PERFORMANCE OF UWRAP AS AN ANCHORAGE SYSTEM IN
EXTERNALLY BONDED FRP REINFORCED CONCRETE ELEMENTS

A Dissertation in
Civil Engineering
by
Jae Ha Lee

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Submitted in Partial Fulfillment
of the Requirements
for the Degree of

Doctor of Philosophy

May 2010
The dissertation of Jae Ha Lee was reviewed and approved* by the following:

Maria Lopez de Murphy  
Will development professor of Civil and Environmental Engineering  
Dissertation Advisor  
Chair of Committee

Andrew Scanlon  
Professor of Civil and Environmental Engineering

Bohumil Kasal  
Professor of Civil and Environmental Engineering

Charles E. Bakis  
Distinguished Professor of Engineering Sciences and Mechanics

Ivica Smid  
Associate Professor of Engineering Sciences and Mechanics

Peggy Johnson  
Professor of Civil and Environmental Engineering  
Head of the Department of Civil and Environmental Engineering

*Signatures are on file in the Graduate School
ABSTRACT

During the past decades, numerous studies for flexural and shear strengthening using FRP have shown good strengthening effects and durability. Many of these studies, in particular those related to concrete structures, however, have also reported a lack of ductility in the strengthened element due to premature debonding failure at the concrete-FRP interface. The possibility of preventing or delaying the premature debonding of concrete-FRP interfaces has lead to the investigation and development of new anchor systems for externally bonded FRP applications. In this study, performance of Uwrap as an anchorage system was studied. Experimental and numerical procedures were proposed to characterize the effect of the Uwrap in the performance of externally bonded FRP reinforced concrete elements (EBFS).

Two key factors were found pertaining to the externally bonded FRP system with the Uwrap. The first factor is the stress concentration at the corner of the Uwrap. The stress concentration at the corner region of the Uwrap was the main cause of the rupture failure of the Uwrap. The second factor is the frictional effect between the debonded surfaces under the region of the Uwrap after the onset of the debonding.

Based on these two key factors, a frictional bond-slip model of the concrete-FRP interface was proposed in this study in order to predict the anchor effect of the Uwrap. The developed FE analysis with the proposed frictional bond-slip model well predicts the bond behavior under the Uwrap locations.
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ACKNOWLEDGEMENTS

Above all, I thank my almighty God. It is just impossible to describe how much he loves me and carries me especially when I was passing thought difficult times in my life. I want to share the words God gave to me. May these words of wisdom be inspirational to those who are experiencing hard times.

“Even to you old age and gray hairs, I am he. I am he who will sustain you. I have made you and I will carry you. I will sustain you and I will rescue you (Isa 46:4)”

Several people also played an important role in accomplishing of this thesis. I also would like to give my special thanks to my parents, Kunhwa Lee and Hoyeon Bae, my sister, Yoosook Lee and my fiancée, Katie Park. Without their devotions, supports, continuous prayers and love for me and my works, I could not finish this thesis.

I am heartily thankful to my supervisor, Dr. Maria Lopez, whose encouragements when I was struggling, guidance and supports from the beginning to the end during the every stage of this work enable me to accomplish this thesis.

This thesis also would not have been possible without kind supports and guidance of my committee, Dr. Andrew Scanlon, Dr. Bohumil Kasal, Dr. Chales E. Bakis and Dr. Ivica Smid. Through their valuable recommendations and comments, I have been able to find problems and solve those. I also would like to express my thanks to all the member of our research group for their helps and supports.

The part of this research activities described in this dissertation has been financially supported from the National Science Foundation (CMS-0330592) and test materials were supported by Fyfe Company. These supports are gratefully acknowledged.
Chapter 1
Introduction

Fiber reinforced polymer (FRP) composites are becoming a prominent material specially for strengthening reinforced concrete (RC) structures. Use of FRP laminate to strengthen the existing structures can significantly reduce the construction cost. The cost of the carbon fiber reinforced polymer (CFRP) laminate including the installation can cost about $160 per meter when width of FRP strip is one meter (e.g. Hutchinson et al. 2003). The average cost of the new bridge per square meter during 2008 was $1840. This indicates that use of FRP laminate to strengthen the structure can minimize the construction cost up to 91% (e.g. California DOT 2008). Because of their light weight and ease of installation (lower labor cost), and minimal cost (lower maintenance cost and longer service life), FRP repair system can be the economical alternative to traditional repair system (e.g. Mirmiran 2004).

Accordingly, during the past decades, numerous studies for flexural and shear strengthening using FRP have been conducted and have shown good strengthening effects and durability (e.g. ACI 440-2R-08, Coronado and Lopez 2006, Coronado and Lopez 2008). Research studies on the flexural and shear strengthening capabilities of externally bonded FRP systems have shown the significant improvements on the performance of concrete, steel, and wood structures (e.g. ACI 440R-07, Borri et al. 2005, Pellegrino et. al 2009).

Many of these studies, in particular those related to concrete structures, have
also reported a lack of ductility in the strengthened element due to premature debonding failure at the concrete-FRP interface (e.g. ACI 440.2R-02, Buyukozturk and Hearing 1998, Smith and Teng 2002). Several approaches have been used to characterize the complex bond behavior of the concrete-FRP interface in order to investigate the premature debonding failure at the concrete-FRP interface. Some of approaches were developed based on Mode 1 loading (e.g. Coronado and Lopez 2006, Coronado and Lopez 2008, Qiao and Chen 2007) while the others use Mode 2 loading to characterize the bond-slip behavior of the concrete epoxy interface (CEI) (e.g. Chajes et al. 1996, Yuan et al. 2004, Yao et al. 2005). Recently, research has been conducted on mixed mode loading on concrete-FRP interface. (e.g. Kishi et al. 2005, Niu et al. 2006, Pan and Leung 2007, Wang and Zhang 2008). The interface concept simplifies from the complicated mechanical behavior among concrete-epoxy-FRP to a simple interface force-separation behavior. To obtain the force-separation behavior of the interface, first, linear elastic fracture mechanics were used (e.g. Karbhari and Engineer 1996, Giurgiutiu et al. 2001). However, there is a limitation on using this framework to characterize the highly nonlinear behavior that could be present at the concrete-FRP interface. The use of non-linear elastic fracture mechanics has been shown to successfully overcome this limitation (e.g. Yuan et al. 2004, Coronado and Lopez 2007). It was also found that the premature debonding mainly takes place along the concrete close to the bond-line. This is a compelling reason to model the behavior of concrete-epoxy interfaces using the principles of concrete behavior based on the non-linear fracture mechanics (e.g. Coronado 2006). The history of non-linear fracture mechanics of the concrete is well summarized in Wang and Zhang (2007).
With the advance of research on the mechanism that control the premature debonding failure using the non-linear fracture mechanics of the concrete, the possibility of preventing or delaying the debonding of concrete-FRP interfaces has lead to the investigation and development of new anchor systems for externally bonded FRP applications. Recently developed anchor systems use fiber anchors, mechanical fasteners, Uwrap (transverse sheet), and metallic rods (e.g. Triantafillou 1998, Ozbakkaloglu 2009, Eshwar 2008, Lee et al. 2009).

Among the various types of anchor systems, one example of the Uwrap (U-shaped fiber sheets) application is shown in Figure 1-1

![Figure 1-1: Typical Uwrap application on a concrete beam](image)

Most studies of Uwrap strengthening show that the proper anchor system can increase the strengthening effects and prevent or delay the debonding failure of the longitudinal FRP sheet (e.g. Sharif et al. 1994, Lee et al. 1997, Shaheen et al. 2002, Oh and Shim 2004, Abdelrahman and El-Ghandour 2007).

The majority of the current literature on anchor systems refers to experimental studies (e.g. Triantafillou and Antonopoulos 2000, Leung et al. 2007). Even though these studies have been successful at showing that properly designed anchor systems can increase the FRP strengthening level (by preventing or delaying the debonding failures of
concrete-FRP interfaces) while at the same time inducing a more ductile failure mode, there is need for a better understanding of the mechanisms associated with the anchoring effects on FRP sheets and plates. The limited number of analytical studies on this topic are reflected in the few anchor-design guidelines described in current design documents such as the European FIB bulletin 14 (2001) and the ACI 440 design 440-2R report (2008). Therefore, comprehensive analytical studies of anchor systems such as Uwrap are a required step in the development of future anchor design guidelines.

FRP Uwraps have been successfully used for shear strengthening of concrete structures (ACI 440.2R-08). The transverse FRP sheet (U wrap) is typically externally bonded to the lateral sides of the flexural member (and sometimes continues through the top and/or bottom soffit) to provide additional shear capacity. Current design equations were developed in analogy to the shear capacity provided by steel stirrups, accounting for a “bond reduction” factor. Very few studies have been devoted to the use of Uwraps as anchorage for longitudinal FRP sheets or plates (e.g. Brena et al. 2003, Abdelrahman and El-Ghandour 2007).

This study focuses on the anchor effect of the Uwrap. The anchor effect is an important factor to analyze the flexural behavior of the strengthened beam with FRP Uwrap since the Uwrap increases debonding strengths as well as prevents the debonding propagation. Furthermore, even after complete debonding, the Uwrap can hold the longitudinal FRP and provide a residual anchoring strength to the FRP sheet and concrete structures, leading to better ductile failure compared to the premature debonding failure. In this study, experimental and analytical procedures will be presented in order to understand and predict the Uwrap behavior. Experiments are designed to obtain basic
material properties, observe the Uwrap behavior, select key parameters, and verify the developed numerical analysis. Numerical models were developed based on non-linear fracture mechanics.

1.1 Problem Statement

This study is designed to fulfill the need for a comprehensive study that addresses the effect of Uwrap on the longitudinal FRP sheet attached to the concrete structures. In particular, the effect that Uwrap has on the post debonding of FRP strengthened structures.

1.2 Objectives

The objective of this study is the characterization of the effect of Uwrap in the performance of externally bonded FRP system (EBFS). This study will focus on the characterization of Uwrap effects on EBFS, the related debonding mechanisms, and the overall behavior of the FRP strengthened RC beam with the Uwrap, leading to the arrest of the interface debonding (crack) propagation on the region near the concrete-epoxy interface.
1.2.1 Research tasks

1. Evaluate the state of the art on externally bonded FRP system (EBFS).
2. Conduct preliminary tests in order to obtain key parameters for controlling the performance of the Uwrap in EBFS.
3. Identify key parameters that control the performance of the Uwrap
4. Design pull-out tests with selected key parameters
5. Perform the pull-out tests
6. Perform experimental tests to obtain basic material properties
7. Design the FRP strengthened RC-beam with the results from the pull-out test
8. Develop numerical models of pull-out tests and beam tests using non-linear fracture mechanics and constitutive models for the concrete, epoxy, and unidirectional FRP sheet.
9. Validate the numerical analysis of the pull-out tests and beam tests with experimental data

1.3 The organization of the thesis

Each of the previously mentioned tasks is summarized in each chapter. The organization of the thesis is shown in Figure 1-2. This flow chart shows the organization of conducted tasks described above.

Chapter 1 presents the current problem statement and the corresponding tasks needed in order to fulfill the objectives of this study of the Uwrap.
Chapters 2 summarizes the literature review. The background information covers the behavior of the concrete-epoxy interface and several of the works related to the FRP Uwrap. Selected constitutive laws for each material will be also summarized in Chapter 2. The principal findings from the review of existing studies will be also presented at the end of Chapter 2. Chapter 3 shows the experimental works for the study of the Uwrap. Experimental works cover conducted pull-out tests and FRP strengthened flexural tests as well as basic material properties, which include the Mode 1 and Mode 2 properties of the interface. Obtained material properties presented in Chapter 3 were used for the numerical models in Chapter 4.
Chapter 4 presents the results from the developed numerical models in 2 dimensions (2D) and 3 dimensions (3D). Firstly, a proposed frictional bond-slip model of the interface will be explained. Based on the developed bond-slip model of the interface, the pull-out tests were modeled in 3 dimensions (3D) and 2 dimensions (2D). With the results of the pull-out tests, a large scale RC-T beam was modeled in 2D. The arresting mechanism of the debonding propagation and strengthening flexural capacity of the
strengthened T-beam with the Uwrap were investigated. This chapter also compares the results from the numerical study with the experimental results and proposes some design recommendations for the EBFS with the Uwrap.

Chapter 5 presents the conclusions of this study and offers recommendations for further studies.
Chapter 2

Literature Review

One of major failure modes of externally bonded FRP systems is the premature debonding failure between the concrete and FRP. In order to prevent or delay this type of failure, an anchoring system such as Uwrap (U-shaped fiber sheets) has been proposed.

The Uwrap is transversely strengthening the beam, and its direction is perpendicular to the main longitudinal FRP sheet. The Uwrap is also connected to the lateral sides of the concrete beam just as the longitudinal FRP sheet is bonded to the soffit of the beam. Therefore, the concrete-epoxy interface (CEI) is one of the most important materials to be characterized since both the Uwrap and longitudinal FRP sheet are bonded to the concrete structures. Accordingly, in this chapter, the bond behavior between the concrete and FRP sheet will be reviewed and discussed.

The possibility of preventing or delaying the debonding of concrete-FRP interfaces has lead to the investigation and development of new anchor systems such as the Uwrap for externally bonded FRP applications. As explained previously, the majority of the current literature on the Uwrap refers to experimental studies. The experimental results show an increment of beam capacity in shear in general when the Uwraps were used. The anchor effect from the Uwrap was not reported from the study of the shear strengthening. Selected papers will be also summarized in this chapter.

As previously mentioned in an introduction, studies on the anchor effect of the Uwrap are very limited and mostly have provided experimental results. There is a need
for a study that looks at the anchor effect from the Uwrap with numerical analysis.
Selected studies will be summarized in this section focusing on the anchor effects of the
Uwrap. The practical method of the finite element analysis for the modeling of Uwrap
will be summarized in this chapter from the selected studies.

In addition to these, an emerged measuring technique which is called digital
image correlation (DIC) will be introduced in this chapter. This is non-contact optical
technique for measuring deformation and strain from the digital images. This technique is
also used for measuring the slip of the experimental specimen. Background information
on DIC will be introduced in this chapter.

As in other parts of the literature review, material models for the numerical study
were studied in order to conduct numerical analysis. There are many existing material
models for the concrete, concrete-epoxy interface, and unidirectional FRP sheet. Selected
material models and the corresponding constitutive law will be summarized and
explained in this chapter.

As the studies had been conducted, the effect of stress concentration at the corner
region of the Uwrap and the frictional behavior between the debonded surfaces of the
concrete and the FRP sheet were observed from the study of the Uwrap. It was found that
these two important effects control the behavior of the Uwrap. To take into account of
these factors, studies related to these effects were reviewed, and important equations will
be presented in this chapter.
2.1 Characterization of the concrete epoxy interface behavior

As previously mentioned, many studies related to the externally bonded FRP system, in particular those related to concrete structures, have reported a lack of ductility in the strengthened element due to the premature debonding failure at the concrete-FRP interface (e.g. ACI 2008, Buyukozturk and Hearing 1998). It was found that the use of non-linear elastic fracture mechanics has been shown to successfully overcome this limitation. The history of fracture mechanics is well summarized in published papers (e.g. Bažant and Planas 1998, Wang and Zhang 2008, Coronado and Lopez 2008).

Two modes of fracture are present in the behavior of the concrete-FRP interface: a normal mode (Mode 1) and a shear mode (Mode 2) as shown in Figure 2-1. Constitutive laws for Mode 1 behavior have been derived from experimental procedures by Qiao and Xu (2004) and Coronado and Lopez (2008), among others (e.g. Coronado and Lopez 2005, Coronado and Lopez 2006, Qiao and Chen 2008). Numerical models using these constitutive laws have also been developed (e.g. Coronado and Lopez 2009). These models assume that the concrete substrate fails as principal tensile stresses reach a maximum value. The post-peak behavior follows a softening law. For the purposes of this study, an interface that follows this type of constitutive law will be named an interface with “Mode 1” behavior. Interfaces with shear constitutive laws or “Mode 2” behavior have been experimentally characterized and modeled and used for the numerical analyses by Lu et al. (2005) among others as shown in Figure 2-1 (e.g. Yuan et al. 2004; Yao et al. 2005, Wang 2007, Baky et al. 2007, and Kotynia et al. 2008). These constitutive laws focused on defining the shear stress-slip behavior of interfaces between concrete and FRP.
rather than the concrete tensile behavior. For both Mode 1 and Mode 2 behaviors, fracture energy of the interfaces can be defined by the area under the stress-crack opening or stress-slip constitutive laws.

Softening behavior can be simplified as linear if the maximum stress and fracture energy, obtained from experiments, remain the same as shown in Figure 2-1 (e.g. Lu et al. 2005). Recent studies have obtained good analytical results from the modeling of the concrete-FRP interface using a mixed mode approach (a combination of Mode 1 and Mode 2). These studies state that the concrete-FRP interface behavior is more related to a mixed Mode rather than a unique mode (e.g. Kishi et al. 2005, Niu et al. 2006, Wang 2007, Wang and Zhang 2008, Kotynia et al. 2008). It has been proven by these studies that the concrete-FRP interface is more related to mixed mode behavior rather than one fixed mode behavior (e.g. Wang 2007, Wang and Zhang 2008). The specific theory and test set up for obtaining the Mode 1 and Mode 2 behavior of the interface will be described in a Chapter 2 and obtained experimental results will be shown in Chapter 3.
2.2 FRP Uwraps

The majority of the current literature on Uwraps refers to experimental studies. A very limited number of analytical studies on this topic are reflected in the few anchor-design guidelines described in current design documents such as the European FIB bulletin 14 (2001) and the ACI 440 design 440-2R report (2008). The studies on the shear strengthening effect are the majority in the study of the Uwrap. Consequently, the developed equations for the shear strengthening will be presented. Secondly, even though the numbers are very limited, some of the studies have been contributed to the understanding of the anchor effects from the Uwrap. Related studies on the anchor effect of the Uwrap will be also presented. Some practical methods for modeling of the Uwrap were also reviewed and will be presented in Section 2.2.3.

2.2.1 FRP Uwraps for shear strengthening

The majority of the studies on the Uwrap have focused on the shear strengthening effect. Experimental works on the beams were conducted, and then some of analytical models for shear strengthening of the RC beam using FRP Uwraps were proposed. The design shear strength of a concrete member strengthened with FRP system can be obtained using strength reduction factor as proposed in ACI 318-05. For calculating contribution of the FRP sheet to the shear strength, additional reduction factor is used. It is a value of 0.85 for the three sided FRP reinforcement (Uwrap).
\[ V_n = V_c + V_s + V_f \]

where, \[ V_f = \frac{A_f E_f \varepsilon_{fe} d_f}{s_f} \]

Where,

\( V_n \): Nominal shear strength,

\( V_c \): Nominal shear strength by concrete,

\( V_s \): Nominal shear strength by steel,

\( V_f \): Nominal shear strength by FRP (Uwrap)

\( A_f \): Area of FRP (Uwrap) plate,

\( E_f \): Elastic modulus of FRP plate

\( \varepsilon_{fe} \): Effective strain of the FRP (Uwrap)

\( d_f \): Depth of beam

\( s_f \): Spacing between FRP shear reinforcing

Most developed equations show similar form as in Eq. 2.1. However, the calculation of an effective strain of FRP shear reinforcing (Uwrap) is different depending on the developed models. Triantafillou (1998) and Khalifa and his coworkers (1998) proposed an analytical model to compute the shear strength of the RC beam strengthened by FRP, and the results were once again calibrated and developed by Triantafillou and Antonopoulos (2000) as shown in Eq. 2.2.
where,

\( E_t \): Elastic modulus of FRP plate.

\( f_c \): Compressive strength of the concrete

\( \varepsilon_{fe} \): Effective strain of the FRP (Uwrap)

\( \varepsilon_{fu} \): Ultimate rupture strain of the FRP (Uwrap)

\( \rho_f \): FRP volumetric ratio to the body of the concrete

\( f_c \): Concrete compressive strength (MPa)


The design equations developed by ACI 440-2R (2008) adopt findings from Triantafillou (1998) and Khalifa and his coworkers (1998). The Uwrap in this study belongs to the category of “two or three sides laminated with CFRP” in ACI equations. It is interesting to note that both two and three sides laminated with CFRP use the same equation. These indicate that the developed equation by ACI did not take into account for the anchor effects from the Uwrap. Shear strengthened beams mostly show flexural failure, while
non-shear strengthened beams show brittle shear failure due to lack of shear capacity (e.g. Chaallal et al. 1998, Cao et al. 2005, Mosallam et al. 2006, Monti and Liotta 2006). Therefore, both shear and flexural capacity should be increased at the same time. It is also found from an existing study that shear capacity was increased when the three sides laminated with CFRP (Uwrap) was used rather than two sides laminated with CFRP (e.g. Monti and Liotta 2006). Furthermore, the beam with the 45 degree orientation of the Uwrap has shown a higher shear capacity than that with the normal 90 degree orientation of the Uwrap (e.g. Monti and Liotta 2006).

