}}{2ab} \]  
(2.39)

The compliance method is also used to calculate the effective axial stiffness, \( E_{11} A_c \) and an estimated value for end correction length, \( \Delta_{III} \). \( C_{II} \) is written as:

\[ C_{II} = \frac{\Delta_{II}}{P_{II}} \]  
(2.40)
where a least squares fit of the compliance plot as a function of the crack length measured from the experimental value, $a_{\text{exp}}$, is expressed as:

$$C_{II} = sa_{\text{exp}} + t$$

where, $s$ is the slope of the line defined as:

$$s = \left( \frac{1}{E_{xx}hB} \right)$$  \hspace{1cm} (2.42)

and $t$ is the intercept with the C-axis. The end correction length, $\Delta_a$, is

$$\Delta_a = \frac{t}{s}$$  \hspace{1cm} (2.43)

The Mode II energy release rate is then expressed as;

$$G_{II} = \frac{P_{II} \Delta_{II}}{2(a + t/s)B}$$

For the mixed Mode I and II loading, the total energy release rate $G$ is calculated as:

$$G = G_I + G_{II}$$

(2.45)
This equation is derived from conservation of energy by Anderson (2004). The MDCB is a very useful experiment that allows for the determination of both Mode I and II strain energies and overall strength of the bond interface.

MDCB tests (Ouyang 2007)

Ouyang (2007) studied the effects of moisture on the bond durability between the FRP and concrete. One of the main contributions of this study was the development of the concept of the ‘residual thickness of concrete’ (RTC). After completing the MDCB test, the detached plates were measured with a digital coordinate measuring machine (CMM), as shown in the figure below (Figure 2.15a).

![Figure 2.15 Measuring the RTC (Ouyang and Wan 2008).](image)

The CMM measured the average thickness of the residual concrete still attached to the FRP, the adhesive and FRP plate. A high resolution digital microscope was used to measure the average thickness of the adhesive at 5 mm intervals along the longitudinal direction of the plate. The adhesive thickness was taken as the average value of those measurements. The difference between the total average thickness from CMM and the
sum of CFRP plate thickness and adhesive thickness was taken as the average residual thickness of concrete, as shown in Figure 2.15b.

\[ RTC = h_T - h_a - h_f \]  

(2.46)

where \( h_T \) was the average total thickness of debonded strip, \( h_a \) was the average thickness of the adhesive layer, and \( h_f \) was the average thickness of CFRP. In dry conditions, the RTC was about 2 mm (Ouyang and Wan 2008).

Coupled with cohesive zone modeling, (CZM), Ouyang and Wan (2008) utilized this value of RTC to help model the experiment in a finite element analysis program. Ouyang and Wan (2008) determined that the interface region relative humidity (IRRH) and RTC are inversely proportional. So for small values of RTC and high values of IRRH, there were lower values for fracture energy recorded.

2.6 Debonding Behavior Studies using Finite Element Analysis

From the previous studies, it had become obvious that the primary area of concern for FRP plate debonding was the cohesive failure within the concrete substrate. In order to better understand the debonding failure, finite element programs were used to create numerical models. A brief introduction to some of these studies follows.

Coronado and Lopez (2005)

After determining the mechanical properties governing the debonding failure through a series of tests, Coronado and Lopez (2005) used the obtained values to develop a finite element model using ABAQUS that represented a direct pull off bond test. Using plastic-damage model and tensile softening curve to model the CFRP/concrete interface, the finite element model used in this study predicted, the strain distributions, failure load
and the failure mechanisms of the single shear lap test. Depicted below in Figure 2.16 is the concept behind the plastic-damage model. Before microcracking occurs, the beam's behavior is virtually elastic and no damage has taken place. Once the interfacial stresses reach a threshold value, microcracks start to form close to the concrete-epoxy interface. At this point, the damage of the interface starts to increase. Under increased loading, the microcracks come together to form macrocracks, which lead to debonding.

**Figure 2.16** Damage of the concrete-epoxy interface during debonding failure (Coronado and Lopez 2005).

*Ouyang and Wan (2008)*  
Using the finite element analysis program ANSYS, Ouyang and Wan(2008) developed a model for MDCB specimens subjected to Mode I loading. By using the cohesive zone to model the CFRP/concrete interface, they were able to establish a valid relationship between interface region relative humidity (IRRH) and the fracture energy of FRP bonded concrete specimens based on the proposed constitutive relation between the residual thickness of concrete (RTC) and the IRRH.
In this study, a four point bending test was used to investigate the parameters that may delay the intermediate crack debonding of the bottom CFRP laminate, and increase the load carrying capacity and CFRP strength utilization ratio, $\beta_w$.

$$\beta_w = \sqrt[2.25-b_f/b_c]{1.25+b_f/b_c}$$  \hspace{0.5cm} (2.47)

where, $b_f$ is the width of the FRP and $b_c$ is the width of the concrete. Using this ratio, Kotynia developed this equation for the fracture energy:

$$G_f = 0.308\beta_w^2\sqrt{f_t}$$  \hspace{0.5cm} (2.48)

A numerical analysis using an incremental nonlinear displacement-controlled 3D finite-element model was developed to investigate the flexural and CFRP/concrete...
interfacial responses of the tested beams. A summary of the Kotynia et al. (2008) model can be seen in Figure 2.17.

**Obaidat et al. (2009)**

Like Kotynia et al. (2008), Obaidat et al. (2009) also used a four point bending test and ABAQUS in their study. The main objective of Obaidat et al.’s study (2009) was to verify that the use of a cohesive zone to model the bond between CFRP and concrete is acceptable. Two different models for the bond were used to show the advantages of the CZM, one with a perfect bond at the interface, and one with a cohesive zone for the interface. Comparing the CZM to the perfect bond model, it was easy to see that the correct failure and fracture of the beam were not achieved in the numerical model if CZM were not used. Obaidat et al. (2009) also proved that the ultimate load increased with the increased length of CFRP.

**Coronado and Lopez (2010)**

Like before, Coronado and Lopez (2010) used damaged band or crack band to model CFRP/concrete interface. Similar to what Obaidat et al. (2009) had done, they also made two models, one with a crack band and one without. And again it was shown that a more accurate prediction of the failure load for the specimens that failed due to debonding was achieved using the crack band. Numerical simulations of RC beams strengthened with FRP were able to predict the load-deflection response, failure mechanisms, post failure behavior, and strain distributions. Multiple loading conditions were tested in order to verify the wide range of application of the crack band concept.
The two tests used to simulate the different loading conditions were the single shear-lap test and the beam tests. Figure 2.18 is a good representation of their work, comparing the finite model to the actual experiment.

![Figure 2.18 Predicted and observed damage distributions after failure of single shear-lap test (Coronado and Lopez 2010).](image)

**Huang and Lyons (2005)**

Huang and Lyons used ABAQUS 6.2 to perform a parametric study of the effects of material properties and dimensions on FRP debonding from concrete. However, unlike the previous experiments which used either a crack band or CZM approach, they used a different method to derive the strain energy release rate. The strain energy release rate was calculated using the domain integral method of 'virtual crack extension'. The domain integral method was used in the finite element model to calculate the J-integral. If the material is linear elastic, the J-integral equals the strain energy release rate $G$. 
2.7 Summary

Previous research shows that the primary mode of debonding is cohesive failure within the concrete substrate. Fracture mechanics was used to develop the concept of the cohesive zone model. For externally FRP reinforced concrete beams constructed in normal conditions, cohesive failure was observed to occur a distance of about 2 mm from the adhesive/concrete interface. It is at this location that cohesive elements will be created in the numerical model for this study. The cohesive elements will represent the crack which will propagate in the numerical model.
Chapter 3 Finite Element Model Calibration

In this research, the goal was to develop a finite element (FE) model to simulate the MDCB test and to perform parametric studies to find the key parameter which controls the debonding behavior of MDCB specimens. The present chapter outlines the development and calibration of the FE model. The ANSYS finite element analysis software (ANSYS 2009) was used for all finite element modeling in this thesis. The model was created with the use of 2-dimensional plane elements to create the concrete block, adhesive, and CFRP. A cohesive zone model was used to model the epoxy/concrete bond interface. All results presented in this thesis that come from the finite element model are “nodal solutions”, meaning the results came from the model nodes.