2.2.2 The anchoring effect of the FRP Uwrap

Sharif et al. (1994) used Uwrap as an anchor system. In this study, for a beam without the Uwrap, premature debonding occurred as a final failure due to no anchoring effect. When steel bolts were used for anchoring FRP plates at both ends, large diagonal tension cracks took place. The best anchoring effects were obtained from the Uwrap system. With the addition of Uwrap, plate separation and diagonal tension cracks did not develop; instead, the flexural capacity and ductile behavior of the RC beam were increased up to 24% and 92% respectively. It is interesting to note that the addition of the Uwrap increased the ductility of the beam significantly. Failure mode was changed from plate debonding to the flexural failure by concrete crushing at the top of the beam.

This strengthening effect and benefits due to the addition of Uwrap can be also found in the studied by Takahashi et al. (1999) and Brena et al. (2003). From these studies, it was concluded that placing the U-wraps in high moment regions within the
shear span is more effective than near the span’s end. This also indicates that the peeling (debonding) of the CFRP sheet in the constant moment region would be occurring first and propagated to the supports. It was also found that resistance strength to peeling (debonding) failure could be increased if the development length of the U-wrap is sufficient.

2.2.3 Numerical modeling of the Uwrap

Few studies have been developed that are related to the numerical modeling of the Uwrap on the FRP strengthened beam. Baky et al. (2007) and Kotynia et al. (2008) have conducted analytical studies on the modeling of the Uwraps. The global behavior of the beam obtained from the numerical study followed the trends of the experimental results; however, localized behavior such as strain gage data was not compared and presented. These studies represent the Uwrap behavior based on both 3-D and 2-D models. For the 3-D models, coarse mesh was used, and so the study on the localized behavior was very limited.

In these studies, it has been attempted to model the Uwrap in two dimensions (2D) as it was simply modeled by adding the contact area of the side bonded sheets to the interface elements between the concrete and the FRP sheet at the locations of the Uwrap. However, this assumption is too simple to replicate the real behavior of Uwrap since Uwraps are not attached to the soffit of the beam but transversely attached at the sides of the beam. Therefore the bonding behavior of the Uwrap would be different than that of
the longitudinal FRP sheet. Furthermore, the flexural-shear crack could affect the bonding behavior of the Uwrap. Further research is also recommended for understanding the influence of transverse FRP (Uwrap) on the debonding strain of longitudinal FRP (e.g. ACI 440-2R-08). In order to understand the influence of Uwrap on the strengthened beam, there is a need for developing analytical methods for the Uwrap system.

2.3 Digital Image Correlation (DIC)

In this study, the DIC technique was used to measure the amount of slip along the FRP plate. Therefore, background information related to the DIC technique is summarized in this section. A theory of the DIC technique in detail is well described in a published paper by Tay et al. (2004). This measuring technique is a non-contact optical technique for the displacement and strain measurement. Recently, this technique is becoming more prominent since high resolution digital cameras have been continuously developed, and digital data processing speed has also increased with newly developed software. This technique can measure two and three dimensional deformations of observed materials; however, in this study, plane deformation (2-dimensional deformation) will be only considered. The basic set up for this technique is described in Figure 2-2.
As Figure 2-2 shows, randomly speckled surface preparation is one of the important features for the DIC technique. This speckled patterns works as an information carrier when data correlation is processed. A set of neighboring points in an undeformed state is assumed to remain as neighboring points after deformation. In Figure 2-3, $S$ is the undeformed subset and $S_I$ is the deformed subset. Each block indicates each pixel in an image. If a subset is sufficiently small, the coordinate of $S_I$ can be estimated by first order Taylor expansion. Eq. 2.3 and Eq. 2.4 show the coordinate of $S_I$ by using first order Taylor expansion.

$$x_{ni} = x_m + u_m + \left(1 + \left.\frac{\partial u}{\partial x}\right|_M\right) \cdot \Delta x + \left.\frac{\partial u}{\partial y}\right|_M \cdot \Delta y$$  \hspace{1cm} 2.3
\[ y_{n1} = y_m + v_m + \left( 1 + \left| \frac{\partial y}{\partial y_{M}} \right| \right) \cdot \Delta y + \left| \frac{\partial v}{\partial x_{M}} \right| \cdot \Delta x \]

Where

\( u_m, v_m \) : Real displacements

\( \frac{\partial u}{\partial x_m}, \frac{\partial u}{\partial y_m}, \frac{\partial v}{\partial x_m}, \frac{\partial v}{\partial y_m} \) : the displacement derivatives of point M

Point M can be calculated using only displacement \( u_m \) and \( v_m \). However, Point N should be interpolated using first order Taylor expansion.

---

**Figure 2-3:** Sketch of digital image correlation (Tay et al. 2005)

For subset \( S \), coefficient \( C \) is defined as shown in Eq. 2.5.
Where

\[ C = \frac{\sum_{n = 1}^{n_s} \left[ f(x_n, y_n) - f_d(x_{nl}, y_{nl}) \right]^2}{\sum_{n = 1}^{n_s} \left[ f(x_n, y_n) \right]^2} \]  \text{Eq. 2.5}

\( C \): Correlation coefficient

\( f(x_n, y_n) \): Grey value distribution of the undeformed image

\( f_d(x_{nl}, y_{nl}) \): Grey value distribution of the deformed image

Function \( F \) in Eq. 2.5 is the gray value from 0 to 255 (e.g. Tay et al. 2005). This value could be changed as it is deformed. As the equations shows, for more accurate results, minimizing the \( C \) value would be an important process. For point \( M \), the \( C \) value becomes zero.

In order to use the DIC technique, a speckled pattern should cover the surface to be measured. Lecompte and his coworkers (2006) studied the influence of the size of the speckles and size of the subset as related to the accuracy of measuring. The subset size is the size of the selected parts of the image which will be correlated. It was proven from this study that a larger subset size shows better results when the speckle size is large. However, when the subset size is small, increasing the speckle size did not show appropriate results. Increasing the subset size would involve more computational time. This DIC technique could be also used for measuring bridge deflection (e.g. Yoneyama et al. 2007). Random shaped magnet plates simply were attached on the I-girder of the bridge and were used to measure the deflection using the DIC technique when a heavy cargo truck weighing 20,000 kg and having a length of 7 m was stopped at various positions on the bridge. The attached magnet plates were used as a speckled pattern since
the bridge scale is much larger than the laboratory material size. Results using the random shaped magnet as a speckled pattern agreed well with the deflection obtained from transducer.

2.4 Theoretical background of the material models needed for the numerical analysis

This study aims to characterize the Uwrap effects on the debonding mechanisms and on the overall behavior of RC-beams strengthened with externally bonded FRP sheets. Therefore, it is important to determine which material properties are to be determined and which simplification steps are to be taken into account in order to develop efficient and strong numerical models. The FRP strengthened RC beams are composed of five different materials: reinforcing rebar (steel), concrete, the FRP sheet, epoxy and the concrete-epoxy interface. Therefore, material constitutive laws and material properties for those materials are required parameters. The constitutive laws and material properties will be used in finite element models in order to predict the behavior of the FRP strengthened T-beam.

Interface behaviors between the rebar and surrounding concrete were not considered in this study since the development length was long enough based on ACI 318-08. This study aimed at the mechanical behavior of concrete epoxy interface with the addition of the FRP sheet and FRP Uwrap. Therefore, predicting yielding strength of the beam is out of scope. Instead, predicting the maximum failure loads due to the fracture of the FRP sheet or premature debonding of the FRP sheet is the focus. Therefore, a localized effect, such as the local slip between the rebar and concrete were neglected for
the simplification of developed models in this study. Gerstle and Ingraffea (1991) also found that the bond slip behavior between the concrete and rebar is governed by the fracture behavior of the concrete rather than by the interface properties. The debonding behavior between the concrete and FRP was the focus of the developed FE models in order to predict the debonding failure of the FRP sheet (Uwrap) or the rupture of the FRP sheet (Uwrap).

Figure 2-4 shows a conceptual drawing for the possible debonded area in the RC beam strengthened by the FRP sheet and Uwrap. It also shows the fiber direction of the Uwrap, region of the stress concentration and the pressures generated from the strained Uwrap due to the deformation between the FRP and concrete on the debonded region.

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**Figure 2-4:** A conceptual drawing of the debonded area in the beam strengthened by the FRP sheet and Uwrap
The first material model for the numerical analysis is the damage-plasticity model. This material model will be mainly used for the concrete and in some cases for the concrete-epoxy interface. A specific explanation will be given in a Section 2.4.1.

The second important material model is the damage model for the interface between the concrete and FRP. This damage model represents the interface behavior between the concrete and FRP/Uwrap. This is important since the major failure mode between the concrete and FRP (Uwrap) is debonding failure. This debonding failure is related to the damage model, and the damage model is related to the fracture mechanics. Therefore, fracture energy and the onset of the damage are important parameters to be determined for the damage model. The specific theory will be explained in Section 2.4.2.

Another important material model will be used for the unidirectional FRP. For the longitudinal FRP sheet, orthotropic FRP properties are not important since the FRP sheet is stressed mainly in the fiber direction. Therefore, it is often modeled with isotropic material properties, assuming that the orthogonal properties could be negligible (e.g. Coronado and Lopez 2006, Baky et al 2007). However, in the case of FRP Uwrap, it must be considered as an orthotropic material. Accordingly, the material properties related to the shear and transverse direction should be also taken into account for this case.

Furthermore, the lower region of the stress concentration should be taken into account as shown in Figure 2-4. It is thought that a smaller corner radius around the corner area of the Uwrap can significantly reduce the ultimate strength of the Uwrap due to stress concentration (e.g. Yang et al. 2001, Campione and Miraglia 2001). The reduced strength of the Uwrap at the corner region will be fractured much earlier than it should be.
due to the stress concentration effect. Therefore, studies related to the stress concentration effect were reviewed and a relevant equation was adopted for this study.

Studies related to the concrete friction models are also reviewed. The friction would be generated between the debonded FRP sheet and the debonded concrete surface as the Uwrap generates the pressures onto those two debonded surfaces as shown in Figure 2-4. This study found that the frictional behavior could not be neglected. If the friction effects are ignored, numerical results underestimate the strengthened behavior which comes from the addition of the Uwrap. Accordingly, a review of the frictional models was conducted, and a friction bond-slip model was developed for the study which will be explained in detail in Chapter 4.

2.4.1 Concrete material modeling

Crack initiation and its evolution is one of the most important failure modes in concrete structures. This concrete cracking behavior is not just a sudden onset of a new free surface as in brittle failure mode of ceramic or glass. A concrete cracking behavior is more related to the formation of continuous micro-cracks and the corresponding interlocking mechanism. These formations of continuous micro-cracks do not allow the stress to drop to zero instantaneously. Therefore, a model was developed to consider softening behavior after the onset of the crack.

The softening behavior of the concrete was first modeled by Scanlon and Murray (1974). In their study, concrete softening was modeled in terms of the degraded concrete
modulus. Vebo and Ghali (1977) also proposed concrete softening in terms of the linear and bilinear descending branch in order to analyze the concrete slabs.

Subsequently, other researchers started to use the softening behavior of the concrete in order to develop a model and predict the behavior of the concrete in tension more precisely. (e.g. Lee and Fenvas 1998, Mehta and Monteiro 1993, Coronado 2006).

This softening behavior of concrete from micro crack formation is also related to pressure sensitivity, inelastic dilatancy, strain softening, and path dependency, resulting in a highly inelastic material behavior (e.g. Kang and William 1999).

To deal with these complicated analytical behaviors induced from the micro crack formation, a continuum damage model was developed. Based on continuum damage mechanics, Mazars and Pijaudier-Cabot (1989) developed a concrete model. It was called the continuum damage model. This continuum damage model does not consider each individual crack but an internal state variable ‘d’, which is a damage in the material. The continuum damage model is based on Eq. 2.6 (e.g. Chen 1991).

\[
\sigma^* = \frac{1}{(1-d)} \sigma
\]

Where

\(\sigma^*\) : Effective stress

\(\sigma\) : Nominal stress

\(d\) : Scalar damage variable

The effective stress is defined as “the average micro-level stress acting in the undamaged material between defects. It is equivalent to the applied force divided by the
undamaged part of the area. Nominal stress is the macro-level stress and is defined as the applied force divided by the total area (e.g. Grassl and Jirasek 2006).

For some orthotropic damage models of concrete, damage is defined as a combination of two damage variables ($d_1$ and $d_2$) for two directions as shown in Eq. 2.7 (e.g. Calayir and Karaton 2005).

\[
\begin{bmatrix}
\sigma_1^* \\
\sigma_2^* \\
\sigma_3^*
\end{bmatrix} =
\begin{bmatrix}
\frac{1}{1-d_1} & 0 & 0 \\
0 & \frac{1}{1-d_2} & 0 \\
0 & 0 & \frac{1}{2} \left( \frac{1}{(1-d_1)^2} + \frac{1}{(1-d_2)^2} \right)
\end{bmatrix}
\begin{bmatrix}
\sigma_1 \\
\sigma_2 \\
\sigma_3
\end{bmatrix}
\]

However, this continuum damage model could not support the deformations such as plastic strain, which is related to the plastic flow of concrete. Consequently, the continuum damage concrete model could not predict dilatancy (e.g. Lee and Fevans 1998, Grassl and Jirasek 2006), which is an important factor especially for concrete structures.

In order to predict the dilatancy, a plastic model was adopted and added to the continuum damage concrete model (e.g. Simo and Taylor 1985, Lubliner et al. 1989, Lee and Fenves 1998, Grassl et al. 2002, Grassl and Jirasek 2006). The inelastic deformation due to microcracks can be estimated using a plastic flow potential function. The plastic flow potential determines the direction of plastic flow in terms of the gradient which controls inelastic dilatancy in terms of the inelastic deformation behavior of a material (e.g. Kang and Williams 1999).
Normally, for concrete, the plastic flow rule is defined by the hyperbolic flow potential as shown in Eq. \ref{eq:2.8}. Furthermore, a non-associated flow was assumed, which means that the yield function \( Y \) and the plastic potential \( G \) do not coincide and, therefore, the direction of the plastic flow is not normal to the yield surface (e.g. Lee and Fenves 1998, Grassl and Jirasek 2006).

\[
G = \sqrt{(\epsilon, \sigma, \tan \Psi)^2 + (R_{mw} q)^2} - p \tan \Psi \tag{2.8}
\]

where

\( R_{mw} \): Elliptical function proposed by Menétrey-Willam

\( \epsilon \): Meridional eccentricity (this defines the rate at which the hyperbolic function approaches the asymptote)

\( \sigma \): Uniaxial tensile stress at failure

\( \Psi \): Dilation angle (friction angle) defined from \( p-q \) plane

\( p \): Equivalent pressure stress (composed of hydrostatic pressure stress)

\( q \): Mises equivalent effective stress (composed of effective stress deviator)

This hyperbolic flow potential could be defined in the meridian plane as well as in the deviatoric plane. The meridian plane is in two dimensions consisting of equivalent pressure stress and Mises equivalent stress. This flow potential function is drawn in these two planes as shown in Figure 2-5. The associated flow rule normally causes unrealistically high volumetric expansion in compression which leads to the overestimated strength of concrete (e.g. Grassil et al. 2002).
According to the Abaqus manual, the shape of the yield curve in a deviatoric plane (b) is controlled by using the variation $K$ (e.g. Hibbitt et al. 2007). The variation $K$ is defined as the ratio of the second stress invariant on the tensile meridian to that on the compressive meridian at the initial yield for any given value of the pressure invariant $p$.

Figure 2-5: Potential flow in meridian plane (a) and its evolution (c) and deviatoric plane (b) and its evolution (d)
This $K$ is also called deviatoric eccentricity. This value is in the range of 0.5 to 1.

Therefore, controlling the $K$ value could change the modeling results (e.g. Hibbitt et al. 2007). For example, if the $K$ value is set to zero, this represents the Rankine failure criterion which was developed to consider tension dominated failure of the concrete. However, this plastic model could not control the stiffness degradation of the concrete as the microcracks form.

Consequently, the previously explained continuum damage model and continuum plastic model were combined in order to enrich each other. The combination of this plasticity of the concrete model with the continuum damage model results in a well-known concrete model, which is called the plastic-damage model.

Lubliner and his co-workers (1989) in particular used fracture energy based scalar damage variables. These variables are coupled with the plastic deformation (strain) of the concrete to form constitutive equations. The main improvement of this model from the previous plastic damage model was that the calibration of parameters became easier since the damage variable was coupled with plastic deformation instead of decoupling from plastic deformation. Eq. 2.9 shows the stress-strain relation for the model.

$$\sigma = (1-d)\sigma^* = (1-d)D_e (\varepsilon - \varepsilon_p)$$  \hspace{1cm} 2.9

Where:

$\sigma^*$: Effective stress

$\sigma$: Nominal stress

$d$: Scalar damage variable

$D_e$: Elastic stiffness
\( \varepsilon \): Total strain

\( \varepsilon_p \): Plastic strain

However, this model still was not able to predict the cyclic behavior of concrete well. Since the concrete under the cyclic loading could have both tensile cracking and compression failure, one damage variable to predict the concrete behavior under the cyclic load could not be an adequate model.

Accordingly, Lee and Fenves (1998) developed two damage variables to account for the different damage in tension and in compression. The concrete damage-plasticity model used in the Abaqus program was developed based on these two variables developed by Lee and Fenves (1998), which are based on the fracture scalar damage variables developed by Lublier and his coauthors (1989).

### 2.4.2 Material modeling of the interface using a cohesive element

As previously explained, for modeling of the interface, the damage model should take into account based on the fracture mechanics between the concrete and FRP. The damage model for the concrete can be modeled with the previously explained damage plasticity model. Likewise, the damage process for the interface also could be modeled with the damage plasticity model, which is a continuum based model. However, the continuum based damage plasticity model in Abaqus could not be used for the interface for this study. The first reason is that the deformation of the interface element in Abaqus could extremely deflate after the complete failure (debonding) since the elements could
not be removed even after the complete failure in the program of Abaqus. Furthermore, a minimum residual stress could not be removed after the complete failure. Therefore, even though the stress should not exist after the complete debonding, there will still be a minimum residual stress defined by Abaqus, resulting in the erroneous behavior. If the analysis is only a focus before the debonding between the concrete and FRP, this damage-plasticity model could be also used for the material model of the interface. However, this study will focus on both the behavior before and after the debonding, indicating that the damage plasticity model would not be appropriate.

Alternatively, a cohesive element was selected instead of using the continuum element based damage-plasticity model. For the cohesive element, a constitutive model specified directly in terms of the traction versus separation rather than the continuum mechanics was used. Abaqus ver. 6.8-1 offers a “cohesive element” in order to deal with the constitutive model which is specified directly in terms of traction versus separation. This cohesive element is a useful tool for modeling adhesive, bonded interfaces, and gasket problems.

This cohesive element assumes that the material such as concrete-epoxy interface (CEI) is subjected to only three direct components of strain. One is through thickness strain, and the other two are through transverse shear strains. In the two dimensions, one will be through the thickness strain and another will be a transverse shear strain.

In Abaqus, there are two other different constitutive laws that could be used in cohesive elements other than the traction-separation behavior. One is related to continuum mechanics, while the other is related to the uniaxial stress state. For the modeling gasket problem, uniaxial stress state based modeling would be appropriate. If
the interface or adhesives have a finite thickness compared with the surrounded elements, often a continuum based cohesive element is used. However, if the thickness of the interface is infinitely small compared to the other surrounding elements like the concrete-FRP interface, a traction-separation based cohesive modeling would be a suitable method. In order to deal with the concrete-FRP interface for this study, a traction-separation based cohesive model was selected.