3.1 Element Types

Plane82 Elements (Figure 3.1) are 2-D elements that can be used for solid structure modeling. Most commonly, this element is used as a plane element (plane stress or plane strain). The element is defined by eight nodes having two degrees of freedom at each node: translations in the nodal x and y directions. In addition, the element has plasticity, creep, swelling, stress stiffening, large deflection, and large strain capabilities (ANSYS 2009). The Plane82 element also has the capability to model multi-elastic, or nonlinear elastic materials. This allows for a multi-linear isotropic stress-strain curve to represent the concrete.
Figure 3.1 Plane82 element (ANSYS 2009).

Inter203 Elements (Figure 3.2) is a 2-D 6-node quadratic interface element used for the 2-D modeling of structural assemblies. Each node has two degrees of freedom, translations in the nodal x and y directions. When used with 2-D quadratic structural elements (Plane82 and Plane183) Inter203 elements simulate the interface surfaces and the subsequent delamination process. Plane82 elements were used in this model due to the element's ability to be modeled with nonlinear material properties. The separation between the surfaces is represented by an increasing displacement between nodes, within the interface element itself, which are initially coincident as shown in Figure 3.3. Because Inter203 element has this property, the cohesive zone model approach can be applied in ANSYS.
3.2 Finite Element Model

As stated before, the experiment being modeled in this finite element model is a MDCB test. The results are being compared to those obtained from Ouyang and Wan (2008). Their test set up is shown below in Figure 3.4.
The MDCB specimen that was modeled included a concrete substrate that was 76 x 76 x 191 mm. The width of CFRP plate was 51 mm and the thickness was 2 mm. The thickness of adhesive layer was 1.2 mm, which was the average value of the experimental specimens. The thickness and length of the hinge plate, which was made of aluminum alloy, were 1.8 mm and 20 mm, respectively. Because the failure happened as FRP debonding failure, the stress in FRP, adhesive and hinge plate would be relatively low and they were assumed to be linear elastic in the FE model. The material properties and geometry are summarized in Table 3.1.
Table 3.1 Material Properties and Geometry in Numerical Model

<table>
<thead>
<tr>
<th>Material</th>
<th>Elastic Modulus (GPa)</th>
<th>Poisson’s Ratio</th>
<th>Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Substrate</td>
<td>22</td>
<td>0.28</td>
<td>76</td>
</tr>
<tr>
<td>Epoxy Adhesive</td>
<td>3.18</td>
<td>0.34</td>
<td>1.2</td>
</tr>
<tr>
<td>CFRP Plate</td>
<td>139</td>
<td>0.20</td>
<td>2.0</td>
</tr>
<tr>
<td>Hinge Plate</td>
<td>70</td>
<td>0.30</td>
<td>1.8</td>
</tr>
</tbody>
</table>

Based on the residual thickness of concrete (RTC) research presented by Ouyang and Wan (2008), the appropriate location for the CZM could be determined. When the interface region relative humidity was between 45% and 55%, which was similar to the normal relative humidity value in a room, the average RTC was approximately 2 mm (Ouyang and Wan 2008). Therefore it is assumed that the crack propagation happens in concrete at 2 mm away from the adhesive/concrete interface. It is at this location of 2 mm away from the concrete/epoxy interface, that the interface element is created to represent the cohesive zone.

3.2.1 Cohesive Element Properties

In this model, the crack is assumed to have happened in the concrete 2 mm away from the epoxy/concrete interface as discussed in previous section. Since the model assumes the crack begins and propagates within the concrete, it would only make sense to use concrete fracture energy for the interface elements. Ouyang and Wan (2008) did not test the concrete fracture energy in their experiment, although the overall interfacial fracture energy was tested. The fracture energy reported by Ouyang and Wan (2008) includes the energy stored in FRP, epoxy and concrete. In the model of present research, the fracture energy of concrete itself is needed.
In a study conducted by Wittman (2002), different fracture energies were determined for different concrete strengths. Those strengths and calculated fracture energies can be found in Table 3.2.

Table 3.2 Concrete Strength and Fracture Energy (Wittman 2002)

<table>
<thead>
<tr>
<th>Concrete Type</th>
<th>A</th>
<th>B</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength (ksi)</td>
<td>5.1</td>
<td>8.3</td>
<td>16.3</td>
</tr>
<tr>
<td>$G_f$ (N/m)</td>
<td>123.55</td>
<td>152.05</td>
<td>158.7</td>
</tr>
</tbody>
</table>

Comparing the concrete strength used in this experiment with those in Wittman's experiment, it can be concluded that the fracture energy for 3.1 ksi concrete should be less than 123 N/m. If we used the data for concrete A and B to extrapolate the $G_f$ for the concrete used in this research, $G_f$ for 3.1 ksi concrete would be around 100 N/m. Therefore, it was assumed that the fracture energy of the concrete was 100 N/m in the FE model. This value was proved to be appropriate during calibration.

From previous research (Wan et al. 2004), it was seen that the Crack Tip Opening Displacement (CTOD) of the FRP debonding crack varied from roughly 0.03 mm to 0.08 mm. This means that $\overline{\delta_n}$, the maximum normal characteristic separation in the CZM, is between 0.03 and 0.08 mm. A value of 0.06 mm was chosen in this research and then plugged into the exponential stress-displacement law along with the fracture energy to determine the maximum stress.

$$G_f = e\sigma_{max}\overline{\delta_n}$$  \hspace{1cm} (3.1)

Equation 3.1 yielded a value of 0.6 MPa for $\sigma_{max}$. 
3.2.2 Mesh Sensitivity

An acceptable mesh size for model was determined using an initial model using element size of 3 mm for the cohesive elements was run. The model was then re-meshed using a length of 1 mm for the elements. Likewise the model was re-meshed using element lengths of 0.5 mm, 0.25mm and 0.1mm. The comparison of results using different element sizes is shown in Table 3.3. The rest of the defined mesh sizes can be found in Appendix A.

<table>
<thead>
<tr>
<th>Element Size (mm)</th>
<th>Maximum Load (N)</th>
<th>Displacement at Maximum Load (mm)</th>
<th>Trial Duration (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>472.397751</td>
<td>1.035625</td>
<td>8</td>
</tr>
<tr>
<td>1</td>
<td>472.913373</td>
<td>1.195</td>
<td>10</td>
</tr>
<tr>
<td>0.5</td>
<td>472.935624</td>
<td>1.195</td>
<td>13</td>
</tr>
<tr>
<td>0.25</td>
<td>472.943743</td>
<td>1.195</td>
<td>25</td>
</tr>
<tr>
<td>0.1</td>
<td>472.901167</td>
<td>1.195</td>
<td>38</td>
</tr>
</tbody>
</table>

By changing the size of the interface elements used in the model, the optimal mesh size was achieved. It was deemed most appropriate to use the 0.5 mm element size due to its converged value and trial duration. It also provided a better estimation for crack propagation due to smaller elements compared to the 1 mm sized model. Using a 0.5 mm element size, a total of 687 cohesive elements were created. Depicted below in Figures 3.5 and 3.6 are the undeformed meshed model as well as an exploded view of the cohesive zone elements respectively.
Figure 3.5 Undeformed model with 0.5 mm size elements.