This model offers a variety of advantages for dealing with the concrete-FRP interface problem. First of all, a delamination between the concrete and FRP can be characterized using this model. Second, the Mode 1 and Mode 2 damage behavior (fracture mechanics) can be put into one element. This indicates, in other words, that the cohesive element could behave for both directions. Furthermore, a mixed mode behavior for these two modes can be characterized as tabulating the experimental data into the input files. There are some limitations, however, that the linear elastic behavior prior to initiation of damage is the only option for the constitutive law. Any other non-linear behavior up until onset of damage is not possible for this element. This linear elastic traction-separation law before the onset of damage can be written as shown in Eq. 2.10.

\[
\begin{bmatrix}
    t_n \\
    t_s \\
    t_t
\end{bmatrix} =
\begin{bmatrix}
    E_{nn} & E_{ns} & E_{nt} \\
    E_{ns} & E_{ss} & E_{st} \\
    E_{nt} & E_{st} & E_{tt}
\end{bmatrix}
\begin{bmatrix}
    \varepsilon_n \\
    \varepsilon_s \\
    \varepsilon_t
\end{bmatrix}
\]

2.10

Where

\( t \) : Nominal stress (\( n \): normal traction stress, \( s,t \): shear traction stress)

\( E \) : Modulus that relates stress to strain

\( \varepsilon \) : Strain
This $E$ matrix shows the coupled behavior as shear components are engaged. For the case of the uncoupled behavior, all shear components in the matrix are vanished, resulting in a diagonal matrix. However often, a small thickness of cohesive elements introduces very large material stiffness, decreasing the stable time increments and ill conditioning element operations in the model. Therefore, often regardless of the initial cohesive element thickness, it is assumed to be “1”. Based on this artificial thickness of the cohesive element, the strain of the cohesive element becomes the same value of the corresponding separation. Since the thickness of the cohesive element is changed artificially, modulus ($E$), defined in Eq. 2.10, must be adjusted for this case in order to maintain same stiffness ($K$). For example, if the thickness of the cohesive element is specified as a 1, while its real initial thickness($t$) is 1E-03 and stiffness($K$) is 1E+9, modulus ($E$), defined in Eq. 2.10 becomes 1E+09. The actual modulus ($E$) is 1E+6 since its real thickness is 1E-03. It can be simply expressed as shown in Eq. 2.11. This is a useful trick. As the initial thickness is set to 1, the separation and strain number become the same number. At the same time, it brings stable solutions and prevents the ill conditioning of the element.

$$Stiffness(K) = \frac{Area(A) \times Modulus(E)}{Thickness(t)}$$

$$\Rightarrow Stiffness(K) = \frac{Area(A) \times 1E + 6}{1E - 03} = \frac{Area(A) \times 1E + 9}{1}$$

For the cohesive element, once it reaches peak values (onset of damage), material stress starts to decrease as it is now in the softening region as shown in Figure 2-6. This damage initiation could be simply determined if the pure Mode 1 or Mode 2 is only associated with the model. However, if both the normal and shear traction are coupled at
the same time, it demands a certain criteria to define the coupled damage initiation from both the normal and shear direction.

In Abaqus, there are many available methods to deal with this problem. In this study, a quadratic equation is used to define damage initiation as shown in Eq. 2.12

\[
\left(\frac{t_n}{t_{nc}}\right)^2 + \left(\frac{t_s}{t_{sc}}\right)^2 = 1 \tag{2.12}
\]

Where

\( t_n \) : Nominal stress in normal direction

\( t_s \) : Shear stress in shear direction

\( t_{nc} \) : Maximum nominal stress in normal direction

\( t_{sc} \) : Maximum shear stress in normal direction

Figure 2-6: Constitutive laws for cohesive element: Normal direction (a), Shear direction (b)
Other criteria are also available, such as maximum stress criteria and strain related
criteria, rather than stress related. In the cohesive element, there is an assumption that
compression in the normal direction does not initiate any damage.

Once it reaches the maximum stress, the material begins to be degraded as shown
in Figure 2-6. To deal with this softening behavior, Abaqus uses the damage variable \( D \). The \( D \) is monotonically evolved from 0 to 1, as separation is close to total separation \( \delta_n', \delta_s' \). The relation between the damage variable and stress are defined in Eq. 2.13.

\[
\begin{align*}
t_n &= (1 - D)t_{nc} \\
t_s &= (1 - D)t_{sc}
\end{align*}
\]

Where

\( t_n \) : Nominal stress in normal direction

\( t_s \) : Shear stress in shear direction

\( t_{nc} \) : Maximum nominal stress in normal direction

\( t_{sc} \) : Maximum shear stress in normal direction

\( D \) : Damage variable

These damage variables are determined based on the combination of normal and
shear deformation across the interface called effective deformation (e.g. Camanho and
Davila, 2002). This effective deformation \( \delta_m \) between the normal and shear mode is
defined as shown in Eq. 2.14.

\[
\delta_m = \sqrt{\delta_n^2 + \delta_s^2}
\]
Where

\( \delta_m \): Effective deformation

\( \delta_n \): Deformation in normal direction

\( \delta_s \): Deformation in shear direction (Slip)

This cohesive element will be used for this study. The interface fracture properties obtained from experiments in Mode 1 and Mode 2 will be incorporated into the cohesive elements. Fracture properties based on the Mode 3 deformation will not be considered in this study since it is assumed that debonding due to this type of loading such as pull-out test, T-beam strengthened for flexure test would not occur in the geometries used here rarely generate Mode 3 debonding failure between the concrete and the FRP sheet. It is possible that, some torsional loads between the body of the concrete and the FRP sheet can generate a Mode 3 dominant behavior. In this study, these loading types were not used.

2.4.3 Failure theory of an unidirectional lamina

A failure criteria for the unidirectional composite in the fiber direction is quite simple since most of the failure experimentally observed is elastic-brittle failure. Therefore, the longitudinal unidirectional FRP laminate can be analyzed easily if the elastic modulus and tensile strength are known. However, in the case of the Uwrap, the fiber direction is transverse to the longitudinal direction of the concrete beam. Accordingly, any type of loading such as tensile, shear, and compression could be
occurring on the Uwrap. This indicates that an anisotropic failure criteria for the unidirectional composite is needed for analyzing the Uwrap behaviors in the model. The failure criteria for the unidirectional material were pioneered by the Tsai-Hill theory developed in 1965. This theory was originally developed for homogeneous anisotropic ductile materials (e.g. Azzi and Tsai, 1965). Daniel and Ishai (2006) classified failure theory into three categories. One is limit or non-interactive theory. This theory defines that specific failure mode could be predicted by each individual stress or strain with corresponding strengths or ultimate strains. Maximum stress and maximum strain theory are included in this category. As known from the name, no interactive behaviors (no coupling behavior) among stresses are considered. However, max strain theory considered the interactive behavior. These failure criteria are straightforward and simple to use. It is also predicts well failure type and load.

The second method is the interactive theory. This method uses one equation to cover any type of failure. Hill (1948) modified the Von-Mises yield criterion for defining the anisotropic failure criteria. Afterwards, Azzi and Tsai (1965) further developed this using this anisotropic criterion.

Another attempt for the interactive failure criteria of anisotropic material is the Tsai-Wu theory (1971). Tsai and Wu modified the tensor polynomial theory. It was assumed that there is no coupling between normal stresses and shear strain and between shear stresses and shear strains. This assumption made the equation quite simple to use. For a two dimensional thin lamina theory, Tsai-Wu propose Eq. 2.15 for a two dimensional state of stress.
Another widely used failure criteria for the unidirectional FRP lamina is the mode based failure theory. Hashin and Rotem (1973) proposed this criterion based on the assumption that the failure of lamina under a general in-plane loading would be a fiber failure or an interfiber failure (matrix failure) as shown in Eq. 2.16.

\[ \left| \frac{\sigma_1}{F_1} \right| = 1 \]

\[ \left( \frac{\sigma_2}{F_2} \right)^2 + \left( \frac{\tau_6}{F_6} \right)^2 = 1 \]

Where

- \( F_1 \): Strength in fiber direction
- \( F_2 \): Transverse strength
- \( F_6 \): Shear strength of lamina.
- \( \sigma_1 \): Stress in fiber direction
- \( \sigma_2 \): Stress in transverse direction
- \( \tau_6 \): Shear stress in plane
From the FE program Abaqus, a failure criterion for the unidirectional FRP could be defined by the Hashin-Rotem failure criterion. Up to the failure of each mode, the linear elastic behavior was assumed, indicating that the shear, tension and compression behavior in any direction would behave elastically. For the material model of the unidirectional FRP sheet (Uwrap), the Hashin-Rotem failure criterion was used.

2.4.4 Damage evolution of the unidirectional FRP sheet

In this section, the damage evolution of the FRP laminate is summarized. Even though the failure mode of FRP is quite brittle, there is a softening behavior after the onset of the damage. The damage behavior of the longitudinal FRP sheet in a strengthened beam under uniaxial loading would not change the behavior of the strengthened beam significantly since brittle failure of the fiber is dominant. Therefore, damage evolution (fracture energy) is a minor issue if the failure is expected to be in the fiber direction. However, the energy dissipation of the Uwrap would affect the behavior of the strengthened beam since the progressive transverse and shear failure could be a dominant failure mode of the Uwrap rather than the single rupture of the fiber. In this case, the damage evolution in transverse and shear direction needs to be considered. For example, the laminate still can carry a certain amount of load even after the entire matrix is failed by shear or transverse load since the fiber is still not broken. This type of failure could occur in the shear dominant area such as some corner region which connects the side of the Uwrap to the bottom of the Uwrap. Therefore, damage evolution properties could be assigned for unidirectional composite laminate, especially for the Uwrap as
fracture properties for the Uwraps are used. For obtaining the fracture energy of the Uwrap, some literature was reviewed, and appropriate numbers were selected for the numerical analysis. This will be described in a Section 2.6.4.

Matzenmiller et al. (1995) made some assumptions for the mechanical model of the unidirectional composite that transverse and shear stresses have no influence on the damage in the fiber direction as far as the tensile load-carrying capacity of the fibers is concerned. This means the damage of the matrix has a secondary effect on the actual elastic properties of tension in the fiber direction. Therefore, the contribution of transverse and shear stresses to tensile fiber damage could be neglected. Based on this assumption, the damage behavior is defined as shown in Eq. 2.17 (e.g. Matzenmiller et al. 1995, Lapczyk and Hurtado 2007).

\[
C_d = \frac{1}{D} \begin{bmatrix}
(1 - d_f)E_i & (1 - d_f)(1 - d_m)\nu_{21} E_i & 0 \\
(1 - d_f)(1 - d_m)\nu_{12} E_1 & (1 - d_m)E_2 & 0 \\
0 & 0 & (1 - d_s)GD
\end{bmatrix}
\]

Eq. 2.17

where

\(\sigma\) : Stress
\(\varepsilon\) : Strain

\(C_d\) : Post-damage elastic behavior
\(d_f\) : Current state of fiber damage
\(d_m\) : Current state of matrix damage
\(d_s\) : Current state of shear damage
$E_1$: Elastic modulus in fiber direction

$E_2$: Elastic modulus in matrix direction

$v_{12}, v_{21}$: Poisson’s ratio.

$G$: Shear modulus

$D: 1 - (1 - d_f)(1 - d_m)v_{12}v_{21}$

The previously shown four damage parameters, $d$, are derived from the equations as shown in Eq. 2.18.

$$
\begin{align*}
    d_f &= \begin{cases} 
    d_f^t & \text{if } \hat{\sigma}_{11} \geq 0, \\
    d_f^c & \text{if } \hat{\sigma}_{11} < 0,
\end{cases} \\
    d_m &= \begin{cases} 
    d_m^t & \text{if } \hat{\sigma}_{22} \geq 0, \\
    d_m^c & \text{if } \hat{\sigma}_{22} < 0,
\end{cases} \\
    d_s &= 1 - (1 - d_f^t)(1 - d_f^c)(1 - d_m^t)(1 - d_m^c)
\end{align*}
$$

where

$\hat{\sigma}_{11}, \hat{\sigma}_{22}$: Effective stress tensor in fiber and transverse respectively

$\hat{\sigma}_{11}$ and $\hat{\sigma}_{22}$ are effective stress tensors. Instead of using normal stress, effective stress is used to calculate damage variables. The effective stress is defined as the stress acting over the damaged area (not a total area) that effectively resists the internal forces (e.g. Hibbitt et al. 2007).

For example, damage variables for fiber tension are calculated using Eq. 2.19.

$$
\begin{align*}
    d &= \frac{\delta_{eq}^f(\delta_{eq} - \delta_{eq}^0)}{\delta_{eq}(\delta_{eq}^f - \delta_{eq}^0)}
\end{align*}
$$
where

\( \delta_{eq}^0 \): the initial equivalent displacement at which the initiation criterion for that mode was met

\( \delta_{eq}^f \): the equivalent displacement at which the material is completely damaged in this failure mode

\( \delta_{eq} \): the displacement at which the material is between initiation of damage and complete damage

To calculated equivalent displacement, an energy based linear softening fracture behavior was used for the damage evolution of the Uwrap as shown in Figure 2-7.

---

**Figure 2-7:** Equivalent stress versus equivalent displacement

Each displacement and stress for four different damage models is calculated as shown in Eq. 2.20.
Fiber tension \((\sigma_{11} \geq 0)\)
\[
d_{eq}^f = L_c \sqrt{\langle \varepsilon_{11} \rangle^2 + \alpha \varepsilon_{12}^2},
\]
\[
\sigma_{eq}^f = \frac{\langle \sigma_{11} \rangle \langle \varepsilon_{11} \rangle + \alpha \tau_{12} \varepsilon_{12}}{\delta_{eq}^f / L_c},
\]

Fiber compression \((\sigma_{11} < 0)\):
\[
d_{eq}^c = L_c \langle -\varepsilon_{11} \rangle,
\]
\[
\sigma_{eq}^c = \frac{\langle -\sigma_{11} \rangle \langle -\varepsilon_{11} \rangle}{\delta_{eq}^c / L_c},
\]

Matrix tension \((\sigma_{22} \geq 0)\)
\[
d_{eq}^{mt} = L_c \sqrt{\langle \varepsilon_{22} \rangle^2 + \varepsilon_{12}^2},
\]
\[
\sigma_{eq}^{mt} = \frac{\langle \sigma_{22} \rangle \langle \varepsilon_{22} \rangle + \tau_{12} \varepsilon_{12}}{\delta_{eq}^{mt} / L_c},
\]

Matrix compression \((\sigma_{22} < 0)\):
\[
d_{eq}^{mc} = L_c \sqrt{\langle -\varepsilon_{22} \rangle^2 + \varepsilon_{12}^2},
\]
\[
\sigma_{eq}^{mc} = \frac{\langle -\sigma_{22} \rangle \langle -\varepsilon_{22} \rangle + \tau_{12} \varepsilon_{12}}{\delta_{eq}^{mc} / L_c},
\]

where

\(L_c\): Characteristic length (square root of the element area).

\(\alpha\): Parameter to contribute fiber tension damage initiation by shear strain

The symbols \(\langle \, \rangle\) in the equations above are the Macaulay bracket operator. By changing parameter, \(\alpha\), shear contribution to the initiation of fiber tension can be adjusted from 0 to 1. For example, by setting this parameter to zero indicates that shear and fiber tension failure are not coupled for initiation of the fiber tension damage. If it is one, the shear strain is totally related to the damage initiation of the fiber tension.

This damage model for the unidirectional laminate was used for the FRP sheet and FRP Uwrap. All other details can be found in the Abaqus 6.8-1 theory manual (e.g. Hibbitt et al. 2007). Corresponding material properties used for this study could be found in Chapter 4.
2.5 Additional factors which control the FRP Uwrap behavior

In this section, additional factors which will govern the Uwrap behavior will be discussed. One is the shear concentration factor. This is important especially at the corner area of the Uwrap since a smaller corner radius can significantly reduce the ultimate strength of the FRP laminate due to stress concentration.

Another factor is the friction between the two debonded surfaces under the Uwrap location. As experimentally observed from the pull-out test, it was found that the friction factor should not be neglected. The Uwrap holds the interface between the concrete and FRP even after the debonding of the FRP sheet occurs. A specific theory related to these factors and developed equations for this study will be explained in next sections.

2.5.1 Stress concentration

As previously seen from the preliminary test and pull-out test, the fracture of the Uwrap mostly occurs at the corner region, and it is considered to be stress concentration effects at the corner. Therefore, the stress concentration factor should be considered and selected for this study. The stress concentration effect on the FRP sheet was studied by several researchers (e.g. Yang et al. 2001, Campione and Miraglia 2001). From these studies, as expected, a smaller corner radius can significantly reduce the ultimate strength of the FRP laminate due to stress concentration (e.g. Yang et al. 2001). Some equations were also developed to consider the stress concentration of the FRP laminate at the small corner radius. Campione and Miraglia (2001) developed the stress concentration factor based on the concrete block size \( b_d \) and corner radius \( r \). An equation developed for
calculating the stress concentration factor between the FRP laminate and the small corner radius of the concrete is shown in Eq. 2.21.

\[
f_r = f_u \left[ \left(1 - \frac{\sqrt{2}}{2} k_i \right) \frac{2r}{b_d} + k_i \frac{\sqrt{2}}{2} \right]
\]

Where,

- \( f_r \): Stress in FRP at corner
- \( f_u \): Ultimate strength of FRP
- \( k_i \): Reduction factor of FRP stress due to the shape of cross section
- \( B_d \): Concrete dimension (center to the FRP perimeter)

Based on this developed equation, the stress concentration factor was estimated and put into an 2D finite element model. The calculated stress concentration factor for Uwraps placed at an angle of 90 and Uwraps placed at an angle of 45 degree has 3.12 and 2.56, respectively. The calculated stress concentration factor could be different for the Normal Uwrap and the 45 degree Uwrap.

2.5.2 The frictional effect

From the study of the Uwrap, it was found that the frictional behavior between the debonded FRP sheet and debonded surface of the concrete would be a critical factor to control the behavior of the beam strengthened by the FRP sheet with the Uwrap. For this study, a couple of shear friction models were also reviewed.
In this study, the contribution of the friction to shear-transfer resistance needs to be well defined since frictional behavior between the roughened FRP surface and roughened concrete surface after the debonding would occur as both surfaces are registering to the shearing off of protrusions on the debonded faces. There are fracture micro-mechanical models for asperity interactions on fracture surfaces (e.g. Mroz 1996, Misra 2001). These asperity models were firstly proposed and developed by Patton (1966). These models consider the dilation of the concrete due to the fracture. It was found from the study that the dilatancy plays an important role in the analysis of a jointed rock mass and cannot be neglected (e.g. Mroz 1996). Two concrete bodies slide relative to each other along asperities thus inducing a dilatancy and a contraction for a reverse sliding. These studies have contributed to the modeling of material damage, fracture propagation under shear and compression, and mechanics of jointed rock masses (e.g. Misra 2001).

One the other hand, there are some empirical models to predict the friction behavior between two cracked surfaces of the concrete. The equations developed (e.g. Mattock 1976, Mattock 2001, ACI 318) are mostly developed for shear friction due to the clamping forces of steel rebar. Recently, an equation was developed in order to obtain the shear friction strength due to the clamping forces of FRP between two concrete surfaces. A developed equation for the shear friction strength due to the CFRP interaction is shown below (e.g. Saenz and Pantelides 2005).

\[ V_n = k_1 \rho_f f_{f_m}^* + k_2 f_c^* \] \hspace{1cm} 2.22

Where,
\( k_1 \): frictional coefficient 1 (0.505)

\( k_2 \): frictional coefficient 2 (0.117)

\( V_n \): Shear strength,

\( \rho_f \): FRP volumetric ratio to the body of the concrete

\( f_{fu}^* \): Effective CFRP composite tensile stress

\( f_c \): Concrete compressive strength

The first term in Eq. 2.22 represents the contribution of friction to shear-transfer resistance and the second term characterizes the resistance to shearing of protrusions on the crack faces. In ACI 318(11-7), shear friction coefficient 0.8 is recommended. To estimate shear friction between two debonded concrete surfaces and the FRP surface, the equation shown above could be used after some modifications. The equation is an empirical equation obtained from experimental works from the concrete to concrete surface due to the clamping force of the CFRP sheet. For the frictional behavior between the two debonded surfaces between the FRP sheet and concrete, the friction would be different. Consequently, Eq. 2.22 was modified based on calibrations of the numerical analysis with the experiments. Developed equations based on Eq. 2.22 and the frictional behavior under the Uwrap location is explained in Chapter 4.
2.6 Determination of material properties for the numerical models

In this section, the procedure to determine important material properties will be explained. For the concrete damage plasticity, inelastic compressive behavior and Mode 1 fracture behavior in tensile loading need to be obtained experimentally. For the cohesive element, both Mode 1 and Mode 2 fracture behavior of the interface between the concrete and FRP needs to be determined. First, in this section, important procedures to obtain Mode 1 fracture energy will be explained in detail. Second, the fracture energy of the unidirectional FRP sheet and Uwrap in tension and shear loading needs to be defined after the onset of damage. For the fracture properties of the unidirectional FRP sheet and Uwrap, data was not obtained from the experiments; instead, data was referred from the selected reference. It should be noted that the fracture energy of the unidirectional FRP sheet is not a critical material property in this study. Interface fracture properties and concrete properties will mostly govern the overall behavior of the strengthened beam and pull-out specimens.

2.6.1 Determination of the Mode 1 fracture properties of the CEI

As previously explained, for the damage based model, the determination of fracture energy is a critical step. Based on the obtained fracture energy, softening curve is determined, and the interface behavior in Mode 1 and Mode 2 direction could be defined. Those variables should be obtained from experiments since those are dependent on the properties of the concrete, epoxy and FRP. A three-point bending test on the notched
beam has been used widely for obtaining the Mode 1 fracture energy of the concrete and concrete-epoxy interface (e.g. Qiao and Xu, 2004, Coronado and Lopez, 2008). Figure 2-8 shows the two required tests to determine the fracture energy, maximum tensile stress, and softening curves of the concrete and concrete-epoxy interface. The maximum tensile stress is obtained from the splitting tensile test (ASTM C496). For quality results of the splitting tensile test, sensitivity to the width of the bearing strip should be considered (e.g. Rocco et al., 2001, Coronado and Lopez, 2008).
It was found that a width of the bearing strip in the range of 4% to 8% to the diameter of the cylinder has shown that failure mechanism mainly involves the formation of the principle crack (e.g. Coronado and Lopez 2008). Therefore, the ratio of the width of the bearing strip to the diameter of the concrete cylinder was selected as 5% for this...
study. From the three-point bending test, the load-crack mouth opening curve is obtained as shown in Figure 2-8.