Figure 3.6 Expanded view of cohesive zone elements.
3.2.3 Linear Elastic vs. Nonlinear Concrete

As previously stated, Plane82 elements have the capability to be modeled as a multi-linear, or nonlinear material. In order to show the importance of modeling the concrete as a nonlinear material, it was necessary to run both linear and nonlinear models. The following discusses the background of multi-linear material property and the stress-strain curve for concrete used in this model.

The Multi-linear isotropic material uses the von Mises failure criterion along with the Willam and Warnke (1974) model to define the failure of the concrete (Wolanski 2004). Using the following equations to compute the multi-linear isotropic stress-strain curve for the concrete, the compressive uniaxial stress-strain relationship is acquired (MacGregor 1992).

\[
f = \frac{E_c \varepsilon}{1 + (\frac{\varepsilon}{\varepsilon_0})^2} \tag{3.2}
\]

\[
\varepsilon_0 = \frac{2f'_c}{E_c} \tag{3.3}
\]

\[
E_c = \frac{f}{\varepsilon} \tag{3.4}
\]

where: \(f \text{ [psi]} \) is the stress at any strain \(\varepsilon\) and \(\varepsilon_0\) is the strain at the ultimate compressive strength \(f'_c\). The multi-linear isotropic stress-strain used requires that the first point of the curve be defined by the user. Young’s Modulus can be calculated using the following equation:

\[
E_c = 57000\sqrt{f'_c} \tag{3.5}
\]
Although more accurate equations relating Young's modulus and the compressive strength exist, the Young's modulus was the only known material property of the concrete making this the ideal equation. Also, it is assumed that the concrete below the RTC will not crack.

![Uniaxial Stress-Strain curve](image)

Figure 3.7 Uniaxial Stress-Strain curve (Wolanski 2004).

Figure 3.7 shows the stress-strain relationship used for this study and is taken from Wolanski (2004). Point 1 is defined by the user. The strain for point one is computed for a stress value of 30% $f'_c$ and uses equation 3.4 (linear range). Points 2, 3, and 4 are calculated using $\varepsilon_0$ (Equation 3.4) and Equation 3.3. Strains were selected and the stress was calculated for each point. Point 5 is the point of ultimate compressive strength and strain. The strains and stresses used in this model are listed in Table 3.4.
Table 3.4 Uniaxial Stress and Strain Values

<table>
<thead>
<tr>
<th>Point</th>
<th>Strain</th>
<th>Stress [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.000295</td>
<td>6.48</td>
</tr>
<tr>
<td>2</td>
<td>0.0005</td>
<td>10.3</td>
</tr>
<tr>
<td>3</td>
<td>0.0008</td>
<td>15.1</td>
</tr>
<tr>
<td>4</td>
<td>0.0012</td>
<td>19.2</td>
</tr>
<tr>
<td>5</td>
<td>0.001964</td>
<td>21.6</td>
</tr>
</tbody>
</table>

Figure 3.8 Comparison of results using linear and nonlinear concrete properties.

Figure 3.8 shows the difference of load versus displacement curves between linear and nonlinear modeling. The difference between the two different modeling methods is clear. The linear model yields a maximum load of only 383 N and the corresponding displacement is 1.44 mm. The multi-linear model yields 473 N and 1.195 mm. Compared to the values obtained from multi-linear model, the linear model underestimates the load capacity of the bond by roughly 19%. The multi-linear curve for concrete can also help with convergence of the nonlinear algorithm for this model.
This result is due to the way the concrete attracts load. The linear model assumes that the acting stress is proportional to the strain for all strains until the ultimate compressive strength is reached. Looking at Figure 3.7, it can be seen that the stress value for a linear model at 0.001 strain is approximately 4150 psi, while the multi linear model has a stress value of only 3300 psi. Since the linear model attracts a higher load, it transmits the higher stresses to the CZM element causing it to reach the allowable maximum stress earlier.

3.3 Calibration of the Model

The bottom of the specimens was bolted to the base of the test frame by using two bolts during experimental tests. Therefore, the base of the model had all degrees of freedom restricted to zero displacement. Because it was displacement control in the experimental test, the load in the FE model was applied by using a displacement boundary condition of 8 mm of the node depicted below in Figure 3.9.

The solution constraints used in this model consist of limiting the number of sub-steps and equilibrium iterations to 400 and 1200 respectively. Also, nonlinear geometry must be turned on. With these constraints in place, a solution can be derived.

The values chosen for the cohesive interface elements were from a range of values determined from previous studies. In order to find the most appropriate combination, the effects that the cohesive interface elements have on the load deflection response curves must be known. Therefore, a series of trials were run to determine the effects caused by changing the properties of the fracture energy equation which represents the cohesive interface elements. It was through these tests that the most
appropriate combination of stress-displacement values were selected. A more detailed explanation of the effects can be found in the next chapter. The deformed shape for the loading conditions is shown in Figure 3.10 and the load versus deflection results for the specimen is shown in Figure 3.11.

![Figure 3.9 Finite element geometry and loading configuration.](image)

The load versus deflection curve generated in the experimental test is shown in Figure 3.11. The curve for the control specimen is the one which the FE model presented in this thesis tried to simulate. By comparing the curves in Figures 3.11, it can be found that the numerical results has a higher maximum peel load that occurs a smaller displacement. However, this can be expected due to the perfect nature of the bond in the finite element analysis program. The overall shape of the load-displacement curve from the numerical analysis is in good agreement with the shape generated by the physical experimental data.
Figure 3.10 Final deflected shape of the model.

Figure 3.11 Comparison of numerical and experimental load vs. displacement curves.
3.4 Summary

In this chapter, the background for the finite elements model developed in this thesis is provided. The geometric and material properties used in the FE model are then defined. How the cohesive zone properties were selected is then covered. Also presented in this chapter is the importance of modeling concrete as a nonlinear material instead of linear elastic material. Finally the model is calibrated by experimental data.
Chapter 4 Parametric Study

This chapter presents the results obtained from the parametric study. The two main categories taken into consideration are the effect caused by changing the model geometry, and the effect caused by changing Young's moduli values for the materials. By examining the effects caused by adjusting the physical characteristics of the model, a better understanding of how each component of the model contributes to the system can be gained. The geometry components to be examined, on an individual case by case basis, are the thicknesses of the FRP, the adhesive, and the RTC. The adjusted Young's moduli to be modeled will be that of the FRP, the adhesive, and the concrete. Another part of the model that was taken into consideration was the cohesive zone elements. The model's bond is represented by these elements, so by adjusting the components, maximum stress and its corresponding displacement, that make up the cohesive element the models bond will respond differently. Finally, the effects of Mode II loading are taken into consideration. All of the control values for the parametric study can be found in Table 4.1. An "*" will denote the control value in all charts and graphs from now on.
Table 4.1 Parametric Control Values

<table>
<thead>
<tr>
<th>Material Properties</th>
<th>Young's Modulus [GPa]</th>
<th>Poisson's Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>22</td>
<td>.28</td>
</tr>
<tr>
<td>Adhesive</td>
<td>3.18</td>
<td>.34</td>
</tr>
<tr>
<td>FRP</td>
<td>139</td>
<td>.2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Material Thickness [mm]</th>
<th>Residual Thickness of Concrete</th>
<th>2.0</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Adhesive</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td>FRP</td>
<td>2.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Cohesive Interface Element Properties</th>
<th>Maximum Stress [MPa]</th>
<th>0.6</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Maximum Displacement [mm]</td>
<td>0.0613</td>
</tr>
</tbody>
</table>

4.1 Effect of Model Geometry

Listed in Table 4.2 are the three series of thickness trials conducted.

Table 4.2 Thickness Series

<table>
<thead>
<tr>
<th>FRP [mm]</th>
<th>Adhesive [mm]</th>
<th>RTC [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>1.5</td>
<td>1.2 *</td>
<td>1.5</td>
</tr>
<tr>
<td>2.0 *</td>
<td>2.0</td>
<td>2.0 *</td>
</tr>
<tr>
<td>2.5</td>
<td>2.5</td>
<td>3.0</td>
</tr>
<tr>
<td>3.0</td>
<td>3.0</td>
<td></td>
</tr>
</tbody>
</table>

4.1.1 FRP Thickness

Six different FRP thicknesses were modeled: 3 mm, 2.5 mm, 2.0 mm (Control), 1.5 mm, 1.0 mm, and 0.5 mm. The effects of the FRP thickness on maximum peel load can be found in Figure 4.1. It can be found in this figure that the maximum peel load, \( P_{\text{max}} \), increases with the increase of FRP thickness for the same applied displacement.
This conclusion is confirmed by Huang and Lyons (2005). Their study proved that the energy release rate \( G \) increases with increasing composite overlay thickness for the same applied displacement (Huang and Lyons 2005). An increase in the load, \( P \), correlates through Equation 2.28, where \( M=P\times a \), to increase in fracture energy, \( G \).

The deflections at maximum load for different models are shown in Table 4.1. After the FRP debonded from concrete, the debonded part can be considered as a cantilever beam. The thinner the plate thickness, the more flexible the composite beam becomes. This is evident in the Figure 4.1 and Table 4.1.

After reaching the maximum load, the FRP starts to delaminate from concrete substrate. The model with 0.5 mm-thick FRP manages only a maximum peel load of 299 N. However, the model does not experience any initial crack propagation until the load displacement reaches a value of nearly 7 mm. On the other side of the spectrum, the thicker plates offer a higher capacity for peel load. Taking a look at the curves for the models with 3 and 2.5 mm-thick FRP in Figure 4.1, it can be seen that the loads go to zero after a certain displacement (5.3 mm of maximum displacement for 3 mm-thick FRP and 6.54 mm of maximum displacement for 2.5 mm-thick FRP). This means that complete delamination of the FRP plate has occurred. Although it may seem attractive to use the thicker material due to the increase in load capacity, one needs to be careful to note that complete delamination of the FRP occurs at a lower vertical displacement. This is evident when comparing the maximum load displacement at which delamination occurs in the models with 3 mm and 2.5 mm-thick FRP.
Figure 4.1 Numerical load vs. displacement curves for models with different FRP thicknesses.

Table 4.3 Critical Values for Different FRP Thicknesses

<table>
<thead>
<tr>
<th>Thickness [mm]</th>
<th>$P_{\text{max}}$ [N]</th>
<th>$\delta$ at $P_{\text{max}}$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>298.7705</td>
<td>6.936</td>
</tr>
<tr>
<td>1</td>
<td>355.4306</td>
<td>2.836</td>
</tr>
<tr>
<td>1.5</td>
<td>412.3943</td>
<td>1.636</td>
</tr>
<tr>
<td>2 *</td>
<td>472.9098</td>
<td>1.195</td>
</tr>
<tr>
<td>2.5</td>
<td>539.3065</td>
<td>0.836</td>
</tr>
<tr>
<td>3</td>
<td>605.5441</td>
<td>0.836</td>
</tr>
</tbody>
</table>

4.1.2 Epoxy Thickness

Five trials consisting of epoxy thicknesses of 3, 2, 1.2 (Control), 1.0, and 0.5 mm were tested. The effects of the epoxy thickness on maximum peel load can be found in Figure 4.2. Because the epoxy is a part of the debonded FRP composite, the effect of the
epoxy thickness is similar to that of the FRP. However, epoxy thickness effect is not as large as that of the FRP by comparing Figures 4.1 and 4.2.

The maximum peel load, \( P_{\text{max}} \), increases with the increase of the epoxy thickness for the same applied displacement. The thinner the adhesive thickness, the more flexible the composite beam becomes. The model with 0.5 mm-thick epoxy manages only a maximum peel load of 414 N compared with 640 N for a thickness of 3 mm. The thicker epoxies offer a higher capacity for peel load. These results coincide with what was found in previous research (Chen and Qiao 2009). It has been found that the size of the softening zone will increase, and as a result, the capacity of the FRP-concrete interface will be improved with the increase of the thickness of adhesive layer (Chen and Qiao 2009).

The deflections at maximum load for different models are shown in Table 4.2. By observing the data in Table 4.2 and Figure 4.2, it can be found that the deflections at maximum peel load do not change significantly with the change of epoxy thickness. This means that the crack starts to propagate at similar deflection although the peak load changes with the epoxy thickness.

Table 4.4 Critical Values for models with Different Epoxy Thicknesses

<table>
<thead>
<tr>
<th>Thickness [mm]</th>
<th>( P_{\text{max}} ) [N]</th>
<th>( \delta ) at ( P_{\text{max}} ) [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>413.892</td>
<td>1.09125</td>
</tr>
<tr>
<td>1</td>
<td>455.5815</td>
<td>1.195</td>
</tr>
<tr>
<td>1.2 *</td>
<td>472.9098</td>
<td>1.195</td>
</tr>
<tr>
<td>2</td>
<td>550.6629</td>
<td>1.043125</td>
</tr>
<tr>
<td>3</td>
<td>639.8355</td>
<td>0.89125</td>
</tr>
</tbody>
</table>
Taking a look at the curves for 3 and 2 mm thicknesses of epoxy in Figure 4.2, it can be seen that the loads go to zero after a certain displacement. This means that complete delamination of the FRP plate has occurred. Similar as discussed in the previous section, although it may seem attractive to use the thicker material due to the increase in load capacity, one needs to be careful to note that crack propagation for the stiffer model occurs at a faster rate. This is evident when comparing the maximum displacement at which delamination occurs in the models with 3 mm- and 2 mm-thick epoxy.
4.1.3 Residual Thickness of Concrete (RTC)

The bond between concrete and adhesive in dry conditions was very good. However, with the increases of duration in water, there was less and less residual concrete retained on the detached CFRP after debonding (tests Ouyang and Wan 2008). Ouyang and Wan (2008) found that the specimen with less residual concrete thickness (RTC) had lower fracture energy.

In order to study the effect of RTC, five trials consisting of RTC of 3, 2 (Control), 1.5, 1.0, and 0.5 mm were tested. The effects of adjusting the RTC on the maximum peel load can be found in Figure 4.3. The maximum peel load, $P_{\text{max}}$, increases with the increase of the RTC for the same applied displacement. Increasing the thickness of the residual concrete has an almost perfectly linear proportional effect on the maximum peel load (Figure 4.4). Stiffness also increases as the thickness of the RTC increases. Increased stiffness means complete delamination occurs at a lower vertical displacement. Similar to the effect of epoxy thickness, RTC does not have significant effect on the deflection at maximum peel load as seen in Table 4.3 and Figure 4.3.

<table>
<thead>
<tr>
<th>RTC [mm]</th>
<th>$P_{\text{max}}$ [N]</th>
<th>$\delta$ at $P_{\text{max}}$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>311</td>
<td>1.036</td>
</tr>
<tr>
<td>1</td>
<td>362</td>
<td>1.036</td>
</tr>
<tr>
<td>1.5</td>
<td>418</td>
<td>1.091</td>
</tr>
<tr>
<td>2 *</td>
<td>473</td>
<td>1.195</td>
</tr>
<tr>
<td>3</td>
<td>596</td>
<td>1.195</td>
</tr>
</tbody>
</table>
Figure 4.3 Numerical load vs. displacement curves for models with different RTC.