These obtained maximum tensile strength and softening curves of concrete and the concrete-epoxy interface are the required parameters to obtain the stress-crack opening (stress-displacement) material behavior of the concrete and concrete-epoxy interface as shown in Figure 2-9.

![Load-CMOD graph and bilinear stress-crack opening curves](image)

**Figure 2-9:** Load-CMOD graph and bilinear stress-crack opening curves

The stress-crack opening curve can be simplified as a bilinear softening curve as shown in Figure 2-9. This bilinear stress-crack opening is determined by three material properties: the tensile strength $f_t$, the size-effect fracture energy $G_f$, and the cohesive fracture energy $G_F$. Specific procedures and required properties are well described in some studies (e.g. Guinea et al. 1994 and Coronado and Lopez 2008). It is also important
to note that softening behavior can be also simplified as linear if the maximum stress and fracture energy, obtained from experiments, remain the same (Lu et al. 2005).

The size-effect fracture energy $G_f$ is equivalent to the area under the initial tangent of the softening curve. The cohesive fracture energy $G_F$ is the energy required to create and fully break a crack of unit length.

First, the size-effect fracture energy $G_f$ is calculated from those two tests as shown in Eq. 2.23. Variables such as $a_0$ and $D$ can be found in Figure 2-8.

$$f_p = \frac{P_{\text{max}} S}{2B(D-a_0)^2}$$

$$G_f = \frac{Df_t^2}{E} \left(1-a_0 \right)^{1.7} \left[ \frac{11.2}{(x^2-1)^2} + \frac{2.365}{x^2} \right]$$

where

$P_{\text{max}}$ : The corrected peak load from three-point bending tests

$f_p$ : Net plastic flexural strength

$a_0 = a_0 / D$

$x = f_t / f_p$

$B$ : width of the beam

$E$ : Elastic modulus of the concrete

The calculation of the corrected peak load ($P_{\text{max}}$) is well described in some studies (e.g. Bažant and Planas 1998, Coronado and Lopez 2008).
Figure 2-10: Calculation of the area underneath the load-displacement curve

Figure 2-10 shows the typical load-displacement curve of the notched beam test. The measured work of fracture is equivalent to the area under the curve between point A and B. Point B is the point where the test was stopped. Since the complete failure of the beam in a three-point bending test is approached asymptotically (e.g. Peterson 1981), the location of T could not be determined experimentally. For this reason, the test needed to be stopped at some point even before complete energy dissipation. Therefore, the measured work of fracture does not represent true fracture energy as area OABT is neglected. To obtain the area of OABT, point T should be known. In order to solve this difficulty, it was found by Guinea et al. (1992) using rigid body kinematics that the area of OABB’ identically equals to the area of B’BT (e.g. Petersson 1981). Based on this assumption, the final calculation of the cohesive fracture energy $G_F$ could be obtained as the following equations are used. (Eq. 2.24) The constant $A$, required was determined for each specimen by least-squares fitting. (e.g. Coronado and Lopez 2008)
\[ G_F = \frac{W_F}{B(D - a_0)} \]

\[ W_F = W_{FM} + \frac{2A}{(u_r - u_a)} \]

\[ P_{\text{max}} = P_r + \frac{A}{(u_r - u_a)^2} \]

where

- \( W_F \): Total work of fracture
- \( W_{FM} \): Measured work of fracture
- \( A \): Far tail constant
- \( B \): Width of the beam
- \( P_{\text{max}} \): Maximum load measured during the three-point bending test
- \( P_r \): Residual force acting in a three-point bending specimen after a failure
- \( u_r \): Displacement when the test is stopped
- \( u_a \): Displacement when the load is reached at \( P_r \)

### 2.6.2 Determination of the Mode 2 fracture properties of the CEI

More research for the characterization of the CEI interface behavior has been conducted under the Mode 2 behavior compared to that of the Mode 1 behavior. It was assumed that the debonding of concrete-FRP interface can be interpreted by the shear stress-slip behavior (e.g. Yuan et al. 2004, Yao et al. 2005, Lu et al. 2005, Wang, 2007). The most used test method for obtaining shear stress-slip behavior between the concrete
and FRP sheet is a single shear lap test because of its simplicity compared to other methods as shown in Figure 2-11 (e.g. Bizindavyi and Neale 1999, Yao et al. 2005). This test method was also used for obtaining bond strength of the CEI. (e.g. Chajes et al. 1996, Chen and Teng 2001). The shear stress (bond) could be obtained from the collected data of the load cell. The slip could be measured from the many techniques such as strain gages, LVDT, interferometer, and digital image correlation.

Figure 2-11: Typical single shear lap test under the Mode 2 loading set up

It was first attempted using the strain gages along the FRP plate in order to obtain bond-slip behavior under the Mode 2 loading, (e.g. Nakaba et al. 2001, Savioa et al. 2003, Lu et al. 2003, Ferracuti et al. 2007). The slip between strain gages could be calculated by strain differences between adjacent strain gages with constant distance (Δx) as shown in Figure 2-12 (a). This is simple and straightforward technique; however, some difficulties exist. First of all, it is difficult to get a smooth curve from this technique since concrete is a heterogeneous material. Often the strain gages could be located between the paste and aggregate, resulting in an unstable strain data. Furthermore, gage attachments with the short constant distance are tedious and difficult (e.g. Dai et al. 2005). Gages only
can be attached to one side of the FRP plate. Therefore, biased results could be obtained from the bending strain of the plate.

Figure 2-12: The measuring slip using strain gages (a) and using LVDTs (b)

Another technique (Figure 2-12 (b)) is developed based on relation between pullout loads and slips at the loaded end in a pullout test (e.g. Dai et al. 2005). This is a simple and rigorous test technique to determine concrete-epoxy interface under the Mode 2 loading since not many strain gages are attached to the FRP plate. This technique measures deformation at the loaded end from the pull-out load. It uses the unique relationship between the strain of FRP sheet and the corresponding slip (e.g. Shima et al. 1987) as shown in Eq. 2.25.

\[
\varepsilon = f(s) = A(1 - \exp(-Bs))
\]

where

\( \varepsilon \): the strain of FRP plate

\( s \): the slip at loaded end

\( A, B \): the constant (obtained from the curve fitting of the experimental results)
\( f(s) \): the function of the corresponding slip (s) at loaded end

Differentiating Eq. 2.25 results in Eq. 2.26.

\[
\frac{d\varepsilon}{dx} = \frac{df(s)}{ds} \frac{ds}{dx} = \frac{df(s)}{ds} \varepsilon = \frac{df(s)}{ds} f(s)
\]

2.26

Accordingly, bond strength can be shown as Eq. 2.27

\[
\tau = E_f t_f \frac{d\varepsilon}{dx} = E_f t_f \frac{df(s)}{ds} f(s)
\]

2.27

where

\( \tau \): shear stress,

\( E_f \): Elastic modulus of FRP plate and

\( t_f \): Thickness of FRP plate

From Eq. 2.25, A and B values could be obtained from the curve fitting of the experimental results. For this study, this equations proposed by Dai et al. (2005) was used to characterize the CEI bond behavior under Mode 2 loading.

Recently, one technique is emerged to characterize CEI in Mode 2 direction.

Some researchers use Digital Image Correlation (DIC) to measure the deformation of the concrete (e.g. Choi and Shah 1997, Lawler et al. 2001). Some use DIC to measure the bond-slip behavior between the concrete and FRP (e.g. Ali-Ahmad et al. 2006 among others). The background theory about the DIC technique was already discussed in a previous section.
2.6.3 Compressive strength of the unidirectional FRP sheet

In order to determine the anisotropic material behavior of the unidirectional FRP sheet, the following material properties should be obtained: maximum longitudinal tensile stress, $f_l$, maximum longitudinal compressive stress, $f_c$, maximum transverse tensile stress, $f_t$, maximum transverse compression stress, $f_c$, elastic modulus of fiber $E_f$, elastic modulus of epoxy $E_m$, elastic modulus in longitudinal direction $E_1$, elastic modulus in transverse direction $E_2$, in plane maximum shear stress $\tau_{12}$ of the laminate, shear modulus in longitudinal-transverse direction $G_{12}$ and shear modulus in transverse-longitudinal direction $G_{21}$, and shear modulus of epoxy $G_m$. Out of plane properties were neglected in this study since the applied FRP sheet is thin. Among those, material properties such as $E_1, E_2, f_l, f_t, G_{12}$ and $\tau_{12}$ were obtained experimentally according to ASTM D3039 and ASTM D5379. Other properties such as $f_l, E_f$ and $E_m$ were obtained from manufacturer’s date sheet.

However, some of those were not obtained from experiments and manufacturer’s date sheet such as the compressive strength of the carbon FRP in both the longitudinal and transverse direction ($f_c, f_c$). Those values were not obtained from experiments; instead, they were estimated using the following equations described in Daniel and Ishai (2006).

Eq. 2.28 shows the equation for estimating the maximum longitudinal compression stress.
\[
f_{tc} = \frac{G_m}{(1 - V_f)}
\]

Eq. 2.29 shows the equation for estimating maximum transverse compression stress.

\[
f_{tc} = \frac{f_{mc} (1 - (4V_f / \pi)^{0.5} (1 - E_m / E_f))}{(1 - V_f (1 - E_m / E_f))}
\]

where

- \( G_m \): Shear modulus of epoxy
- \( V_f \): Fiber volume ratio
- \( f_{mt} \): Tensile strength of epoxy
- \( f_{mc} \): Compressive strength of epoxy
- \( \nu_m \): Epoxy Poisson’s ratio
- \( E_m \): Elastic modulus of epoxy
- \( E_f \): Elastic modulus of fiber

It should be noted that no matter how these compressive material properties are, compressive material properties are not critical as much as tensile properties are since the all material failure of the unidirectional FRP sheet for this study were due to the tensile stresses or shear stresses.
2.6.4 Determination of the fracture energy of the unidirectional FRP sheet

The fracture energies of FRP laminates will be found in this section. As previously discussed, two parameters such as fracture energies for the FRP sheet in a longitudinal direction and a transverse direction are needed even though those are not as critical factors as much as other material properties such as tensile strength of the FRP.

For fracture energy of the tensile fiber, Kirk et al. (1978) measured the fracture energy of a carbon and glass fibers. In this study, Young’s modulus for carbon and glass fiber were 240 GPa and 70 GPa, respectively. The maximum tensile strength for both were 2.4 GPa and 1.65 GPa respectively. In their study, the ratio of carbon fiber to glass fiber was test parameters to observe the tensile fracture energy of the composite. For the pure carbon fiber, $100 \pm 19 \text{ kJm}^{-2}$ was obtained in Mode 1 (tension). For the pure glass fiber $316 \pm 45 \text{ kJm}^{-2}$ was obtained.

Recently, the fracture energy for carbon fiber was also measured by Pinho et al. (2006). In their study, T300/913 unidirectional carbon fiber laminate was used. A longitudinal modulus and transverse modulus for this material were 131 GPa and 8.8 GPa, respectively. In this study, the compressive fracture energy of laminate was also measured. The tensile fracture energy of laminate was 91.6 kJm$^{-2}$, and the compressive fracture energy of laminate was 79.9 kJm$^{-2}$.

For the transverse fracture energy of the laminate in Mode 1 direction was also measured by many research groups. Lucas (1992) measured the delamination fracture energy of the 16 ply unidirectional laminate [0$_{16}$] consisting of T300 fibers in a 934 epoxy resin. The fiber tensile modulus was 135 GPa. In this study, the test parameter
was the misorientation angle of fiber. From a 0 degree angle (no misorientation), the average value of 205 $Jm^{-2}$ was obtained.

Kusaka et al. (1998) also measured the interlamina fracture energy of the unidirectional carbon fiber composite (T300/2500). The longitudinal modulus and transverse modulus for this laminate were 117 $GPa$ and 8.1 $GPa$. The tensile strengths for both direction were 1.64 $GPa$ and 0.6 $GPa$. Their test parameters were the loading rate. An interesting finding in this study is that the increased loading rate decreased the fracture energy of the interlamina debonding. From the stabilized (quasi-static rate defined in their study) loading rate, the average value of 203 $Jm^{-2}$ was obtained.

From selected studies, it can be concluded that the obtained fracture energies from the various studies show similar values to each other. For this study, the most recently obtained values: 91.6 $kJm^{-2}$ and 203 $Jm^{-2}$ were used for fracture energies of FRP laminate in the fiber direction and in the transverse direction, respectively. It should be noted that the chosen values could be different for the selected material (SCH41s) for this study. However, these values negligibly affect the numerical results. Based on these values, the linear softening behavior after the onset of damage was assumed as a built-in function in Abaqus (e.g. Hibbitt et al. 2007).

2.7 Principal findings from the literature review

Principal findings from the review of the existing studies are shown in this section.
1. The use of the FRP Uwrap would increase the flexural capacity as well as shear capacity. With the addition of the Uwrap, flexural capacity and ductile behavior of the RC beam were found to increase up to 24% and 92% (Sharif et al. 1994). It was concluded that placing the U-wraps in high moment regions (constant moment region) within the shear span is more effective than near the span’s end.

2. There is no current design guideline for the FRP anchor in European FIB bulletin 14 (2001) and the ACI 440 design 440-2R report (2008).

3. It was found that the shear capacity was increased when the three sides laminated with CFRP (Uwrap) was used rather than two sides laminated with CFRP. Furthermore, Uwrap placed at an angle of 45 degrees showed a higher shear capacity than Uwraps placed at 90 degree for shear strengthening. It was also found that resistance strength to peeling (debonding) failure could be increased if the development length of the U-wrap is sufficient.

4. It has been shown in recent studies that the interface behavior between the concrete-FRP is more related to a mixed mode behavior rather than to a single fixed Mode behavior.

5. It was found that the softening behavior in the bond-slip curve of the concrete epoxy interface can be simplified from the actual exponential curve to a simple linear line if the tensile strength and total fracture energy magnitudes are kept the same.

6. Baky et al. (2007) was modeled the RC beam with the FRP Uwrap in two
dimensions (2D) by adding the contact area of the side bonded sheets to the interface elements between the concrete and the FRP sheet at the locations of the Uwrap.

7. The Digital Image Correlation is becoming a prominent technique since it is a non-contact technique, and digital data processing speed is also increased with recent software. To perform the DIC technique, randomly speckled surface preparation is one of important features for the DIC technique.

8. Many sensors for measuring the slip between the concrete and FRP such as strain gage, LVDT, DIC and interferometer have been used. Among those, a single shear lab test with the LVDT data is a useful technique to obtain the Mode 2 bond-slip behavior due to its simplicity.
Chapter 3

Experimental program

In this chapter, conducted experimental works will be presented. An overview of the experimental program, experimental procedures, and results will be shown in detail and discussed.

As a first step, a preliminary study was conducted to select important parameters and material properties needed for the modeling of the FRP strengthened beam with the Uwrap. Based on selected parameters from the preliminary study, a parametric pull-out test on the Uwrap was conducted. Material characterizations for the concrete, concrete epoxy interface (CEI), epoxy, unidirectional FRP sheet were also conducted and is presented. Using the results from the pull-out tests and the numerical parametric study, a large scale T-beam was fabricated and strengthened with Uwrap. The capacity enhancement and the additional ductility of the tested beam due to the addition of the Uwrap was observed and is discussed.

3.1 Outline of the experimental program

An overview of the experimental program is shown in Figure 3-1. It shows the outline of the experimental program. Preliminary tests were conducted for two large-scale T-beam and a small-scale beam. The major failure modes due to the addition of the Uwrap and important parameters to control the overall behavior of the beam were
observed from the preliminary test. After performing a preliminary study, material properties to be characterized were identified, such as unidirectional properties of the FRP sheet (FRP Uwrap), the concrete, and bond-behaviors between the concrete and FRP.

EXPERIMENTAL PROGRAM

Chapter 3

(1) Preliminary study
- Large scale T-beam
- Small scale beam

(2) Basic material properties
- Concrete
- FRP sheet (Uwrap)
- Epoxy

(3) Mode 1 and Mode 2 tests
- Test methods
- Plain concrete
- Concrete-epoxy-interface (CEI)
- CEI under the Uwrap

(4) Mode 1 and Mode 2 tests with Uwrap
- Test methods
- CEI under the Uwrap

(5) Pull out tests
- Load-displacement
- Strain profile
- Digital image correlation (DIC)
- Thermocamera

(6) T-beam test
- Load-displacement
- Strain profile

Figure 3-1: Outline of the experimental program

Material characterizations for the bond behavior between the concrete-epoxy interface (CEI) were determined since it is the governing parameter for the debonding of the longitudinal FRP sheet and the Uwrap. For both bond behavior of plain concrete and CEI, Mode 1 fracture properties were obtained from the three-point bending test. For the bond behavior of the CEI, Mode 2 fracture energy was also obtained.

As a further study of the concrete-epoxy interface, modified CEI fracture properties under the Uwrap were also studied. These tests were planned and performed based on the hypothesis that the fracture energy of CEI under the Uwrap would be
different than the zone without the Uwrap. Specific details for the test set up and the specimen preparation will be shown in a Section 3.3.5.

The orthotropic properties of the unidirectional FRP were also considered as important material properties in this study. They are not important material properties for the longitudinal FRP sheet since the FRP sheet is primarily located in a longitudinal direction in the beam system. Therefore, in this case, the FRP sheet could be modeled with isotropic material properties. By contrast, since the Uwrap is deformed in the transverse direction and is attached perpendicular to the axis of the beam length, these transverse tension properties and the shear properties are important parameter of this study. Accordingly, they were obtained experimentally.

After completing tests for the material characterization, a parametric study on the pull-out test was planned and conducted. These tests were developed in order to observe the effect of the different types of Uwraps on the arresting mechanism of the crack propagation and the increase in the debonding capacity.

Based on the observation from a parametric study on the pull-out test and results of the numerical analysis, a large scale RC T-beam was strengthened with FRP sheets and transverse FRP Uwrap. The strengthened beam was then tested under a four point flexural test set up. Obtained results were compared with results of the numerical analysis.
3.2 Preliminary tests

In this section, preliminary tests are presented and discussed. The purpose of conducting a preliminary study was to identify the general behavior of the Uwrap, and to plan a parametric study and material characterizations. First, two large scale T-beams were fabricated and strengthened with the FRP sheet and the Uwrap prior to this study; however testing of those was conducted as a part of this study. Second, a small scale rectangular beam was fabricated and strengthened by the FRP sheets and the Uwrap. From these two types of tests, two different failure modes were obtained due to the addition of the Uwrap.

3.2.1 Large scale T-beam tests

To see the effect of the Uwrap, two large scale T-beams were fabricated as a part of a prior study. In this chapter, firstly, the design and fabrication of the T-beam, conducted by a former researcher will be shown. Secondly, as a part of this study, the displacement at the mid-span, and the strain data at designated locations will be analyzed and compared with the numerical analysis in Chapter 4.

3.2.1.1 The design of the large scale T-beam

To design a typical deteriorated type of bridge girder which needs to be strengthened, some data was collected. A considerable part (25%) of the United States
bridge infrastructure is in need of repair or replacement because structural deterioration has resulted in loss of load capacity (e.g. TRIP National Report 2009).

According to TRIP, the deterioration of the bridge has been rapidly increasing, since a percent of an increasing vehicle traveled across the country by 41%, while new road mileage increased by only four percent from 1990 to 2007.

From some bridge deterioration studies, it can be concluded that most of American bridges in need of strengthening and repair have been in service between 55 and 60 years (e.g. Ramey and Wright 1997, Higgins et al. 2004). Considering these statistics, this study has narrowed the search for typical bridge geometry to bridges constructed between 1950 and 1960.

According to collected information (e.g. TDOT bridge manual 2001), a typical bridge geometry constructed between 1950 and 1960 is summarized in Table 3-1

<table>
<thead>
<tr>
<th>Span (m)</th>
<th>H (m)</th>
<th>bw (m)</th>
<th>Girder spacing (m)</th>
<th>Deck thickness (m)</th>
<th>$F_y$ (MPa)</th>
<th>$f'c$ (MPa)</th>
<th>Truck loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>12~18</td>
<td>Span/12</td>
<td>0.36</td>
<td>1.98</td>
<td>0.165</td>
<td>420</td>
<td>21</td>
<td>H20, H15, H10</td>
</tr>
</tbody>
</table>

Table 3-1: Criteria used for defining girder proportions

Based on a selected geometry of girders, changes in the girder design due to variations of span and truck load were investigated. In addition, the strengthening ratios required to upgrade the flexural capacity of bridges built in 1950s to meet current AASHTO specifications (2002) were investigated.