Figure 4.4 Comparison of RTC and $P_{max}$. 
4.2 Effect of Young's Modulus

Listed in Table 4.6 are the three series of Young's Modulus variances. The concrete values are listed as the strength values of the material.

<table>
<thead>
<tr>
<th>FRP [GPa]</th>
<th>Adhesive [GPa]</th>
<th>Concrete [ksi]</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>0.39</td>
<td>2</td>
</tr>
<tr>
<td>41</td>
<td>1.0</td>
<td>3.1*</td>
</tr>
<tr>
<td>100</td>
<td>2.41</td>
<td>4</td>
</tr>
<tr>
<td>139*</td>
<td>2.45</td>
<td>5</td>
</tr>
<tr>
<td>160</td>
<td>3.18*</td>
<td>6</td>
</tr>
<tr>
<td>300</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

4.2.1 FRP

There are many different types of FRP with different Young's moduli. A normal carbon fiber reinforced polymer (CFRP) strip has a Young’s Modulus between 155-165 GPa. A high modulus CFRP has a Young’s Modulus of 300 GPa. Glass fiber reinforced polymer (GFRP) and carbon reinforced vinyester resin strips have moduli of 41 GPa and 131 GPa respectively (Bank 2006). Since the control value was 139 GPa, a value of 100 GPa was also selected in addition to the control modulus in this part of the parametric study in order to increase the number of load-deflection curves. The value of 10 GPa was a trial run on accident, instead of 100 GPa. However, it offers yet another curve to view the bond behavior so it was kept in the analysis. As a result, six models were run using different Young's moduli selected from a range of different FRPs as well as arbitrary values. The effects of Young’s modulus of the FRP on maximum peel load can be found in Figure 4.5. The deflections at maximum load for different models are shown in Table 4.4.
The maximum load value increased as the Young's modulus of the FRP was increased. This was expected after completing the geometric parametric study because higher Young's modulus means higher stiffness.

Comparing the CFRP with high modulus (300 GPa) with that of the control model (139 GPa), even though the stiffness of the material is more than doubled, only a 12.8% increase in maximum load capacity is found. In addition to only a relatively small increase in load capacity, the beam also sees a 30% reduction in allowable displacement before crack propagation occurs. When comparing the GFRP (41 GPa) to the high modulus CFRP(300 GPa, an increase in Young’s modulus of more than 700%), those percentages change from 12.8% and 30% to 33% and 54%, respectively.

Although differences in load and displacement exist among different models with different FRP Young's moduli, the overall impact on the specimen stiffness is relatively small. Looking at the models with 100, 139 and 160 GPa of Young's modulus for FRP, a range of 60 GPa of Young's modulus change only causes the load to increase 72 N. This is confirmed by Dai et al. (2005). They stated that the maximum bond stress increases and the interfacial ductility decreases slightly with the increasing of the FRP stiffness.

<table>
<thead>
<tr>
<th>Young's Modulus [GPa]</th>
<th>Pmax [N]</th>
<th>δ at Pmax [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>307.2978</td>
<td>2.635625</td>
</tr>
<tr>
<td>41</td>
<td>400.8662</td>
<td>1.835625</td>
</tr>
<tr>
<td>100</td>
<td>453.5684</td>
<td>1.195</td>
</tr>
<tr>
<td>139 *</td>
<td>472.9098</td>
<td>1.195</td>
</tr>
<tr>
<td>160</td>
<td>485.9773</td>
<td>1.043125</td>
</tr>
<tr>
<td>300</td>
<td>533.5443</td>
<td>0.835625</td>
</tr>
</tbody>
</table>
Figure 4.5 Numerical load vs. displacement curves for models with different FRP Young's moduli.

4.2.2 Epoxy

There are also many different types of epoxy with different Young's moduli. Five different epoxies reported by Dai et al. (2005) were selected to perform study in this research. Their properties are tabulated in Table 4.5. The effects of the adhesive Young's modulus on the load versus deflection of the specimens can be found in Figure 4.6 and the critical values are reported in Table 4.6.
Table 4.8 Adhesive Material Properties

<table>
<thead>
<tr>
<th>Type of Adhesive</th>
<th>Elastic Modulus [GPa]</th>
<th>Poisson’s Ration</th>
</tr>
</thead>
<tbody>
<tr>
<td>CN-100</td>
<td>0.39</td>
<td>0.45</td>
</tr>
<tr>
<td>SX-325</td>
<td>1.0</td>
<td>0.38</td>
</tr>
<tr>
<td>FR-E3P</td>
<td>2.41</td>
<td>0.38</td>
</tr>
<tr>
<td>FP-NS</td>
<td>2.45</td>
<td>0.38</td>
</tr>
<tr>
<td>Tyfo-TC *</td>
<td>3.18</td>
<td>0.34</td>
</tr>
</tbody>
</table>

There is no significant variance across the five models pertaining to the maximum peel load, especially among the epoxies with Young’s modulus from 2.41 to 3.18 GPa. Comparing the results obtained in this study to those obtained by Dai et al. (2005) show conflicting results. Dai et al. (2005) used the CN-100 and FR-E3P adhesives and found that the interface fracture energy decreases with an increase in the adhesive elastic modulus. However, it should be noticed that the specimens used by Dai et al. (2005) were subjected to Mode II loading while the model used in the present study was subjected to Mode I loading. Therefore, the effect of epoxy Young's modulus is also related to loading modes.

Table 4.9 Critical Values for Different Adhesive Young's Modulus

<table>
<thead>
<tr>
<th>Type</th>
<th>Pmax [N]</th>
<th>δ at Pmax [mm]</th>
<th>Young's Modulus [GPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>CN-100</td>
<td>429.8257</td>
<td>1.435625</td>
<td>0.39</td>
</tr>
<tr>
<td>SX-325</td>
<td>460.5679</td>
<td>1.235625</td>
<td>1.0</td>
</tr>
<tr>
<td>FR-E3P</td>
<td>470.0765</td>
<td>1.235625</td>
<td>2.41</td>
</tr>
<tr>
<td>FP-NS</td>
<td>469.9824</td>
<td>1.235625</td>
<td>2.45</td>
</tr>
<tr>
<td>Tyfo-TC *</td>
<td>472.9098</td>
<td>1.195</td>
<td>3.18</td>
</tr>
</tbody>
</table>
4.2.3 Concrete

Five different concrete strengths, i.e., $f'_c = 2, 3.1^*, 4, 5,$ and $6$ ksi were selected to perform this part of parametric study. Because concrete Young’s modulus is directly related to its strength by Equation 4.1, five models with different concrete Young’s modulus were run in this study.

$$E_c = 57,000\sqrt{f'_c} \quad (4.1)$$

where $E_c$ is Young’s modulus and $f'_c$ is concrete strength in psi. The effects of the concrete strength on maximum peel load can be found in Figure 4.7 and the critical values for models with different concrete are shown in Table 4.7. There was a noticeable
increase in the maximum peel load when raising the compressive strength of the concrete. The increase is most prominent when looking at the load-displacement curves for 2 ksi and 3.1 ksi concrete. The maximum peel load increases from 408.9 N to 472.9 N when the strength is raised from 2 ksi to 3.1 ksi, a 15.7% increase. Doubling the strength from 3.1 to 6 ksi only brings an 11.9% increase in maximum load. As seen before, the stiffer material properties lead to a stronger bond to resist initial debonding. However, the higher strength concretes correspond with a lower vertical displacement at point of complete delamination.

Table 4.10 Critical Values for models With Different Concrete

<table>
<thead>
<tr>
<th>Concrete Strength [ksi]</th>
<th>Young’s Moulus [GPa]</th>
<th>$P_{\text{max}}$ [N]</th>
<th>$\delta$ at $P_{\text{max}}$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>18</td>
<td>408.8946</td>
<td>0.901929</td>
</tr>
<tr>
<td>3.1</td>
<td>22</td>
<td>472.9098</td>
<td>1.195</td>
</tr>
<tr>
<td>4</td>
<td>25</td>
<td>491.1031</td>
<td>1.235625</td>
</tr>
<tr>
<td>5</td>
<td>28</td>
<td>511.9969</td>
<td>1.235625</td>
</tr>
<tr>
<td>6</td>
<td>30</td>
<td>528.1511</td>
<td>1.235625</td>
</tr>
</tbody>
</table>

In Figure 4.7, the model uses multi-linear stress-strain curves for concrete which are closer to the real curves. If using linear elastic curve for concrete, the load versus deflection curves for the specimens are shown in Figure 4.8.

The comparison between Figures 4.7 and 4.8 shows the importance of correctly modeling the concrete. Not only does modeling the concrete linearly reduce the maximum peel load, it also fails to show that models that used higher strength concretes went through complete delamination.
Figure 4.7 Numerical load vs. displacement curves for models with different concrete.

Figure 4.8 Numerical load vs. displacement curves for models using concrete with linear elastic property.
4.3 Adjusted Cohesive Element Properties

A closer look at the effects of the cohesive element on the bond behavior is taken in this section. Three parameters were looked at; the effect of adjusting the total fracture energy \( G_f \) by holding the displacement at the control value of 0.0613 mm while adjusting the maximum stress, holding the fracture energy constant at the control value of 100 J/m\(^2\) while adjusting the maximum stress, and lastly, holding the maximum stress at the control value of 0.6 MPa while adjusting the displacement values.

4.3.1 Constant Displacement

The cohesive properties for these trials are shown in Table 4.8. Figure 4.9 shows the load vs. displacement curves for models with different concrete fracture energies obtained by changing the maximum stress in the cohesive elements. The results derived from the models make sense. Increasing the maximum stress in the fracture energy equation allows for the cohesive element to retain its shape better, meaning that the cohesive element can be subjected to a greater load before exceeding the maximum displacement of 0.0613 mm. When the maximum displacement is reached, the crack propagates to the next element.

<table>
<thead>
<tr>
<th>( G_f ) [J/m(^2)]</th>
<th>( \sigma ) [MPa]</th>
<th>( \Delta ) [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>0.3</td>
<td>0.0613</td>
</tr>
<tr>
<td>100</td>
<td>0.6</td>
<td>0.0613</td>
</tr>
<tr>
<td>150</td>
<td>0.9</td>
<td>0.0613</td>
</tr>
<tr>
<td>200</td>
<td>1.2</td>
<td>0.0613</td>
</tr>
<tr>
<td>250</td>
<td>1.5</td>
<td>0.0613</td>
</tr>
</tbody>
</table>
4.3.2 Constant Maximum Stress

For a constant maximum stress, the effects of the displacement variable can be studied. The cohesive properties for these trials are shown in Table 4.9. Figure 4.10 shows the load vs. displacement curves for models with different concrete fracture energies obtained by changing the displacement at maximum stress in the cohesive elements. Increasing the displacement in the fracture energy equation allows for a greater change in the cohesive elements shape, meaning that the cohesive element can go through a greater displacement before the crack propagates. Figure 4.10 shows that the model
with the largest displacement at maximum stress ($\Delta = 0.18 \text{ mm}$) has the largest peel load and experiences initial crack propagation at a greater vertical displacement than other models.

Table 4.12 Cohesive Properties and Critical Values

<table>
<thead>
<tr>
<th>$G_f$ [J/m$^2$]</th>
<th>$\sigma$ [MPa]</th>
<th>$\Delta$ [mm]</th>
<th>$P_{\text{max}}$ [N]</th>
<th>$\delta$ at $P_{\text{max}}$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td>0.6</td>
<td>0.008</td>
<td>361.9158</td>
<td>0.1825</td>
</tr>
<tr>
<td>50</td>
<td>0.6</td>
<td>0.0307</td>
<td>409.7191</td>
<td>0.835625</td>
</tr>
<tr>
<td>100*</td>
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<td>0.0613</td>
<td>472.9098</td>
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</tr>
<tr>
<td>200</td>
<td>0.6</td>
<td>0.123</td>
<td>582.7583</td>
<td>1.635625</td>
</tr>
<tr>
<td>294</td>
<td>0.6</td>
<td>0.18</td>
<td>640.4165</td>
<td>2.035625</td>
</tr>
</tbody>
</table>

Figure 4.10 Numerical load vs. displacement curves for models with variable displacements in cohesive elements.
4.3.3 Comparison of the Effects of Stress vs. Displacement in Cohesive Elements

The highest fracture energy curves from both Figure 4.9 and Figure 4.10 can be seen together in Figure 4.11. This figure provides a good comparison of the effects of displacement and stress components of the fracture energy equation for the cohesive element. The higher load actually occurs in the lower fracture energy with higher maximum stress in the cohesive element. Therefore, the maximum stress in the cohesive element has a greater impact on the maximum peel load. The impact of the displacement value of the cohesive element can be seen at specimen displacement at the maximum peel load. Having a greater displacement value in the cohesive element results in a more flexible system. This allows for more deflection before crack propagation occurs.

![Comparison of load vs. displacement curves for varied cohesive element properties.](image)

The effects of the cohesive element properties can more easily be seen in Figure 4.12 which shows load vs. displacement curves consisting of constant fracture energy of 100 J/m². The low maximum stress values coincide with a higher displacement value in
the cohesive elements. As the value of maximum stress increases, the maximum peel load of the whole MDCB specimen increases. At the same time, the crack propagation occurs sooner in the model using the cohesive element with higher maximum stress.

![Figure 4.12 Load vs. displacement curves for the model with $G_f = 100 \, J/m^2$.](image)

4.4 Mixed Mode Loading

For the mixed mode loading, an additional displacement condition was introduced in order to induce the mixed mode behavior. However, caution must be used in the selection for the location of this displacement condition. If the mode II force, $P_{II}$, is applied at the same node where $P_I$ is applied, a moment, $M=P_{II}e$, will be applied to the composite overlay as shown in Figure 4.13. Huang and Lyons (2005) showed that the
fracture energy in such loading condition was notably higher than the values found using theoretical equations. In this study, after running one trial with a mode II load (displacement) at the same node as the mode I load, it was confirmed that higher fracture energies result. In order to prevent this, the mode II displacement must be applied at a different point.

Figure 4.13 Schematic figure of MDCB specimen under mode II loading (Huang and Lyons 2005).

For the mixed mode loading models in this study, the mode II loading was applied to the 3 nodes which compose the end of the CFRP and epoxy layer as shown in Figure 4.14. Also, in order to see the effects that mode II loading has on the load-displacement response curve, the proper loading steps must be applied in ANSYS. The mode II loading is first taken into account in load step 1. This insures that the total mode II stress is present at the beginning of the mode I loading.

Due to the severe effects of Mode II loading, a different geometry for the model was used. Because of the load step process, the mode II loading takes effect first. In the original model, complete delamination occurred with only a 0.126 mm displacement in the horizontal direction. In order to obtain a broader range of Mode II loads, the length of the specimen was increased from 173 mm to 350 mm. This allowed the maximum
Mode II load to increase to 0.158 mm. Figure 4.15 shows the results from the Mixed Mode loading conditions.

Figure 4.14 Mixed-mode loading diagram.

Figure 4.15 Load vs. displacement curves for the models subjected to mixed mode loading with different mode II displacement.
A couple things stand out in this graph compared to the curves for pure mode I loading. The first thing to be considered with this loading situation is the decrease in maximum peel loads for initial debonding. As a greater Mode II loading was applied, the peel load to initially propagate the crack greatly diminished. Mode II completely controls delamination once a displacement of 0.158 mm is applied to the model. Compare this to the necessary 34.13 mm required for complete pure mode I delamination.