Calculated results are shown below. As span length increases, rebar area is
also increased. Likewise, nominal moment was shown in Figure 3-2 (b). As span length increased, designed nominal moment is increased. This calculated nominal moment capacity of the girder based on the allowable stress design is compared with the current design load based on load and resistance factor design. To generate the current design load, HS 20 truck load was selected. Likewise, to design the capacity of the girder, three truck loads were considered: H10, H15 and H20. As Figure 3-2 shows, most nominal moments of girders ($M_n$) are less than the required moments, indicating that designed girders using allowable stress design are not sufficient to carry current design load HS20. Therefore, these types of girders need to be strengthened. The required strengthening ratio to bring the capacity at least equal to $M_u$ (Factored moment due to the truck loading) was found to be in the range between 1.03 and 1.50.

---

Figure 3-2: Calculated rebar area (a) and nominal moment (b) depending on varied span length

Among the required strengthening ratio, a girder design based on the H15 truck load was selected as a moderate truck load among H10, H15, and H20. A designed...
girder based on the H15 truck required 20% to 30% strengthening effect to meet the recent design guideline base on HS20. Accordingly, an average value of 25% strengthening ratio was selected. Based on an analysis of moment capacity, the strengthening effects from 27% to 20% could be obtained if one ply of carbon FRP sheet (Typo® SCH-41s) or one ply of glass FRP sheet (Typo® SEH-51a) is used, respectively. For this moment analysis, it was assumed that FRP would fail in tension.

The geometry shown in Figure 3-3 (a) was obtained as a typical girder section design. However, the designed girder was scaled down using a selected scale factor due to the space and logistic limitations in the CITEL structural laboratory. The scale factor of $\frac{5}{8}$ was chosen (e.g. Harris and Sabnis 1999) because, the flange to height ratio for the girder will make its fabrication and flexural test impractical in the laboratory since the web width is much thinner than the flange width. Finally, a final section with equivalent flexural and shear capacity was selected as shown in Figure 3-3 (b).

---

![Figure 3-3](image_url)

*Figure 3-3: Specimen geometry. (a) Typical girder section; (b) Scaled girder section for laboratory tests*
3.2.1.2 Strengthening large scale T beam

After strengthening of the glass fiber Uwrap, the shear strength of the beam was 254 kN, which is more than the predicted maximum failure load. Layouts of the FRP Uwraps for two beams are shown in Figure 3-4. Strain gages were applied along the length of the longitudinal FRP sheet as well as the Uwrap. Strain gage locations on the Uwrap are shown on the sketches.

Figure 3-4: Longitudinal geometry of the beam, locations of the Uwraps and the strain gages
3.2.1.3 Results from T-beam flexural tests

As a part of this study, the fabricated beams were tested in flexure. Results from fabricated two beams are shown in this section. Load-displacement and strain profiles were investigated. Failure modes of each beam were also indentified. The overall behavior of two beams follows the typical failure mode of the reinforced concrete beam. First, steel rebar yielded and the FRP sheet was ruptured.

![Graph](image.png)

**Figure 3-5**: Load-displacement graph from large scale T-beam tests

From the CFRP beam, the yielding of steel was reached at an average load of 139.8 kN (31.4 kips) as shown in Figure 3-5. Fiber rupture took place in two steps. Fiber rupture initiated catastrophically at the center at an average load of 143.5 kN (32.3 kips) followed by a second major catastrophic failure of fiber at 196 kN (44.1 kips).

From the GFRP beam, the failure started in the center with concrete cover failure and a gradual FRP rupture at an average load of 170.6 kN (38.4 kips) after the yielding of the reinforcing steel rebar in tension at 156 kN (35 kips). As the load increased, concrete
cover failure was observed to extend towards the supports. To sum up, both beams failed by FRP ruptures at the center line of the beam. Failure images of both beams from the laboratory are shown in Figure 3-6. Data from strain gages are not shown in this chapter. Specific strain data would be explained with FEM results in Chapter 4.

(a) CFRP beam failure (FRP rupture at centerline)

(b) GFRP beam failure (FRP rupture at centerline)

Figure 3-6: Failure image from CFRP beam (a) and GFRP beam (b)
3.2.2 Test of a small scale RC beam

A small scale RC beam was designed and fabricated based on the ACI 318-05 to fail by the yielding of the tensile reinforcement followed by the crushing of the concrete in both the unstrengthened and strengthened cases.

---

Figure 3-7: The designed beam cross section (a) and the placement of the Uwrap and strain gages (b)
Figure 3-7 shows the beams cross section (depth of 20 cm, a width of 15 cm, a length of 152 cm) and a strengthening scheme. The depth of the concrete cover (to the center of the flexural steel reinforcement) is 38 mm. The reinforcing ratio and stirrup size were chosen based on these dimensions (see Figure 3-7). The stirrup spacing was 90 mm. For the FRP strengthening, two layers of the FRP sheet were used to strengthen the flexural capacity of the beam. Two layers of the FRP sheet were selected to initiate the debonding failure rather than the rupture of the FRP sheet. At both ends, one layer of the Uwrap was attached. The behavior of the Uwrap was observed during the test.

3.2.2.1 Results from test of a small scale beam

The small scale RC beam was tested under the four point bending load. Its load-displacement curve is shown in Figure 3-8. The strain gage data was used for comparison with the result of the numerical analysis. The Uwrap was not debonded even after the complete debonding of the FRP sheet. As Figure 3-8 shows, the first observed failure was the debonding between the concrete and FRP sheet. After this ultimate failure mode, several partial debondings of the Uwrap occurred before both the FRP sheet and Uwraps were completely debonded. Due to the anchor effect of the Uwrap, sudden failure was minimized.
Two interesting observations resulted from this test. The Uwrap allowed a slip immediately after the debonding of the concrete-FRP. This sudden slip caused a sudden drop of the applied load as shown in Figure 3-8. Figure 3-9 explains this sequence. This slip also caused a severe deformation of the Uwrap after the FRP sheet was debonded. Before the debonding of the FRP sheet, the slip was not initiated. The localized damage was also observed from the bottom part of the Uwrap as shown in Figure 3-9. An interesting observation is that the Uwrap can still carry the tensile stress even after the severe deformation because most damage part of the Uwrap by a shearing is the matrix rather than the fiber. This phenomenon explains that the stress transferred from the longitudinal FRP sheet was still carried by the Uwrap even after the debonding of the FRP sheet, generating a residual strength.

It was also found that the bottom area of the Uwrap could not hold the FRP sheet effectively if the concrete width is wider than that of the FRP sheet. Therefore, the same
size of the FRP sheet with the beam width is recommended in order to minimize the slip since the Uwrap holds the FRP sheet more effectively. The angle of the fiber in the Uwrap could be also the critical factor to control the slip behavior. Depending on the angle of the fiber, shear deformation of the Uwrap and matrix failure between fiber bundles could be reduced or increased. These observed key parameters will be studied in the next section.

Figure 3-9: Debonded surface of the Uwrap and observed slip between the concrete and FRP
Secondly, it was observed that the lower area of the Uwrap at the side of the concrete beam was not well bonded as shown in Figure 3-10.

![Figure 3-10: Observed flaw region of the Uwrap due to the sharp edge of the concrete](image)

Figure 3-10: Observed flaw region of the Uwrap due to the sharp edge of the concrete

Approximately 20% of the bonded area between the Uwrap and the side of concrete was not well bonded. It was found that the corner area of the Uwrap can cause a small gap between the Uwrap and concrete due to the small viscosity of the used epoxy during the curing time, resulting in a flaw area at the lower region of the Uwrap. To eliminate this type of the flaw between the FRP and concrete before the epoxy is cured, an epoxy which has high viscosity should be used to secure the attachment between the Uwrap and the concrete corner before the epoxy is completely cured. From the manufacturer of the epoxy (Fyfe Company), the epoxy, Tack-coat is recommended to minimize this kind of problem. However, it was found that it takes a lot of effort to mix the components of the Tack-coat due to the high viscosity.

Since the Tack-coat epoxy did not provide appropriate initial bond capacity before the epoxy is cured, the Fyfe Company recommend using Cabosil, which is a
primarily a silica fume. Mixing the Cabosil with Tack-coat epoxy would increase the viscosity of the epoxy.

3.2.3 Principal findings from the preliminary study

Based on the conducted preliminary tests, the following observations and test plan were made.

1. Basic failure mode from both large scale T-beams was the rupture of the FRP sheet. Therefore, material properties such as a tensile strength and a tensile modulus in fiber direction need to be obtained experimentally.

2. The debonding of the FRP sheet and the Uwrap were obtained as a failure mode for the small scale rectangle beam. Therefore, material characterizations for the concrete-epoxy interface where the FRP and the Uwraps are located should be investigated.

3. Even after the complete debonding between the FRP and the soffit of the concrete, the Uwrap can hold the FRP sheet, and transfer a certain load from the FRP sheet to the bond between the Uwrap and the side of the beam.

4. As the Uwrap transfers the load from the FRP sheet to the side of the beam, a progressive debonding on the side of Uwrap occurs. Matrix failure of the Uwrap was also observed; however, the carbon fibers were not ruptured and were able to still carry the load.
5. From the observations, 3 and 4, it can be concluded that the use of Uwrap minimizes the brittle failure of the debonding between FRP sheet and concrete, enhancing the ductility in the externally bonded FRP system.

6. The connection between the side of the Uwrap and bottom of the Uwrap is an important factor to control the slip effect. If the beam width is larger than the width of the FRP sheet, the Uwrap will allow a certain amount of slip.

7. According to observation 6, a parametric study is recommended to see the effect of the ratio of the width of the FRP sheet to the beam width. Specific details about the parametric study are shown in Chapter 4.

8. From the observation of the small scale beam, the angle of the Uwrap (90 degree) which is perpendicular to the longitudinal FRP sheet could not be an effective angle for generating the anchor effect of the Uwrap. Accordingly, it could be conjectured that the angle of the fiber in the Uwrap to the FRP sheet will control the shear deformation of the Uwrap. Therefore, the orientation of the fiber in the Uwrap was selected and studied as one of the main test parameters to be investigated.
3.3 Material characterization

For the material characterizations, the concrete, the concrete-epoxy interface (CEI), and the FRP sheet as well as FRP Uwrap were selected and characterized in order to predict the behavior of the externally bonded FRP system.

<table>
<thead>
<tr>
<th>Material</th>
<th>Constitutive model</th>
<th>Required material properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>Damage plasticity (Continuum element)</td>
<td>Modulus of elasticity, $E$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Tensile strength, $f_t$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Compression strength, $f_c$</td>
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<tr>
<td></td>
<td></td>
<td>Fracture energy, $G_F$</td>
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<tr>
<td></td>
<td></td>
<td>Poisson’s ratio, $\nu$</td>
</tr>
<tr>
<td>Concrete-epoxy interface</td>
<td>Damage plasticity (Continuum element) Damage mechanics (Cohesive element)</td>
<td>Modulus of elasticity, $E$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Tensile strength, $f_t$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Fracture energy: $G_F$ (Mode 1), $G_{F2}$ (Mode 2),</td>
</tr>
<tr>
<td>FRP</td>
<td>Hashin-failure criteria (Shell element) Damage mechanics (Shell element)</td>
<td>Modulus of elasticity in fiber direction, $E_f$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Modulus of elasticity in transverse direction, $E_t$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Shear Modulus of elasticity, $G$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Tensile strength, $f_t$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Transverse tensile strength, $f_{t}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Compression strength, $f_c$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Transverse Compression strength, $f_{ct}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Fracture energy in fiber direction, $G_{F1}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Fracture energy in transverse direction, $G_{F2}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Poisson’s ratio, $\nu$</td>
</tr>
</tbody>
</table>

Based on these materials, the required materials properties for a numerical study could be summarized as shown in Table 3-2. Constitutive models and corresponding
required material properties could be also found in the theoretical background in Chapter 4. These required properties are selected based on the selected constitutive model.

### 3.3.1 Compressive strength of the concrete

To obtain the concrete compressive strength, a total of six compression tests were performed on the concrete cylinders according to ASTM C39. Both 15 cm by 30 cm (6 inch by 12 inch) and 10 cm by 20 cm (4 inch by 8 inch) were tested, and average values were used for the numerical analysis. The concrete was prepared from a local concrete company in State College, the Centre Concrete Company. The ordered concrete had a 6% air entrainment. Maximum aggregate was 1B stone which has a 12 mm diameter. Measured slump was 4 inch. Target strength was designed to be 27.6 Mpa (4000 psi) in order to represent a deteriorated aging structure condition. Tests were performed at 52 days after the date of concrete pouring. The pull-out test was conducting during two weeks at 52 days after the date of concrete pouring. Obtained results are shown in Table 3-3.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Size(inch)</th>
<th>Obtained load (kN)</th>
<th>Maximum stress ( f'_c ) (Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>COMP-1</td>
<td>6x12</td>
<td>710.17</td>
<td>38.93</td>
</tr>
<tr>
<td>COMP-2</td>
<td>6x12</td>
<td>729.25</td>
<td>39.98</td>
</tr>
<tr>
<td>COMP-3</td>
<td>6x12</td>
<td>656.93</td>
<td>36.02</td>
</tr>
<tr>
<td>COMP-4</td>
<td>4x8</td>
<td>339.96</td>
<td>41.93</td>
</tr>
<tr>
<td>COMP-5</td>
<td>4x8</td>
<td>325.02</td>
<td>40.09</td>
</tr>
<tr>
<td>COMP-6</td>
<td>4x8</td>
<td>327.68</td>
<td>40.42</td>
</tr>
<tr>
<td>avg</td>
<td></td>
<td></td>
<td>39.56</td>
</tr>
</tbody>
</table>
The averaged value of the concrete compressive strength is 40 MPa.

### 3.3.2 Splitting tensile tests on the concrete and concrete epoxy interface (CEI)

To obtain the maximum tensile strengths of concrete and the concrete epoxy interface, the splitting tensile test was performed on the concrete cylinders according to ASTM C496. Figure 3-11 shows a sketch of specimens for the splitting tensile test. The test was performed on the two different types of the concrete cylinder. One is plain concrete while another is concrete-epoxy interface. Specimens of the concrete-epoxy interface fabricated as the cylinder was cut into a half moon shape by the brick saw and were bonded again to each other by the epoxy as shown in Figure 3-11.

![Diagram](image.png)

**Figure 3-11:** Tested specimen type and geometry (ASTM C496)

As previously explained in Chapter 2, for this type of test, the width of the bearing strip (b) controls the test results. Accordingly, the 5% of the relative width of the bearing strips was used to obtain the splitting tensile strength of the concrete for the both dimensions. Table 3-4 shows the obtained results from the plain concrete (SPL) and the
concrete-epoxy interface (SPL-IE). Both types of specimens show the closed value of the
tensile strength to each other. The plain concrete strength is 5% higher than the concrete-
epoxy interface. The standard deviation for the tensile strength of the epoxy interface is
somewhat high. It should be noted that especially the tensile strength of the epoxy
interface could be determined by many factors including the surface preparation, surface
level and fabrication defect (Coronado and Lopez 2008).

Table 3-4: Obtained results from the splitting tensile tests

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Size(inch)</th>
<th>Obtained load (kN)</th>
<th>Maximum stress (MPa)</th>
<th>Standard deviation</th>
<th>CV</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPL-1</td>
<td>6x12</td>
<td>272.75</td>
<td>3.74</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SPL-2</td>
<td>6x12</td>
<td>239.88</td>
<td>3.29</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SPL-3</td>
<td>6x12</td>
<td>282.14</td>
<td>3.87</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SPL-4</td>
<td>4x8</td>
<td>125.39</td>
<td>3.87</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SPL-5</td>
<td>4x8</td>
<td>138.51</td>
<td>4.27</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>avg</strong></td>
<td></td>
<td><strong>3.81</strong></td>
<td><strong>0.35</strong></td>
<td><strong>9.22%</strong></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Size(inch)</th>
<th>Obtained load (kN)</th>
<th>Max stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPL-IE-1</td>
<td>6x12</td>
<td>211.24</td>
<td>2.9</td>
</tr>
<tr>
<td>SPL-IE-2</td>
<td>6x12</td>
<td>326.17</td>
<td>4.47</td>
</tr>
<tr>
<td>SPL-IE-3</td>
<td>6x12</td>
<td>290.99</td>
<td>3.99</td>
</tr>
<tr>
<td>SPL-IE-4</td>
<td>4x8</td>
<td>129.88</td>
<td>4.01</td>
</tr>
<tr>
<td>SPL-IE-5</td>
<td>4x8</td>
<td>95.54</td>
<td>2.95</td>
</tr>
<tr>
<td>SPL-IE-6</td>
<td>4x8</td>
<td>108.09</td>
<td>3.33</td>
</tr>
<tr>
<td><strong>avg</strong></td>
<td></td>
<td><strong>3.61</strong></td>
<td><strong>0.64</strong></td>
</tr>
</tbody>
</table>

These obtained values are important for the numerical analysis since this defines
one of the important parameters, the onset of the interface damage. As previously
explained, these obtained values would be used in order to characterize the concrete
tensile fracture behavior, which is a form of stress-crack opening.
3.3.3 The effect of the FRP sheet in the splitting tensile test

Previously calculated splitting tensile strength could be used for the interface where the Uwrap is not located. However, the splitting tensile strength at an onset of the damage in the Mode 1 direction under the location of the Uwrap could be different (stronger) than the normal splitting tensile strength due to the additional confining effect of the Uwrap. To evaluate the effects of the Uwrap on the tensile strength, splitting tensile tests of cylinder strengthened with the FRP sheet (SCH41s) were designed. It should be noted that the concrete batch for this test group is different than that for the ordinary splitting tensile test, which was previously shown. As Figure 3-12 shows, on the cylinder of the concrete-epoxy interface (CEI), one layer of the FRP sheet was attached to both ends.

Figure 3-12: A specimen of the splitting tensile tests with the FRP sheet

The FRP sheet was adjusted to be perpendicular to the expected lines of tensile cracks of the cylinder when it was bonded to the surfaces of the cylinder. The same test set up and procedures based on ASTM C496 were planned and performed. The results
were compared with the results from the plain concrete and that from the concrete-epoxy interface (CEI).

Table 3-5: Obtained results from the splitting tensile test

<table>
<thead>
<tr>
<th>Type</th>
<th>Specimens</th>
<th>Maximum load (N)</th>
<th>Maximum stress (MPa)</th>
<th>Standard deviation (MPa)</th>
<th>CV(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain Concrete (PC)</td>
<td>4x8_P_1</td>
<td>80913</td>
<td>2.50</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4x8_P_2</td>
<td>63298</td>
<td>1.95</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4x8_P_3</td>
<td>92211</td>
<td>2.84</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6x12_P_1</td>
<td>158623</td>
<td>2.17</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6x12_P_2</td>
<td>146657</td>
<td>2.01</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6x12_P_3</td>
<td>144344</td>
<td>1.98</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>avg</td>
<td>114341</td>
<td>2.24</td>
<td>0.36</td>
<td>15.93%</td>
</tr>
<tr>
<td>Concrete-epoxy interface (CEI)</td>
<td>4x4_CEI_1</td>
<td>37187</td>
<td>2.29</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4x4_CEI_2</td>
<td>43993</td>
<td>2.71</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4x8_CEI_1</td>
<td>54802</td>
<td>1.69</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4x8_CEI_2</td>
<td>84516</td>
<td>2.61</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6x6_CEI_1</td>
<td>114941</td>
<td>3.15</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6x6_CEI_2</td>
<td>85183</td>
<td>2.33</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6x6_CEI_3</td>
<td>71305</td>
<td>1.95</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6x6_CEI_4</td>
<td>97682</td>
<td>2.68</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6x12_CEI_1</td>
<td>227392</td>
<td>3.12</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6x12_CEI_2</td>
<td>171567</td>
<td>2.35</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6x12_CEI_3</td>
<td>191095</td>
<td>2.62</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>avg</td>
<td>107242</td>
<td>2.5</td>
<td>0.44</td>
<td>17.67%</td>
</tr>
<tr>
<td>Concrete-epoxy interface + FRP sheet (CEIF)</td>
<td>4x4_CEIF_1</td>
<td>71394</td>
<td>4.40</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4x4_CEIF_2</td>
<td>75041</td>
<td>4.63</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4x4_CEIF_3</td>
<td>62275</td>
<td>3.84</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4x4_CEIF_4</td>
<td>80379</td>
<td>4.96</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6x6_CEIF_1</td>
<td>125884</td>
<td>3.45</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6x6_CEIF_2</td>
<td>104844</td>
<td>2.87</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6x6_CEIF_3</td>
<td>100262</td>
<td>2.75</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6x6_CEIF_4</td>
<td>101775</td>
<td>2.79</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>avg</td>
<td>90232</td>
<td>3.71</td>
<td>0.33</td>
<td>8.82%</td>
</tr>
</tbody>
</table>

Dimensions of the specimen and test results are tabulated in Table 3-5. The dimensions of the cylinders are indicated by the name of the specimens. For example, specimens were called “Diameter (inch)-Length (inch)_Type_Number”. For the use of
practical unit, size was shown as a unit of inch. Averaged values for the three different types of the test results are compared to each other in Figure 3-13.