The next difference from pure Mode I loading case is the increasing Mode I load as the displacement increases to a certain value. This can be attributed to the deformation of the model. As the Mode I loading is applied, it begins to deflect the FRP up. Once a certain vertical displacement takes place, the Mode II load causes larger moment at the crack tip which is opposite to the moment caused by the Mode I load as shown in Figure 4.16. Therefore, the Mode I load has to be increased in order to overcome the effect of Mode II load to continue debonding the FRP. Fracture energy to debonding FRP in mixed mode loading conditions is larger than that requirement for pure Mode I loading (Wan 2002; Huang and Lyons 2005). Increasing fracture energy in mixed mode loading condition can also explain this phenomenon. The requirement of larger load to debond FRP from concrete when MDCB specimen is subjected to mixed-load was also observed in experimental test by Wan (2002).
The last item to note in Figure 4.15 is that the curves for mixed mode loading conditions do not originate at the origin (non-zero deflection with zero Mode I load). This is due to the deformation under Mode II loading in the pre-cracked section and the location to calculate Mode I load and displacement. As shown in Figure 4.14, the Mode I load is applied at the right most point of the hinge while the Mode II load is applied to the end of the FRP and epoxy. When Mode II load is applied, the FRP and epoxy will deform to the right. Because the stiffness of epoxy is smaller than that of the FRP, it will deform more than the FRP and cause the unbonded part of the specimen (the part not bonded to concrete including the hinge) to bend up. Therefore, the right end of the hinge will have an upward deflection (Mode I deflection direction) even though no Mode I load is applied.
The FRP debonding from concrete under mixed mode loading is a complex problem. It is not only related to the magnitude of the Mode II applied at the end of the specimen, but also related to the phase angle (a measurement of mixed mode loading fracture energy ratio between $G_{II}$ and $G_{I}$) at the crack tip and critical total fracture energy for debonding. The actual criterion for crack propagation in such loading conditions is still not clear in the current FRP research community. Therefore, more research should be conducted to obtain a better understanding of this problem.

4.5 Summary

Throughout the parametric study involving the geometric makeup of the finite element analysis model one consistent conclusion can be made: as the material thicknesses were increased, so did the maximum peel load. But as seen in both the FRP and epoxy trials, the increased thickness resulted in increased stiffness of the debonded FRP; the stiffer the debonded FRP, the lower the vertical displacement at complete delamination.

Increasing the Young's modulus of the FRP component in the model yielded similar results to those derived from increasing the FRP plate thickness. There was little effect noticed by adjusting the Young's modulus of the adhesive under mode I loading. Increasing the strength of the concrete increased the maximum peel load for the MDCB specimens.

It was determined that the stress component of the energy equation for the cohesive element has a greater impact on the maximum peel load than the displacement
component. The displacement component of the cohesive element has more effect on the ductility of the specimen.

The presence of even a little mode II loading greatly impacts the specimen's behavior. The loading conditions create a opposite moments. Therefore, in order for crack propagation to continue, the mode I load is increased to overcome the mode II moment.

Tables 4.13, 4.14, and 4.15 provide another tool for analyzing the structural influence each model component has.

<table>
<thead>
<tr>
<th>Table 4.13 Sensitivity Values for FRP</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>FRP</strong></td>
</tr>
<tr>
<td>Thickness Comparison Values [mm]</td>
</tr>
<tr>
<td>0.5-1.0</td>
</tr>
<tr>
<td>1.0-1.5</td>
</tr>
<tr>
<td>1.5-2</td>
</tr>
<tr>
<td>2-2.5</td>
</tr>
<tr>
<td>2.5-3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Modulus Comparison Values [Gpa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.0-41.0</td>
</tr>
<tr>
<td>41.0-100.0</td>
</tr>
<tr>
<td>100.0-139.0</td>
</tr>
<tr>
<td>139.0-160.0</td>
</tr>
<tr>
<td>160.0-300.0</td>
</tr>
</tbody>
</table>
Table 4.14 Sensitivity Values for Adhesive

<table>
<thead>
<tr>
<th>Thickness Comparison Values [mm]</th>
<th>% Increase in Thickness</th>
<th>% Increase in P&lt;sub&gt;max&lt;/sub&gt;</th>
<th>% Change in δ at P&lt;sub&gt;max&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5-1.0</td>
<td>100.0</td>
<td>10.1</td>
<td>9.5</td>
</tr>
<tr>
<td>1.0-1.2</td>
<td>20.0</td>
<td>3.8</td>
<td>0.0</td>
</tr>
<tr>
<td>1.2-2</td>
<td>66.7</td>
<td>16.4</td>
<td>-12.7</td>
</tr>
<tr>
<td>2.0-3.0</td>
<td>50.0</td>
<td>16.2</td>
<td>-14.6</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Modulus Comparison Values [Gpa]</th>
<th>% Increase in Modulus</th>
<th>% Increase in P&lt;sub&gt;max&lt;/sub&gt;</th>
<th>% Decrease in δ at P&lt;sub&gt;max&lt;/sub&gt;</th>
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<td>7.2</td>
<td>13.9</td>
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<tr>
<td>1.0-2.41</td>
<td>141.0</td>
<td>2.1</td>
<td>0.0</td>
</tr>
<tr>
<td>2.41-2.45</td>
<td>1.7</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>2.45-3.18</td>
<td>29.8</td>
<td>0.6</td>
<td>3.3</td>
</tr>
</tbody>
</table>

Table 4.15 Sensitivity Values for Concrete

<table>
<thead>
<tr>
<th>RTC Comparison Values [mm]</th>
<th>% Increase in Thickness</th>
<th>% Increase in P&lt;sub&gt;max&lt;/sub&gt;</th>
<th>% Increase in δ at P&lt;sub&gt;max&lt;/sub&gt;</th>
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<td>50</td>
<td>26.0</td>
<td>0</td>
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</table>

<table>
<thead>
<tr>
<th>Modulus Comparison Values [Gpa]</th>
<th>% Increase in Modulus</th>
<th>% Increase in P&lt;sub&gt;max&lt;/sub&gt;</th>
<th>% Increase in δ at P&lt;sub&gt;max&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>18-22</td>
<td>22.2</td>
<td>15.7</td>
<td>32.5</td>
</tr>
<tr>
<td>22-25</td>
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<td>3.4</td>
</tr>
<tr>
<td>25-28</td>
<td>12.0</td>
<td>4.3</td>
<td>0.0</td>
</tr>
<tr>
<td>28-30</td>
<td>7.1</td>
<td>3.2</td>
<td>0.0</td>
</tr>
</tbody>
</table>
Chapter 5 Conclusions and Future Research

A comprehensive literature review of debonding failure of externally FRP bonded concrete members was provided. The review covered the debonding failure modes, debonding analysis using fracture mechanics, and cohesive zone modeling. The primary failure mode for FRP debonding is cohesive failure in the concrete substrate near the concrete adhesive interface.

Finite element modeling of MDCB test specimens to evaluate the bond between concrete and FRP overlays has been performed. The FE model was calibrated based on experimental results and then a parametric study was performed.

5.1 Conclusions

This study demonstrates using a finite element model to simulate a MDCB specimen to assess the bond behavior between the concrete and FRP. The parametric study analyzed the effects of geometric dimensions, material properties, and cohesive properties. The study showed:

- Increasing the thickness of the FRP, the maximum peel load increased
- Increasing the thickness of the adhesive, the maximum peel load increased
- Increasing the thickness of the RTC, the maximum peel load increased
- Increasing the Young's modulus of the FRP, the maximum peel load increased.
- Increasing the Young's modulus of the adhesive, little effect was noticed.
- Increasing the Young’s modulus of the concrete, the maximum peel load increased
• The stress involved in the energy equation for cohesive element has a greater impact on the maximum peel load.

• The displacement involved in the energy equation for cohesive element has a greater impact on the flexibility of the model.

Although increasing the thickness and Young's modulus of the FRP, epoxy or residual concrete on the debonded FRP can increase the maximum peel load, this may cause complete delamination at a lower vertical displacement. The mixed mode loading has significant effect on the behavior of FRP bonded concrete specimens.

It was seen that the most important component of the composite beam was the FRP. When the plate debonds from the concrete, the FRP, adhesive, and RTC act as a cantilever. The Young's modulus for the FRP being so high contributes almost all of the stiffness present in the debonded cantilever. Therefore, changing the thickness and elastic modulus of the FRP will have a greater impact on the stiffness of the cantilever. When dealing with retrofitted structures, the concrete material is already present, meaning the only control in the design is with the selection of FRP and adhesive materials, giving the FRP an even greater influence on the bond behavior since it will be one of the two selected materials.

5.2 Future Research

Experimental MDCB tests should be conducted of some of the parametric values selected to see the accuracy of the numerical model.
The continuation of research in this area should focus primarily on the mixed mode modeling. Good insight could be gained by finding a more appropriate way to induce the Mode II loading on the model.

The adhesive layer in this model was represented as an elastic material. It is valid for the models in this study because debonding happened within the concrete at relatively low stress. In actuality, this adhesive layer should be modeled as a visco-elastic material. However, this is made difficult due to restrictions on information from the material provider. Modeling the adhesive as a viscoelastic material may affect the local stress distribution around the crack tip, but is not expected to have significant effect on the overall load versus deflection response of the MDCB specimens. Another aspect to consider for future finite models, is to study the effects caused by modeling the concrete above the RTC with tensile stress limits versus compression. Also, the FRP component of the composite beam specimen acted in an isotropic manner. This however is not true as FRPs are anisotropic, the FRP provides different elastic moduli depending on the orientation of the fibers.

One area where potential bond strength could be found is the RTC. It was found that a bond formed under normal conditions had a RTC value of 2 mm. What if there were different steps taken prior to casting the FRP, would this be able to increase the RTC value for normal conditions and thus increase the capacity of the bond?
References


Debonding Using a Crack Band Approach.” *Journal of Composites for
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of fiber reinforced plastics sheet-concrete interfaces with a simple method.”
*Journal of Composites for Construction*, 9(1), 52-62.

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Dugdale, D.S. (1960). “Yielding of Steel Sheets Containing Slits.” *Journal of the
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Hillerborg A., Modeer, M., and Petersson, P. “Crack Model to Different Materials.”

Composite-Concrete Bond Durability.” *Journal of Reinforced Plastics and
Composites*, 24 (13), 1387-1405.

Irwin, G. R. (1957). “Analysis of Stresses and Strains Near the End of a Crack Traversing

Karbhari, V.M., Engineer, M. (1996). "Investigation of Bond between Concrete and
Composites: Use of a Peel Test." *Journal of Reinforced Plastics and Composites*,
15, 208-227.

rehabilitation schemes for concrete: use of a peel test." *Journal of Materials
Science*, 32, 147-156.


Appendix A

*Commands in Italics*

> Denotes GUI (Graphic User Interface)

The following are the steps used to create the control model.

```
/title, Control
/units, SI
/prep7

(First define the key points of the model)

k, 1, 0, 0
k, 2, 173, 0
k, 3, 0, 72
k, 4, 173, 72
k, 5, 0, 74
k, 6, 173, 74
k, 7, 0, 76
k, 8, 173, 76
k, 9, 0, 77.2
k, 10, 173, 77.2
k, 11, 0, 79.2
k, 12, 173, 79.2
k, 13, -28, 79.2
k, 14, -8, 79.2
k, 15, -28, 77.2
```
(Second, Connect the key points with lines)
The model should now look similar to this:

Figure A.1 Line Model
(Next, create the area elements)

\[ a,1,2,4,3 \]
\[ a,3,5,6,4 \]
\[ a,5,7,8,6 \]
\[ a,7,9,10,8 \]
\[ a,9,11,12,10 \]
\[ a,13,14,16,15 \]
\[ a,14,11,9,16 \]
\[ a,15,16,18,17 \]
\[ a,16,9,7,18 \]
\[ a,17,19,20,21 \]
\[ a,17,21,22,18 \]

Figure A.2 Area Model

(Establish the element type)

\[ et,1,82 \]
(Establish Material Properties)

>>Preprocessor>Material Props>Material Models

>>Material>New Model>Structural>Linear >Elastic>Isotropic

Table A.1 Material Properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Young’s Modulus [GPa]</th>
<th>Poisson’s Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>22</td>
<td>0.28</td>
</tr>
<tr>
<td>Epoxy</td>
<td>3.18</td>
<td>0.34</td>
</tr>
<tr>
<td>FRP</td>
<td>139</td>
<td>0.2</td>
</tr>
<tr>
<td>Hinge Plate</td>
<td>70</td>
<td>0.30</td>
</tr>
</tbody>
</table>

(For the concrete, non linear properties must also be input)

Table A.2 Nonlinear Concrete Properties

<table>
<thead>
<tr>
<th>Point</th>
<th>Strain</th>
<th>Stress [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.000295</td>
<td>6.48</td>
</tr>
<tr>
<td>2</td>
<td>0.0005</td>
<td>10.3</td>
</tr>
<tr>
<td>3</td>
<td>0.0008</td>
<td>15.1</td>
</tr>
<tr>
<td>4</td>
<td>0.0012</td>
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</tr>
<tr>
<td>5</td>
<td>0.001964</td>
<td>21.6</td>
</tr>
</tbody>
</table>

(Next attach the material properties so the corresponding areas)

>>Preprocessor>Meshing<Mesh Attributes>Picked Areas

(Next assign mesh values to the lines)

>>Preprocessor>Meshing>Size Cntrls>ManualSize>Lines>Picked Lines
Figure A.3 Line Numbers for Mesh

(Lines 1,2,3 will be divided 10 times using the ndiv command)

(Lines 4,5,6,7,8 will have element size of 0.5 mm)

(Lines 9,10,11,12 will have element size of 2 mm)

(The remaining lines will all have an element size of 1 mm)

(Mesh the Elements)

>Preprocessor>Meshing<Mesh>Areas>Free
Figure A.4 Element Model

(Next insert the cohesive zone elements located at the RTC.)

Figure A.5 Cohesive Element Model
(Select the concrete elements shown above from the entire model.)

(Assign real constant and material properties for the special CZM element)

real, 2
mat, 4

(Select only the top row of elements and apply the component command)

cm, e1, elem

(Select only the bottom row of elements and apply the component command)

cm, e2, elem

(Next select all and apply new element type)

alls
et, 2, 203

(Use the czmesh command to create and mesh the interface)

czmesh, e1, e2

(Activate a data table for nonlinear material properties)

tb, czm, 5 (5 represents the material number)

(Use TBDATA command to define the data)

tbdata, 1, C1, C2, C3

(C1 is the normal maximum interface stress $\sigma_{\text{max}}$ in the CZM)

(C2 is the normal characteristic separation displacement $\delta_n$ in the CZM)

(C3 is the tangential characteristic separation displacement $\delta_t$ in the CZM)

(For this example, C1 = 0.6 MPa, C2=C3 = 0.0613 mm)

(Next, Apply the displacement load, and boundary conditions according to the test setup)

(Set the proper nonlinear solution control options)

/solu

outres, all, all
alls

nsubst, 400

neqit, 1200

nlgeom, on

solve

(The deformed shape should resemble the figure below)

Figure A.6 Deformed Model