Figure 3-13 shows the comparisons of the results as an average value. It should be noted that the increment of the strength due to the addition of the FRP sheet is dependent on the ratio of the FRP to the concrete. The ratio of the FRP to the concrete is different depending on the geometries of the concrete specimen and attached FRP area. The averaged value of the ratio of the FRP to the concrete is around 1.6%. It shows that the addition of the FRP sheet could increase the maximum splitting tensile strength.

![Figure 3-13: Averaged splitting tensile strength depending on the types](image)

From this test, it could be concluded that the FRP sheet could enhance the maximum tensile strength up to 67%. This indicates that the maximum tensile strength of the interface in Mode 1 direction under the Uwrap locations could be also increased due to the effect of the Uwrap. This conclusion was used in order to develop a frictional bond-slip behavior under the Uwrap locations as is presented in detail in Chapter 4.
3.3.4 Characterization of the CEI under the Mode 1 loading

In this section, the characterization of the CEI under the Mode 1 loading will be summarized with the selected literature.

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Figure 3-14: Three-point bending tests (a), Splitting tensile test (ASTM C496) (b), Softening curve (c)

Required tests and set-ups for obtaining Mode 1 fracture properties of the CEI in Mode 1 loading has been studied (e.g. Qiao and Xu 2004, Coronado and Lopez 2005, 2006, Qiao and Chen 2008). For this research, it should be noted that the debonding of the concrete-FRP interface mainly occurred in the concrete, indicating that this concrete
tensile properties and fracture energy is related to Mode 1 fracture energy of concrete or concrete-epoxy interface. In order to capture Mode 1 behavior of the concrete-epoxy interface (CEI), three-points bending test on the notched beam were proposed by Qiao and Xu (2004) and developed by Coronado and Lopez (2007, 2008) as shown in Figure 2-8. The specific dimensions of the specimen and loading apparatus for the three-point bending test could be also found in a draft ASTM standard written by Gerstle and Mobasher (2007).

For the sake of clarity, a characterization of interface in Mode 1 is composed of two different test set ups. One is the three-point bending test and the other is the splitting tensile tests. Obtained results from these two tests determine the softening curve of the CEI. From the splitting tensile tests, tensile strength ($f'_t$) could be obtained. Three-point tensile tests determine other fracture properties such as brittleness length ($l_l$), the size-effect fracture-energy ($G_f$), and the cohesive (total) fracture energy ($G_F$). Mostly, linear softening curves or bilinear softening curves are used to simplify the exponential softening behavior of CEI. Detailed procedures to obtained softening curves from those two tests are well described in Gerstle and Mobasher (2007) and Coronado and Lopez (2008). The area underneath the softening curve is the fracture energy ($G_F$) of the concrete-epoxy interface. This test method was used to obtained CEI fracture properties as well as concrete fracture properties.
3.3.4.1 Obtained Mode 1 fracture energy

As previously mentioned, in order to obtain the fracture energy of the concrete and concrete-epoxy interface \((G_{F1})\) in Mode 1 direction, the three-point bending test is recommended. The three-point bending test is widely used due to its stability even though it is not a direct tensile test. A total of 12 specimens were tested and analyzed. The dimension of the cross section for the specimen was a 152 mm by 152 mm with a length of 560 mm. Draft ASTM test standard (e.g. Gerstle and Mobasher, 2007) recommends that notch depth shall be closed to 33.3% of the beam depth. Thus, by using a brick saw machine, a 50 mm notch was made at the mid-span on the bottom side of the beams. Specific sketches and specimens are shown in Figure 3-15. Important features of this test set up could be found from the self weight compensation frame and the reference frame. The self-weight compensation frame generates larger weight than the self weight of the notched beam. When a crack starts to propagate from the notch to the top of the specimens at mid-span, it holds the specimen in a stable manner (e.g. Gerstle and Mobasher 2007). Furthermore, it removes generated error due to the self weight of the notched beam. The reference frame is also an essential device for this test since it helps to avoid measuring the additional deformation generated from the deformed support. Therefore, the reference frame helps to exactly measure the pure displacement of the beam at mid-span. For the support and the loading head design, the hardened steel shafts were used, and one of the supports was designed to be free from the torsional behavior of the beam. Both supports were also designed to minimize the friction at the supports (e.g. Gerstle and Mobasher 2007).
Two different types of loading were used in order to obtain the plain concrete fracture energy ($G_{F1}$). It was aimed to see the effect of the loading rate on the fracture energy of the plain concrete. First, five specimens were tested using a stoke (loading head) control set up. Another five specimens were tested under the clip-on-gage control set up. The test loading rate was selected from the Draft ASTM test standard (e.g. Gerstle and Mobasher 2007). It recommends that the test should be run at a constant CMOD rate. The rate must be selected as the peak load is reached within 3 to 5 minutes from the start.

A load cell (MTS 661-20E-03) has a resolution of 20 N and 0.08% non-linearity of 100 kN full scale. A clip-on-gage (Epsilon 3541-005M-100M-ST) has a resolution of
1.5 µm and 0.064% non-linearity of 10 mm full scale. Furthermore, a 1 GPM servo valve was used for the MTS hydraulic test machine to control the slow test in a stable manner. Two high precision LVDTs with a range of ± 5 mm (0.2 in) and nonlinearity less than 0.2% of the full scale were used for measuring the displacement.

Figure 3-16 explains the designed procedure for the stroke control. For the loading procedure of the stroke control, it was programmed first, then a 5 kN was gradually applied on the notched beam using the rate of 2.5 kN/min in order to remove the geometrical misfit and gaps.

![Figure 3-16: Observed loading rate based on stroke control](image)

Secondly, a constant stroke control with the rate of 0.018 mm/min was used until the end of test. From Figure 3-16, it is shown that the stroke control is linear after 5 kN of initial load were applied, indicating that a constant rate was used for the stroke. Corresponding CMOD and LVDT data, however, do not show the constant rates as
slopes of those lines are changed. At the onset of the cracking, CMOD and LVDT rate were changed drastically, indicating that the crack opening rate could be changed even though the constant loading rate from the stroke is used. For the CMOD control test, 2kN/min of the loading rate to remove geometrical misfit and gaps were used until the load reaches 2 kN. Then a constant CMOD loading rate (0.01 mm/min) was being used until the end of test as shown in Figure 3-17. From Figure 3-17, it can be seen that the rates of CMOD are constant until the end of the test. However, the slopes for the LVDTs and stroke were changed at the onset of the cracking.

Figure 3-17: Observed loading rate based on CMOD control

It should be noted that the test would never reach the zero level of the loading. Instead, it approaches asymptotically. Therefore, at some point, the test should be stopped. Otherwise, it will never be reach the zero loads. Draft ASTM standard recommends the use of 2 mm as an ending point of the test. Accordingly, at 2 mm, the
test was stopped, and the far tail fracture energy was estimated using constant $A$, based on the recommendation from the Draft ASTM (e.g. Gerstle and Mobasher 2007). Far tail constant $A$ is a constant which will be used for estimating an additional fracture energy after 2 mm. A slope in the load-CMOD graph around 2 mm determines the constant $A$. Specific details could be found from the articles (e.g. Gerstle and Mobasher 2007, Bazant and Planas 1998). From this curve, two important material properties could be obtained. One is elastic modulus ($E$) of the concrete or CEI. The other is fracture energy ($G_{F1}$) of the concrete or CEI. Obtained load-CMOD graphs from both CMOD control and stroke control are shown in Figure 3-18. The load-CMOD graph and load-LVDT graph for each specimen can be found in Appendix A.

Figure 3-18: Load-CMOD graphs of the three point bending tests

The obtained fracture energy and related parameters are shown in Table 3-6. The average value of plain concrete under the stroke control was 153 N/m, while plain concrete under the CMOD control was 150 N/m. Therefore, it can be concluded that the
displacement rate is not a sensitive parameter to control the fracture energy. A total of 10 specimens under the two different types of the loading rate prove that the fracture energy is not much changed (2%) depending on the loading types. Therefore, when the CMOD control is not an affordable test method due to the limitation of the test machine, the stroke control test could be used alternatively. However, it should be noted that the loading head should move slowly enough to minimize a dynamic effect on the specimens when the stroke control test is used.

Table 3-6: Obtained fracture energies and important parameters

<table>
<thead>
<tr>
<th></th>
<th>( E ) (GPa)</th>
<th>( A ) (Nmm(^2))</th>
<th>( l_1 ) (mm)</th>
<th>( W_G ) (mm)</th>
<th>( G_F ) (N/m)</th>
<th>( G_I ) (N/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain concrete</td>
<td>Stroke_1</td>
<td>21602</td>
<td>450</td>
<td>58.0</td>
<td>182.7</td>
<td>141.28</td>
</tr>
<tr>
<td>Stroke control</td>
<td>Stroke_2</td>
<td>25961</td>
<td>704</td>
<td>69.6</td>
<td>226.8</td>
<td>178.16</td>
</tr>
<tr>
<td></td>
<td>Stroke_3</td>
<td>27737</td>
<td>543</td>
<td>45.6</td>
<td>213.1</td>
<td>146.06</td>
</tr>
<tr>
<td></td>
<td>Stroke_4</td>
<td>27025</td>
<td>714</td>
<td>48.5</td>
<td>227.9</td>
<td>179.96</td>
</tr>
<tr>
<td></td>
<td>Stroke_5</td>
<td>25991</td>
<td>213</td>
<td>35.5</td>
<td>98.8</td>
<td>123.86</td>
</tr>
<tr>
<td></td>
<td>avg</td>
<td>25663</td>
<td>525</td>
<td>51.4</td>
<td>189.9</td>
<td>153.86</td>
</tr>
<tr>
<td>Plain concrete</td>
<td>cmod_2</td>
<td>30457</td>
<td>416</td>
<td>58.8</td>
<td>194.9</td>
<td>122.30</td>
</tr>
<tr>
<td>CMOD control</td>
<td>cmod_3</td>
<td>34069</td>
<td>301</td>
<td>36.3</td>
<td>153.2</td>
<td>112.98</td>
</tr>
<tr>
<td></td>
<td>cmod_4</td>
<td>24416</td>
<td>750</td>
<td>75.3</td>
<td>250.6</td>
<td>171.71</td>
</tr>
<tr>
<td></td>
<td>cmod_5</td>
<td>33043</td>
<td>712</td>
<td>44.7</td>
<td>237.1</td>
<td>172.28</td>
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<td></td>
<td>cmod_7</td>
<td>28922</td>
<td>391</td>
<td>61.6</td>
<td>128.0</td>
<td>175.35</td>
</tr>
<tr>
<td></td>
<td>avg</td>
<td>30181</td>
<td>514</td>
<td>55.3</td>
<td>192.8</td>
<td>150.92</td>
</tr>
<tr>
<td>CEI</td>
<td>cmodIE_1</td>
<td>39277</td>
<td>332</td>
<td>70.1</td>
<td>169.5</td>
<td>112.42</td>
</tr>
<tr>
<td>CMOD control</td>
<td>cmodIE_2</td>
<td>44843</td>
<td>612</td>
<td>107.6</td>
<td>244.5</td>
<td>143.70</td>
</tr>
<tr>
<td></td>
<td>avg</td>
<td>42060</td>
<td>472</td>
<td>88.9</td>
<td>207.0</td>
<td>128.06</td>
</tr>
</tbody>
</table>

To see the effects of the Uwrap on the fracture energy in Mode 1 direction, a three-points bending test of a notched beam with bonded FRP sheet (SCH41s) was designed and performed. From these tests, it was found that the use of Uwrap could significantly increase the Mode 1 fracture energy of interface. Specific information on these tests can be found from Appendix B.
3.3.4.2 Obtained stress-crack opening graphs

In this section, the stress-crack opening displacement curves (the softening curve) will be presented. The tensile behavior of the concrete or CEI could be simplified using the obtained stress-crack opening curves, which will be determined in this section. A simple approach to get a softening curve is bilinear approximation. This bilinear curve can be shaped based on 4 parameters proposed by Guinea et al. (1994), which are the tensile strength of concrete ($f_t$), true fracture energy ($G_F$), abscissa of the centroid of the softening curve, and initial tangent intercept $w_I$. The theory is based on rigid body kinematics, and derived equations can be found in Guinea et al.(1992), Guinea (1994), and Bažant and Planas (1998). Fracture energy ($G_F$), far tail constant ($A$), and elastic modulus ($E$) from the ascending branch of load-CMOD data were used to form the bilinear softening curve. An averaged bilinear curve was drawn instead of showing each bilinear curve from each specimen. Two bilinear stress-crack opening graphs from specimens under the stroke control and under the CMOD control based on Guinea’s bilinear approximation (1992, 1994) were determined and compared to each other. A detailed description for the procedure of the obtained bilinear softening curve can be found from some published papers (e.g. Guinea et al. 1992, Guinea et al. 1994, Gerstle and Mobasher 2007). Calculated bilinear graphs based on obtained fracture energy are shown in Figure 3-19. It was found from the comparison that overall shapes are almost identical and the obtained a difference of fracture energy was only 2%.
It is difficult to see the difference between the results from the CMOD control and stroke control. The data is also tabulated in Table 3-7, showing the obtained values for each point. Data obtained from the concrete-epoxy interface (CEI) shows some different results. The stress-crack opening graph for the CEI showed a smaller maximum tensile strength and a smaller fracture energy as previously seen. However, the crack opening at a complete debonding was a bit larger than that of the plain concrete.

Table 3-7: Calculated points for bilinear softening curves

<table>
<thead>
<tr>
<th></th>
<th>$f_t$ (MPa)</th>
<th>$f_k$ (μm)</th>
<th>$w_k$ (μm)</th>
<th>$w_c$ (μm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain concrete (Stroke control)</td>
<td>3.8</td>
<td>0.382</td>
<td>13.695</td>
<td>668.994</td>
</tr>
<tr>
<td>Plain concrete (CMOD control)</td>
<td>3.8</td>
<td>0.378</td>
<td>12.550</td>
<td>671.887</td>
</tr>
<tr>
<td>CEI (CMOD control)</td>
<td>3.6</td>
<td>0.272</td>
<td>14.068</td>
<td>756.529</td>
</tr>
</tbody>
</table>
3.3.5 Characterization of the Concrete Epoxy-Interface (CEI) under the Mode 2 loading

Simple technique was developed based on relation between pullout loads and slips at the loaded end in a pullout test. This technique measures a deformation at the loaded end using two LVDTs from the pull-out load and corresponding load from the load cell. Obtained load is used to estimate the strain of the FRP sheet. It uses the unique relationship between the strain of FRP sheet and the corresponding slip (e.g. Shima et al. 1987).

3.3.5.1 Obtained Mode 2 fracture energy of CEI

The Mode 2 fracture energy of the concrete-epoxy interface was also obtained. A specific theory about the Mode 2 fracture energy is explained in Chapter 2.

Based on the single lab shear test (pull-out test), the displacement of the loading end was measured using the two LVDTs. To remove the additional deformation from the specimen itself, LVDTs were installed in the reference frame. The reference frame was also attached to the side of the concrete block. LVDTs were located at the position of the loading end and measured the displacement of the loading end of the FRP sheet. The MTS designed grip was used to hold the tip of the FRP sheet. The tip of the FRP sheet was designed by 3 layers to avoid grip failure. However, the number of layer for the FRP sheet was one. The specific geometry and test setup is also shown in Figure 3-20.
From the pull-out test, shear stress could be obtained from the load cell which is connected to the grip. However, the slip between the FRP sheet and concrete could be obtained from many measuring techniques. There are a lot of techniques to measure the slip such as strain gage, LVDT, Digital Image Correlation (DIC) and electronic speckle pattern interferometry (ESPI) (e.g. Nakaba et al. 2001, Dai et al. 2005, Ferracuti et al. 2006, Cao et al. 2007).

For this analysis, the LVDT technique was used since the high accurate LVDTs were equipped in the laboratory. Two high precision LVDTs with a range of ± 5 mm (0.2 in) and nonlinearity less than 0.2% of the full scale were used for measuring the displacement. During the pull-out test, load-displacement at the loading end was recorded. The pullout forces were measured through the load cell, and displacement at the loaded end was measured from average value of two LVDTs at both sides as shown in Figure 3-20.
Based on the obtained load-displacement graph at the loaded end, shear stress-slip curves ($\tau$–$s$ curve) were obtained. The theory proposed by Dai et al. (2005) is explained in Chapter 2. A total of three specimens were tested, and obtained results are shown in Figure 3-21.

![Graph showing load-displacement curves with FRP sheet debonding highlighted.]

**Figure 3-21**: Load-displacement curves at loaded end of the pull out specimens

Based on the obtained load-displacement curves, the shear stress-slip curve is estimated. As Dai et al. (2005) recommended, exponential functions were used to fit the experimental data as shown in Eq. 2.21. The strain of the FRP sheet was calculated using the applied load and the elastic modulus of the FRP sheet.

$$\varepsilon = f(s) = A(1 - \exp(-Bs))$$  \hfill (3.1)

Where

- $\varepsilon$ : Strain of FRP sheet at any location
- A and B: experimental parameters
Corresponding slip at that location

Obtained softening curves based on Mode 2 load are shown in Figure 3-22.

Those three curves in Figure 3-22 (a) were averaged, and a simple linear softening curve (b) was made to represent Mode 2 shear stress-slip behavior of the concrete-epoxy interface.

Figure 3-22: Obtained shear stress-slip curves (a) and simplified shear stress-slip line (b)

Areas underneath the obtained curves in Figure 3-22 (a) are calculated as fracture energies in the Mode 2 direction. Averaged fracture energy under the Mode 2 loading was calculated as a 749 N/m. The averaged maximum shear stress was 4.74 MPa. The slip at damage initiation was 0.056 mm. The slip at a complete debonding was 0.32 mm. The slip at the complete debonding was calculated using the obtained fracture energy 749 N/m and an assumption of the linear softening behavior. These obtained values were incorporated to the numerical analysis, which is shown later.
3.3.5.2 Mode 2 behavior between the FRP sheet and the concrete under the Uwrap

The previously obtained fracture energy in Mode 2 could be used for the area where the Uwrap is not located. However, the fracture energy under the Uwrap locations in Mode 2 direction could be different (stronger) than that of the interface without an Uwrap due to the additional confining effects of the Uwrap. It was already seen that the maximum tensile strength and fracture energy in Mode 1 direction increases due to the addition of the FRP sheet. Likewise, increased concrete fracture energy in Mode 2 direction is also expected from the addition of the FRP sheet. Accordingly, specimens were developed and tested under the pull-out test set up.

Figure 3-23: Pull-out test specimens with Uwraps for the Mode 2 pull out tests
The purpose on this test is to see the effect of the Uwrap on the fracture energy in Mode 2 direction and its corresponding bond-slip behavior. From the obtained strain-slip behavior at the loading end, the bond-slip behavior was conjectured using Eq. 2.21.

Figure 3-23 shows the developed specimens to obtain the Mode 2 bond-slip behavior under the Uwrap. A dimension of the concrete blocks is 152 mm by 152 mm by 305 mm. Width and thickness of the FRP sheet are 32 mm and 2 mm (two layers), respectively. A total of three different types of specimen were tested. The first one is the control specimen which does not have an Uwrap (No Uwrap). Both the second (Uwrap 1) and third one (Uwrap 2) are specimens with an Uwrap. The side length of Uwrap 1 was 0 mm and the side length of Uwrap 2 was 50 mm. Figure 3-24 shows the obtained strain-slip curves of the loading end. Based on the obtained strain-slip curves, the bond stress-slip curves were conjectured. First of all, it could be observed that the maximum shear stress increases due to the addition of the Uwrap. Both the Uwrap 1 and Uwrap 2 show the higher maximum shear stress compared to the no Uwrap. Secondly, it could be found that after the onset of the debonding, stress does not drop to zero but to non zero stress, which is less than the maximum shear stress. After that, stress is gradually increased.

From this test, it can be found that there is residual stress between the concrete and FRP even after the complete debonding between those due to the addition of the Uwrap. It was also found that the Uwrap with the three sides (Uwrap 2) rather than that with one side (Uwrap 1) shows more strengthened bond-slip behavior. However, it should be noted that the obtained bond-slip behavior shown in Figure 3-24 includes the effect of the Uwrap. The deformed (strained) Uwrap due to the pull-out load from the FRP sheet would, anyhow, contribute to the strengthened bond-slip behavior, indicating that the
pure bond-slip behavior could be obtained after excluding the effect of the Uwrap. Therefore, pure bond-slip curves of the interface under the location of the Uwrap could be smaller in magnitude than the obtained bond-slip behavior in this section. However it is thought that the overall shape and trend of the bond-slip curves due to the Uwrap could be similar even though the magnitude is smaller than that. Accordingly, an overall shape of the bond-slip curve obtained in this section would be also used to form pure frictional bond-slip model under the Uwrap locations, which will be explained in Chapter 4.

Figure 3-24: Different bond-slip behaviors depending on the existence of the Uwrap

In Chapter 4, developed frictional bond-slip model will be explained. The idea of the frictional bond-slip model was inspired after performing the test in this section and developed in order to obtain pure bond-slip behavior of the interface under the Uwrap. Specific information will be shown in Chapter 4.

3.3.6 Transverse tensile properties of the FRP sheet (including FRP Uwrap)

In this section, a material characterization for the unidirectional FRP sheet will be presented. Transverse tensile properties of the FRP sheet ($f_{tt}$) were obtained from the
tensile test according to the ASTM D3039. As explained previously, the tensile strength of the FRP in the transverse direction is important especially for the unidirectional Uwrap. The transverse tensile property is also related to the shear behavior of the unidirectional Uwrap. A dimension of the specimen and test set up is shown in Figure 3-25. One layer of the FRP sheet was tested instead of testing a stacked FRP sheet because one layer of the Uwrap was used for the pull-out test. The measurement of the thickness of one layer varied from 1.15mm to 1.36 mm.

![Figure 3-25: Prepared coupons for transverse tensile test (a) and test set up (b)](image)

Obtained stress strain graphs from a total of 5 specimens are shown in Figure 3-26. From these curves, transverse tensile modulus and strength were obtained. As expected, an elastic behavior up until the failure was observed. After the failure, specimens did not take any further load due to the matrix failure between fiber bundles.
A calculated modulus and a maximum stress from previous stress strain curves are shown in Table 3-8. The average value of 9 \text{ GPa} and 37.8 \text{ MPa} were obtained as a transverse modulus and a maximum stress of transverse direction.

### Table 3-8: Obtained transverse tensile strength and modulus from experiments

<table>
<thead>
<tr>
<th></th>
<th>Modulus, (E) (GPa)</th>
<th>Maximum stress, (f_{tt}) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tr_CFRP_1</td>
<td>8.52</td>
<td>36.03</td>
</tr>
<tr>
<td>Tr_CFRP_2</td>
<td>6.91</td>
<td>37.04</td>
</tr>
<tr>
<td>Tr_CFRP_3</td>
<td>9.6</td>
<td>40.09</td>
</tr>
<tr>
<td>Tr_CFRP_4</td>
<td>9.8</td>
<td>36.22</td>
</tr>
<tr>
<td>Tr_CFRP_5</td>
<td>10.47</td>
<td>39.41</td>
</tr>
<tr>
<td>\text{avg}</td>
<td>9.06</td>
<td>37.76</td>
</tr>
</tbody>
</table>

**3.3.7 Shear test on the FRP sheet**

In this section, the shear properties of the unidirectional FRP sheet are presented. For shear properties of FRP sheet, a v-notched beam test was selected and performed.
according to ASTM D5379. Firstly, stacked laminates ([90]_4) were cut into a v-notched beam shape. Fiber orientation of the notched beam was perpendicular to the loading direction. At the top and bottom of the midsection, 4mm v-notches were made to introduce the shear failure at a designated location. The size of the coupon is 7.5 cm by 1.9mm. A detailed geometry of the specimen can be found in ASTM D5379.

Test results from stacked laminates were not representing real material properties since observed failure type was not an ideal failure mode specified in ASTM D5379. Most specimens were failed by bottoming out as shown in Figure 3-27. Failure by bottoming out would occur if the compressive strength of the contact area is smaller than the expected shear strength of the notched beam. For these experiments, the obtained shear modulus and a maximum shear stress were 3.7 GPa and 65 MPa. However, the specimen was failed by bottoming out before pure shear failure. Therefore, it is expected that that actual shear strength could be larger than these values.

![Fiber direction](image)

Figure 3-27: Picture and sketch of bottoming out of a multi-layer V-Notch specimen

It was found from these tests that staked specimens ([90]_4) might have different material properties compared to one layer of specimens. Shear stress-strain behaviors of the unidirectional FRP sheet is epoxy dominated behavior as the epoxy is weaker than the
fiber. Carbon fiber is relatively much stronger than epoxy, and shear failures occur
dominantly in the epoxy region rather than in the carbon fiber itself. Therefore, the final
obtained maximum stress and modulus of the stacked layers ([90]_4) is larger than the
results obtained from the one layer of lamina. As Figure 3-28 shows, the fractured area
could be much larger than one layer when epoxy failure occurs since each layer is not
well aligned and crossed over to each other in the case of the stacked layers. Therefore,
the maximum shear stress and shear modulus obtained from multiple layers overestimate
the true material properties of the one layer lamina. For the Uwrap system, multiple
layers are not a common type, and mostly one layer of Uwrap has been used (e.g. Monti

Figure 3-28: Different fracture paths depending on number of layers

Accordingly, one layer of v-notched specimens was prepared rather than the
stacked laminate. To prevent buckling and local crushing due to the thin thickness of one
layer, tabs for each side were attached. Figure 3-29 shows the one layer specimen to be
tested under the shear load fixture.

For the single layer of the FRP sheet, a total of 10 specimens were tested. The fiber orientations for the specimens are as follows: Four were [90]₁, three were [0]₁, and three were [90/0], which is not a single layer. For example, [90] indicates the angle of 90 degrees. [90/0] indicates the cross plies of two layers.

![Image of a V-notched specimen under the shear test fixture]

**Figure 3-29**: One layer of a V-notched specimen under the shear test fixture

The obtained failure modes are compared to the ASTM ideal failure modes for different fiber orientations. For example, [90] shows a different fracture path compared to [0].

![Images of failure modes]

**Figure 3-30**: Different fracture paths due to the fiber orientation
All specimens were not broken before the shear strain reaches the 5% limit. Therefore, maximum stresses were obtained at 5% strains as ASTM D5379 recommended. From the one layer specimens, the bottoming out failure did not occur. Instead, two ideal types of failure modes recommended by ASTM D5379 were observed as shown in Figure 3-30. Final obtained stress strain curves from selected specimens are shown in Figure 3-31. As shown below, maximum shear stress was obtained at each 5% strain.

![Figure 3-31: Stress-strain curves obtained from shear tests on one layer V-notched specimens](image)

The analyzed data from the obtained results are tabulated in Table 3-9. Shear modulus and shear strength from one layer of lamina were 1.66 GPa and 35.94 MPa, respectively. As expected previously, the shear modulus and shear strength values of the single layer are smaller than that of the multiple layers of FRP sheet. The shear modulus and strength of the single layer was 45% and 55% of the multiple layers of the FRP sheet, respectively. The averaged data from the total of 10 specimens was used for the numerical analysis of the pull-out test and beam test.
Table 3-9: Obtained results from shear tests on the FRP sheet

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Shear Modulus (GPa)</th>
<th>Shear Strength (MPa)</th>
<th>Maximum shear strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>[90]_1</td>
<td>1.86</td>
<td>33.56</td>
<td>5.00%</td>
</tr>
<tr>
<td>[90]_2</td>
<td>1.54</td>
<td>36</td>
<td>5.00%</td>
</tr>
<tr>
<td>[0]_1</td>
<td>1.16</td>
<td>37</td>
<td>5.00%</td>
</tr>
<tr>
<td>[0]_2</td>
<td>1.62</td>
<td>37</td>
<td>5.00%</td>
</tr>
<tr>
<td>[0]_3</td>
<td>1.79</td>
<td>38.2</td>
<td>5.00%</td>
</tr>
<tr>
<td>[90/0]_1</td>
<td>1.86</td>
<td>35.13</td>
<td>5.00%</td>
</tr>
<tr>
<td>[90/0]_2</td>
<td>1.82</td>
<td>34.7</td>
<td>5.00%</td>
</tr>
<tr>
<td>avg</td>
<td>1.66</td>
<td>35.94</td>
<td>5.00%</td>
</tr>
</tbody>
</table>

Table 3-10 shows the summarized results of the material properties of the unidirectional FRP sheet. These obtained data will be used for the numerical analysis, which will be discussed in Chapter 4. Tensile properties in longitudinal direction was obtained from manufacturer due to the limitations of the test set up and hydraulic machine.

Table 3-10: Summary of obtained material properties of one layer V-notched specimens

<table>
<thead>
<tr>
<th>Properties</th>
<th>CFRP lamina</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of elasticity, $E$ (GPa)</td>
<td>72.4*</td>
</tr>
<tr>
<td>Transverse modulus of elasticity, $E_t$ (GPa)</td>
<td>9</td>
</tr>
<tr>
<td>Shear modulus of elasticity, $G$ (GPa)</td>
<td>1.66</td>
</tr>
<tr>
<td>Tensile strength, $f_t$ (MPa)</td>
<td>876*</td>
</tr>
<tr>
<td>Transverse tensile strength, $f_{tt}$ (MPa)</td>
<td>38</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.27</td>
</tr>
</tbody>
</table>

*indicate properties obtained from manufacturer data sheet
3.3.8 Tensile test on epoxy coupons

Tensile properties of epoxy were obtained and will be presented in this section. Since the epoxy is assumed to be an isotropic material, the tensile test in the transverse direction is not required unlike the unidirectional FRP sheet. The epoxy used for this study is Typo S, a product from the Fyfe Company. Tensile tests were performed to obtain the maximum tensile stress \( f_t \) and Poisson’s ratio \( \nu_{12} \). A total of five specimens were tested. To avoid the bending effects, two strain gages were mounted at both sides and monitored to check whether an unbalanced strain is generated or not. One strain gage is mounted in a transverse direction to obtain Poisson’s ratio of epoxy. According to ASTM D3039, less than \( \pm 3\% \) of bending strain is the recommendation for reasonable results. For most of the specimens, less than \( \pm 3\% \) bending strain was generated during the test as shown in Figure 3-32 (b), indicating that the test set up was well aligned. However, strain differences between two surfaces of the epoxy coupon were increased up to 9% as the stress-strain starts to follow the non-linear behavior. Figure 3-32 (a) shows stress-strain curves for both the longitudinal and transverse direction. From this test, Poisson’s ratio for the epoxy was also calculated.
Since stress-strain curves are showing non-linear behavior, stress values are found at 0.001 and 0.003 of strain values. These obtained experiment values were used for elastic-plastic behavior of epoxy for the numerical analysis. Obtained data analyzed results are shown in Table 3-11.

Table 3-11: Obtained results from epoxy coupons

<table>
<thead>
<tr>
<th></th>
<th>Tensile Strength (MPa)</th>
<th>E(MPa)</th>
<th>Stress at 0.001</th>
<th>Stress at 0.003</th>
<th>Poisson’s ratio</th>
<th>Tensile strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tyfo S_1</td>
<td>59.22</td>
<td>3199</td>
<td>3.3</td>
<td>9.7</td>
<td>0.39</td>
<td>&gt; 5</td>
</tr>
<tr>
<td>Tyfo S_2</td>
<td>56.81</td>
<td>3150</td>
<td>3.0</td>
<td>9.3</td>
<td>0.40</td>
<td>&gt; 5</td>
</tr>
<tr>
<td>Tyfo S_3</td>
<td>56.97</td>
<td>3050</td>
<td>3.0</td>
<td>9.1</td>
<td>0.38</td>
<td>&gt; 5</td>
</tr>
<tr>
<td>Tyfo S_4</td>
<td>54.09</td>
<td>2850</td>
<td>3.2</td>
<td>8.9</td>
<td>0.39</td>
<td>&gt; 5</td>
</tr>
<tr>
<td>Tyfo S_5</td>
<td>56.03</td>
<td>3000</td>
<td>2.9</td>
<td>8.9</td>
<td>0.38</td>
<td>&gt; 5</td>
</tr>
<tr>
<td>avg</td>
<td>56.62</td>
<td>3049</td>
<td>3.1</td>
<td>9.2</td>
<td>0.39</td>
<td>-</td>
</tr>
</tbody>
</table>

3.4 Pull-out test (Uwrap parametric study)

In this chapter, a conducted parametric study on the pull-out test will be presented and discussed. It was found that from the preliminary study that the larger width of the
Uwrap compared to the width of the beam allowed a slip, minimizing the anchor effects of the Uwrap. It was also found that the angle of Uwrap will be a critical factor since a large amount of shear deformation on the Uwraps at both ends was observed from the small scale beam test. Therefore, some parameters were selected to see the effects of a different geometry of Uwraps on the load-displacement behavior of the loading end. Slip profiles along the length of the FRP sheet depending on the types of the Uwrap were also investigated. Obtained results from the variety of Uwrap geometries were used to offer the design recommendations. Furthermore, test results were used for the verification of the developed numerical analysis which will be presented in Chapter 4.

First, concrete blocks were fabricated for a pull-out test. Second, the FRP sheet and Uwrap were attached to the concrete block using the epoxy. Specific test set up and geometries of the specimen are shown in Figure 3-33. The same test set up used for the Mode 2 fracture test was also used for this parametric study except for the addition of the Uwrap. Two LVDTs were used to measure the displacement of the loading end. A load cell is connected to the grip which is holding the top of the FRP sheet. The loading head was constantly moved with a rate of 1mm/min to generate the tensile stress on the FRP sheet. Data was collected at 10 Hz in order to capture any significant loading changes in a short time.
A major observed parameter is the load and the displacement between the concrete blocks and FRP sheets. Displacements of the loading end were measured by two LVDTs at both sides. To remove the additional displacement from the block movement itself, LVDTs are attached to the reference plate which is also attached to the concrete blocks using mechanical anchors as previously shown in Figure 3-20. Average values from both LVDTs were obtained from the tests to represent the slip of the loading end. Furthermore, the slip profile along the longitudinal FRP sheets during the test was measured using the previously introduced DIC technique. To measure a small amount of slip between the concrete and the FRP sheet using the DIC technique, a digital camera was installed in front of the specimens in the test set up. The sketch for test setup is also shown in Figure 3-34. The basic theory of the digital image correlation (DIC) was previously explained in Chapter 2. A thermo-camera was also used to attempt to capture the sequence of the debonding propagations and arresting mechanism of the debonding propagation from specimens during the test. The maximum temperature resolution of the
thermo-camera was 0.04 °K. It was found that it is difficult to distribute the concentrated heat from the halogen lamp. Alternatively, a thermo-camera was used to capture the increased temperature generated from the frictional behavior between the FRP and concrete. The debonding propagation and the stressed zone were captured from this method. Specific results and a discussion of limitations of this measuring technique are presented in Appendix F.

![Diagram of slip measurement setup](image)

**Figure 3-34:** Slip measurement setup from digital image correlation (DIC) and thermo-camera setup

For the parametric study, important parameters were selected before the fabrication of the specimens. From the previously observed results of the small scale beam, it was found that bottom area of the Uwrap allows a slip between the concrete surface and longitudinal FRP sheet after the debonding. The sharp corner shape of the beam was found to be an important factor to control the behavior of the Uwrap as a flaw in the bonded area was observed around the sharp corner. It can also be expected that a sharp corner could generate a stress concentration on the corner of the Uwrap. Therefore,
the corner shape was selected as a parameter of Uwrap behavior. Figure 3-35 shows the difference between a sharp edge and round edge of the prepared concrete blocks.

Figure 3-35: Sharp edge (left) and round edge (right) after grounding

Figure 3-36 shows each selected parameter in the sketch of the specimen. As explained previously, the corner length was also selected as a critical parameter related to the slip between the Uwrap and FRP sheet. In this study, the difference between the width of the beam and width of the longitudinal FRP sheet is named as the corner length. It is expected that if the width of the beam is larger than the width of the longitudinal FRP, the slip effect could be significant as seen in the small scale beam. Therefore, the corner length (the difference between the width of the beam and the width of the FRP sheet) was also selected as a test parameter. A specific sketch of the corner length will be presented in Figure 3-36.

The angle of the Uwrap was selected as a key parameter. For instance, the Uwrap in 45 degrees could more effectively hold the longitudinal FRP sheet since fiber direction is more associated to the longitudinal FRP sheet, while the Uwrap in a fiber direction of 90 degrees is not related to since it is perpendicular to the axis of the beam length.

The Uwrap height is also an important factor since increasing height will provide
a more bonded area, resulting in a larger bond capacity of the Uwrap if the height is less than the effective bond length. The Uwrap height was selected from an effective bond length of 105.5mm, calculated based on a strength model developed by Chen and Tang (2001). One set of tests was designed less than the effective bond length. Another set of tests was designed more than the effective bond length. The Uwrap thickness (1 layer and 2 layers) and the Uwrap width were also considered as parameters to control Uwrap behavior. It is expected that the thickened layer of the Uwrap would minimize the severe shear deformation of the Uwrap. The larger Uwrap width would generate the larger anchoring effect as expected; however, the efficiency of the anchoring effect might be decreased.

To sum up, the corner shape of concrete block, the corner length, the angle of Uwrap, the thickness of Uwrap, and the width and height of the Uwrap will be the 5 selected parameters to be explored in this parametric study. Based on these selected parameters, experimental tests were planned.

Figure 3-36: Selected parameters for the pull-out tests
Table 3.12 shows all pull-out test specimens to be tested and the selected values for each parameter. The geometry of the concrete block is already shown in Figure 3-33.

For each case, a total of 3 specimens were tested to represent one case. Obtained test results from the specimens will be compared to each other. At the end of the parametric study, a design recommendation will be proposed.

<table>
<thead>
<tr>
<th>Specimen Name</th>
<th>Specimen Shape</th>
<th>Bonded length (mm)</th>
<th>Long. FRP width (mm)</th>
<th>Corner length (mm)</th>
<th>Corner shape</th>
<th>Angle of Uwrap</th>
<th>Uwrap height (mm)</th>
<th>Uwrap thickness (mm)</th>
<th>Uwrap width (mm)</th>
<th>Number of specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>PU-C</td>
<td>Control</td>
<td>254</td>
<td>57</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>3</td>
</tr>
<tr>
<td>PU-N</td>
<td>Normal</td>
<td>254</td>
<td>57</td>
<td>5</td>
<td>round</td>
<td>90</td>
<td>127</td>
<td>1</td>
<td>64</td>
<td>3</td>
</tr>
<tr>
<td>PU-CN</td>
<td>Corner</td>
<td>254</td>
<td>57</td>
<td>64</td>
<td>round</td>
<td>90</td>
<td>127</td>
<td>1</td>
<td>64</td>
<td>3</td>
</tr>
<tr>
<td>PU-SP</td>
<td>Sharp 45 degree</td>
<td>254</td>
<td>57</td>
<td>5</td>
<td>sharp</td>
<td>90</td>
<td>127</td>
<td>1</td>
<td>64</td>
<td>3</td>
</tr>
<tr>
<td>PU-ST</td>
<td>Short 45 degree</td>
<td>254</td>
<td>57</td>
<td>5</td>
<td>round</td>
<td>90 (&lt;L_e)</td>
<td>127</td>
<td>1</td>
<td>64</td>
<td>3</td>
</tr>
<tr>
<td>PU-L</td>
<td>Long</td>
<td>254</td>
<td>57</td>
<td>5</td>
<td>round</td>
<td>90 (&gt;L_e)</td>
<td>127</td>
<td>2</td>
<td>64</td>
<td>3</td>
</tr>
<tr>
<td>PU-T</td>
<td>Thick</td>
<td>254</td>
<td>57</td>
<td>5</td>
<td>round</td>
<td>90</td>
<td>127</td>
<td>2</td>
<td>64</td>
<td>3</td>
</tr>
<tr>
<td>PU-SM</td>
<td>Small</td>
<td>254</td>
<td>57</td>
<td>5</td>
<td>round</td>
<td>90</td>
<td>127</td>
<td>1</td>
<td>32</td>
<td>3</td>
</tr>
<tr>
<td>PU-MN</td>
<td>Many</td>
<td>254</td>
<td>57</td>
<td>5</td>
<td>round</td>
<td>90</td>
<td>127</td>
<td>1</td>
<td>32 x #3</td>
<td>3</td>
</tr>
</tbody>
</table>

3.4.1 Obtained results from pull-out tests

In this section, obtained results of the pull out tests from LVDTs, DIC, thermo-camera, and strain gages will be presented. From several conducted tests, different load-displacement behaviors of the loading end of the FRP sheet were obtained with respect to the different geometries of the Uwrap that were defined in the previous section. From the measured slip of the FRP sheet and profiles along the length of the
FRP, it was found that a 45 degree Uwrap allows a less amount of slip compared to a normal 90 degree Uwrap. It was found that Uwrap prevents debonding propagation as well as increases the maximum bond capacity of the FRP sheet.

### 3.4.1.1 Obtained results from LVDTs

As previously explained, two LVDTs were attached to the reference plate to minimize additional displacement and obtain true relative displacement of the FRP sheet out of the constrained concrete block. One representative result from the LVDTs is shown in Figure 3-37.

![Figure 3-37: Different load-displacement graphs depending on the existence of the Uwrap (PU-N-3 and PU-C-1)](image)

Failure mode from the PU-N-3 specimen is that first the FRP sheet is debonded, and then the Uwrap is fractured. This indicates that the Uwrap can increase the ductile behavior as it holds the FRP sheet. Furthermore, the FRP sheet of the PU-N-3 was debonded at a much higher load than the PU-C-1 specimen (control). This also indicates
that the Uwrap was arresting the debonding propagation along the interface between the concrete and FRP sheet. The average obtained maximum load from the PU-C specimen (No Uwrap) was 19.7 kN, while that from the PU-N specimen (Uwrap) was 33.4 kN. A large amount of strength increment (70%) was obtained from the Uwrap.

These three load-displacement curves for the tested specimens were averaged and simplified based on the points which would be defined in this study for comparison purposes. The purpose of the simplification is to compare the results of each type of the specimen in a comprehensible way.

The load-displacement of the FRP sheet under the pulling load can be simplified by four points. As shown in Figure 3-38, the first point (point 1) is selected from the maximum load. The maximum load and displacement at the maximum load were selected as a first point. After the maximum load, the load is suddenly dropped when the debonding failure of the FRP sheet occurs. In this study, dropped load is defined as the lowest load after the debonding. The dropped load after the debonding and displacement at the dropped load was selected as a second point (point 2).

After the sudden drop of the maximum load, the Uwrap is now carrying most of the load from the FRP sheet since the bond between the concrete and the FRP sheet is broken. In addition to this, some friction force between the Uwrap and the debonded surface of concrete is expected. After the debonding of the FRP sheet, the load is not consistent as seen in Figure 3-37 while the Uwrap is holding the debonded FRP sheet from both sides of the concrete block. A complicated mechanism related to the friction, and local failure (debonding and rupture) of the Uwrap after the debonding shows the fluctuating load-displacement graph as shown in Figure 3-37. For simplicity and
modeling the complicated fluctuating behavior after the debonding of the FRP sheet, an average load was calculated as a third point (point 3) after a drop of the load. Accordingly, point 3 was selected from the average load after the debonding and displacement at the load drop due to debonding. The displacement at the complete failure of both the Uwrap and FRP sheet obtained from the experiment was selected as a fourth point (Point 4).

Figure 3-38: Simplified load-displacement behavior using four selected points (PU-N)

It was found that the selected four points are sensitive to the selected test parameters. All test results can be summarized by four selected point from experiments. The obtained four points from each type of specimen will be compared. Table 3-13 shows the obtained results from all the specimens.
<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximum Load (kN)</th>
<th>Displacement at Max. load (mm)</th>
<th>Displacement after drop (mm)</th>
<th>Load just after drop (kN)</th>
<th>Load Amount of Drop (kN)</th>
<th>Average Load after Drop (kN)</th>
<th>Displacement area after the drop (kN-mm)</th>
<th>Amount of Wrap Fiber (mm³)</th>
<th>Final Failure Mode</th>
<th>Strength enhancement (%)</th>
<th>Ductility ratio</th>
</tr>
</thead>
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<tr>
<td>PU-C-1</td>
<td>20.40</td>
<td>0.23</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>0</td>
<td>N.A.</td>
<td>SD</td>
<td>N.A.</td>
<td>N.A.</td>
</tr>
<tr>
<td>PU-C-2</td>
<td>17.18</td>
<td>0.21</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>0</td>
<td>N.A.</td>
<td>SD</td>
<td>N.A.</td>
<td>N.A.</td>
</tr>
<tr>
<td>PU-C-3</td>
<td>22.13</td>
<td>0.32</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>0</td>
<td>N.A.</td>
<td>SD</td>
<td>N.A.</td>
<td>N.A.</td>
</tr>
<tr>
<td>PU-N-1</td>
<td>32.00</td>
<td>0.96</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>20564</td>
<td>GF</td>
<td>161%</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>PU-N-2</td>
<td>31.40</td>
<td>1.04</td>
<td>N.A.</td>
<td>5.95</td>
<td>21.14</td>
<td>10.26</td>
<td>26.36</td>
<td>103</td>
<td>20564</td>
<td>UR</td>
<td>158%</td>
</tr>
<tr>
<td>PU-N-3</td>
<td>37.39</td>
<td>0.89</td>
<td>N.A.</td>
<td>1.65</td>
<td>7.3</td>
<td>24.87</td>
<td>12.52</td>
<td>30.91</td>
<td>175</td>
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<td>188%</td>
</tr>
<tr>
<td>PU-L-1</td>
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<td>0.87</td>
<td>N.A.</td>
<td>1.69</td>
<td>6.01</td>
<td>19.04</td>
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<td>21.3</td>
<td>92</td>
<td>30242</td>
<td>UR &amp; SR</td>
</tr>
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<td>PU-L-2</td>
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<td>0.99</td>
<td>N.A.</td>
<td>1.43</td>
<td>5.7</td>
<td>25.82</td>
<td>5.4</td>
<td>27.92</td>
<td>119</td>
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</tr>
<tr>
<td>PU-L-3</td>
<td>37.73</td>
<td>1.18</td>
<td>N.A.</td>
<td>2.27</td>
<td>9.04</td>
<td>23.32</td>
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<td>28.33</td>
<td>192</td>
<td>30242</td>
<td>UR &amp; SR</td>
</tr>
<tr>
<td>PU-ST-1</td>
<td>35.16</td>
<td>1</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>10887</td>
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<td>177%</td>
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<td>1.04</td>
<td>N.A.</td>
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<td>10.08</td>
<td>25.6</td>
<td>43</td>
<td>10887</td>
<td>UD</td>
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<td>23.75</td>
<td>33</td>
<td>10887</td>
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<tr>
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<td>1.9</td>
<td>7.77</td>
<td>18.99</td>
<td>13.77</td>
<td>24.45</td>
<td>144</td>
<td>20564</td>
<td>UD &amp; UR</td>
<td></td>
</tr>
<tr>
<td>PU-SP-2</td>
<td>28.76</td>
<td>0.99</td>
<td>1.74</td>
<td>6.52</td>
<td>16.56</td>
<td>12.2</td>
<td>24.99</td>
<td>119</td>
<td>20564</td>
<td>UD</td>
<td></td>
</tr>
<tr>
<td>PU-SP-3</td>
<td>29.90</td>
<td>1.03</td>
<td>2</td>
<td>6.65</td>
<td>13.84</td>
<td>16.06</td>
<td>23.87</td>
<td>111</td>
<td>20564</td>
<td>UD</td>
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<td>PU-SM-1</td>
<td>33.82</td>
<td>0.87</td>
<td>3.2</td>
<td>8.8</td>
<td>2.37</td>
<td>31.45</td>
<td>14.41</td>
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<td>10282</td>
<td>UD</td>
<td></td>
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<td>PU-SM-2</td>
<td>32.30</td>
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<td>3.15</td>
<td>6.83</td>
<td>5.12</td>
<td>27.18</td>
<td>15.28</td>
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<td>10282</td>
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<td>PU-SM-3</td>
<td>33.22</td>
<td>0.92</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>10282</td>
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<tr>
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<td>0.85</td>
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<td>6.28</td>
<td>17.02</td>
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<td>PU-CN-2</td>
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<td>8.70</td>
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<td>PU-CN-3</td>
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<td>0.86</td>
<td>1.83</td>
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<td>13.18</td>
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<td>25806</td>
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<tr>
<td>PU-T-1</td>
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<td>2.2</td>
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<tr>
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<td>1.8</td>
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<td>6.62</td>
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<td>134</td>
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<td>SR</td>
<td></td>
</tr>
<tr>
<td>PU-45-2</td>
<td>38.91</td>
<td>0.8</td>
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<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>20564</td>
<td>SR</td>
<td></td>
</tr>
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<td>43.46</td>
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<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>20564</td>
<td>SR</td>
<td></td>
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<tr>
<td>PU-MN-1</td>
<td>36.19</td>
<td>0.96</td>
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<td>30.4</td>
<td>5.79</td>
<td>37.24</td>
<td>123</td>
<td>25706</td>
<td>UR</td>
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<tr>
<td>PU-MN-2</td>
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<td>0.97</td>
<td>1.57</td>
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<td>6.40</td>
<td>34.20</td>
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<td>13.27</td>
<td>35.05</td>
<td>144</td>
<td>25706</td>
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<td></td>
</tr>
</tbody>
</table>

UD: Uwrap debonding  
UR: Uwrap rupture  
SD: Sheet debonding  
GF: Grip failure  
SR: Sheet rupture  
SUD: Delamination between Sheet and Uwrap
Based on the obtained test results, four-point curves, which simply represent the load-displacement behavior at the loaded end of the FRP sheet, are drawn and compared to each other. Standard deviations of the residual strength are also shown in the graphs. The real load-displacement behavior can be also found in Appendix C.

First, the effect of the height of the Uwrap on the load-displacement graph was accessed. Results obtained from PU-N, PU-ST, and PU-L were compared to each other. Based on Chen and Tang’s bond strength model (e.g. Chen and Tang 2002), the obtained effective bond length for one layer of the FRP sheet was 105 mm (see Appendix G). For the PU-N specimen, the Uwrap height was 127 mm, and for the PU-L specimen, 203 mm. For the PU-ST specimen, the Uwrap height was 51 mm, which is shorter than the calculated effective bond length of 105 mm. From the study of the Uwrap height, it was
observed that overall load-displacement was not much changed. As Figure 3-39 shows, the maximum load and stiffness from the zero loads to the maximum load were not much changed. The short height of Uwrap also increases the maximum load as much as the increases in the PU-N and PU-L specimens. However, the PU-ST specimen shows a much earlier debonding failure of the Uwrap. After the debonding of the FRP sheet, the residual strength of the PU-ST specimen due to the friction and the Uwrap is also smaller than that of the PU-N and PU-L specimens. This indicates that the anchor effect of a short height of Uwrap is also effective enough to increase the maximum load, but the Uwrap is debonded earlier than other specimens since the bonded length is short. Between the PU-N and PU-L, similar behavior and same failure mode were obtained as the bond length of both PU-N and PU-L were larger than that of the calculated effective bond length.

From this observation, it can be concluded that if the height of the Uwrap is shorter than the effective bond length, it also could increase the maximum debonding load. This is because the Uwraps arrest and delay the debonding process effectively, but could decrease ductility compared to the longer height of the Uwrap since the short height of the Uwrap could be debonded much earlier due to the lack of bonded length. It was also found that even though the Uwrap height is longer than the effective bond length, it would not increase the maximum load and residual strength effectively.
The effect of the width of the Uwrap on the load-displacement graph was accessed. Accordingly, the PU-N, and PU-SM specimens were compared to each other. Figure 3-40 shows the different obtained results from the PU-N specimen and the PU-SM specimen. As already shown, the PU-SM specimen has a smaller width of Uwrap compared to the PU-N. For the PU-SM specimen, the width of Uwrap was 32 mm, while the PU-N specimen was 64 mm. It is interesting to note that the PU-SM specimen also effectively increased the maximum debonding load of the FRP sheet as much as did the PU-N specimens. However, after the debonding load, the dropped load from the maximum load was just 17% of that of the PU-N specimens. The displacement at the complete failure of the Uwrap and FRP sheet are almost the same as that of the PU-N specimens, indicating that even though the width of Uwrap is decreased, the ductility would not be changed much. From previous observations, it can be concluded that Uwrap width is related to the residual capacity after the load drop due to the debonding of the FRP sheet. It is thought that a smaller area of the frictional behavior and smaller load-
carrying capacity due to the smaller width (50 \%) decreases the residual strength after the debonding of the FRP sheet. However, it is interesting to note that the maximum load was reached up to the level of the maximum load from the PU-N even though the width of the Uwrap was decreased up to 50 \% of the original size.

Figure 3-41: Obtained results due to the different Uwrap thicknesses

Figure 3-41 shows the effect of Uwrap thickness. It was found that even though the Uwrap thickness is increased from 1 mm (PU-N) to 2mm (PU-T), stiffness in the load-displacement axis is not much changed. On the other hand, maximum load was noticeably increased by 15 \%, and residual strength was increased by 29 \%. The complete failure mode was not a debonding of Uwrap or a rupture of Uwrap at the corner but the rupture of the FRP sheet.

To sum up the results from the varied thickness of the Uwrap, it can be concluded that a strengthening effect was obtained due to the thickened Uwrap from one layer to two layers. However, at the same time, the amount of material for the Uwrap should be also considered for the economic design point of view. For the economic design, the
amount of the material used for the strengthening could be an important factor to be considered.

Figure 3-42 shows the obtained results due to the increased number of the Uwrap. For the PU-MN specimens, the same amount of material for the Uwrap was used. The Uwrap height was also the same as that of the PU-N specimen. However, instead of using a single Uwrap, Uwrap was divided by three small Uwraps (21 mm), which have one third of the Uwrap width (64 mm) as shown in Figure 3-42.

![Graph showing results](image)

Figure 3-42: Obtained results due to the different number of Uwraps

From the PU-MN specimen, an increment of the maximum load and residual strength after the debonding of the FRP sheet was obtained. This result shows that the multiple, small Uwraps (PU-MN) could be more effective than a large, single Uwrap when the amount of the materials for both cases are the same. As explained, the quantities of Uwrap from tested specimens were the same for both cases. Unlike the PU-T specimen, which shows low strengthening effectiveness due to use of 2 layers, the PU-MN specimens increased the strengthening effectiveness. This obtained result will be used for the designing strengthening plan for the large scale T-beam.
Figure 3-43 shows the results from the change of corner length. The corner length was previously defined as the difference between the width of the concrete blocks and the width of the longitudinal FRP sheet (see Figure 3-36). As previously observed from the small scale rectangle beam, a certain amount of corner length (25 mm) was a main factor to allow a lot of slip and decrease strengthening effects.

From these pull-out tests, it was also found that the large corner length (64 mm, PU-CN) would minimize the strengthening effects and the residual load after the debonding of the FRP sheet. The dropped load due to the debonding of the FRP sheet was also much larger than that of the PU-N specimens (corner length: 5 mm). All of these phenomena indicate that the corner length is a significant factor to control both maximum load and residual strength. This implies that reducing corner width is an important design step in order to maximize the anchor effect from the Uwrap.

In contrast, obtained displacement at a failure was larger than the PU-N indicating that the ductility could be obtained from the corner length.
Figure 3-44 shows the results from the corner shape of the concrete block. As expected, the PU-SP specimen which has a sharp corner shows a smaller maximum load and a smaller residual strength. Due to the sharp edge of the PU-SP specimen, the strengthening effect from the Uwrap was decreased. However, the amount of difference is not significant. Maximum failure load and residual capacity from the PU-SP specimen was not much decreased compared to the level of the PU-N specimen by 91% and 85%.

It can be thought that due to the sharp edge of the PU-SP specimen, which affects the flaw in the bonded area around the corner, the strengthening effect from the Uwrap was decreased as observed in the small scale rectangle beam.

These test results indicate that the corner area of the beam where Uwrap would be placed should be grinded in order to build a secure contact around the corner area and an effective force transferring from the bottom of the Uwrap to the side of the Uwrap.

Figure 3-45 shows results obtained from the PU-45 specimens. The failure mode of the obtained PU-45 is not an Uwrap debonding or an Uwrap rupture at the corner. Instead, the FRP sheets were ruptured because the holding force from the 45 degree
Uwrap was larger than the tensile capacity of the FRP sheet. Hence, the debonding failure of the FRP sheet could not be observed due to early failure of the FRP sheet.

This indicates that PU-45 is holding the load more effectively compared with the PU-N specimen, which has a 90 degree fiber orientation to the loading direction. The maximum load is obtained from the rupture of the FRP sheet. Therefore, the actual anchor capacity of the Uwrap could be larger than obtained values from this study. The PU-45 specimen will be investigated further using a digital image correlation (DIC) measurement and a thermo-camera. The slip measured from the DIC and a failure sequences captured from a thermo-camera is presented in Section 3.4.1.3 and Appendix F.

3.4.1.2 Strain distributions

In this section, the strain data obtained from the mounted strain gages on the surface of the FRP sheet will be presented. For each pull-out specimen, strain gages were
attached to study the debonding propagation along the length of FRP sheet. The result will be compared with a numerical analysis for the verification in Chapter 4. Figure 3-46 shows the locations of the strain gage along the length of the FRP sheet. One strain gage was mounted on the surface of the FRP sheet near the loading end. A second strain gage was installed at the center of the Uwrap. A third was attached near the end of the FRP sheet. The orientations of those strain gages are parallel to the loading direction.

![Diagram of strain gages](image)

Figure 3-46: Locations of the strain gages along the length of the FRP sheet

During the test, data was recorded at a 10 Hz frequency in order to plot continuous and sudden changes of the strain in a short time period. As strain differences are investigated and compared with different types of specimens, arresting and delaying of the debonding propagation along the length of the FRP sheet could be observed and also compared with different types of specimens.
The obtained results from the PU-C-1, PU-CN-1, PU-N-3, and PU-45-3 specimens were compared to each other as shown in Figure 3-47. Strain differences between strain gage 1 (SG1) and strain gage 2 (SG2), and between strain gage 2 (SG2) and strain gage 3 (SG3) were plotted in charts as shown in Figure 3-47. As strain differences between neighboring strain gages are investigated and compared with different types of specimens, arresting and delaying of the debonding propagation along the length of the FRP sheet could be observed and also compared with different types of specimens. For example, SG1-SG2 increases if SG1 is in the debonded region, while SG2...
is still fully bonded to the concrete substrate (bonded area). If the debonding is propagated up to the SG 2, SG1-SG2 decreases. However, SG2-SG3 increases since now debonding is propagated up to the location of SG2 but has not yet reached to SG3. When debonding reaches the location of SG3, SG2-SG3 decreases.

With this insight about strain gage data, it could be conjectured whether the Uwrap is effectively preventing the slip and debonding propagation or not.

First, the strain gage data of the PU-C specimen was investigated. As expected, the control specimen which has no Uwrap shows a sudden debonding propagation. Once the debonding started from the location of the SG1, the speed of the debonding propagation quickly reached the location of the SG3. As Figure 3-47 (a) shows, a sudden increase and a decrease of SG1-SG2 and SG2-SG3 occur almost at the same time. This indicates that debonding was propagated at a fast speed and could not be delayed or arrested by anything since there is no Uwrap.

On the other hand, both the PU-CN-1 and PU-N-3, which have the Uwrap, show different strain data compared to the PU-C-1. In the case of PU-N-3, data from SG1-SG2 increased up until 16.7 kN and suddenly decreased. At the same time, data from SG2-SG3 starts to increase up to 22.8 kN and then suddenly deceased. This observation shows that delaying the debonding propagation and increasing the maximum load (debonding load) were a results of the Uwrap in the PU-CN-1.

The delay and increased strength could be also seen in the PU-N-3. In the PU-N-3 specimen, data from SG1-SG2 is increased up to 21.9 kN and decreased. Then the data from SG2-SG3 starts to increase up to 34.8 kN and decease. This observation also shows that delaying the debonding propagations and increasing the debonding load was gained
from the Uwrap of the PU-N-3 specimen. Furthermore, debonding propagation of the PU-N-3 was observed from the load between 21.9 kN and 34.8 kN when that of the PU-CN was observed from 16.7 kN and 22.8 kN. This indicates that the Uwrap without corner length increases the anchoring effects.

To sum up, both the PU-CN-1 and PU-N-3 were delayed debonding and increasing the capacity. However, the PU-N-3 was more effective in delaying the debonding and increasing the maximum strength as the PU-N-3 has a small corner length (5 mm) compared to that of PU-CN-1 (64 mm).

In the case of the PU-45-3, data from the SG1-SG2 is just increased and did not decrease as the FRP sheet was fractured instead of being debonded from the concrete. Therefore, the debonding process was not fully observed for this case and stopped between the location between the SG1 and SG2. This observation indicates that the debonding was not much propagated up to the location of SG2 before the FRP sheet was fractured.

Accordingly, it can be concluded that the debonding propagation could be effectively controlled as the corner length and degree of the Uwrap are adjusted. A small corner length and smaller angle of Uwrap provided much better prevention of debonding as well as an enhanced strength. Therefore, a 45 degree Uwrap (PU-45) without any corner length would be a prototype to be used as an anchor. This 45 degree Uwrap could be also effectively used for the shear strengthening. The shear strengthening of the beam using the Uwrap is beyond the scope of this study.

From the previous observations, it could be conjectured that theoretically, angles which are smaller than 45 degrees would provide better anchoring effects. However, an
angle which is smaller than 45 degrees would not be possible to be handled in the field due to geometric difficulties. At the same time, angles which are larger than 45 degrees would not be as effective as one of 45 degrees. However, it should be also considered that the 90 degree Uwrap (PU-N-3) is always going to be preferred for the construction field since the installation would be much easier. That is the reason why the decision for the selection between the 45 degree and 90 degree of the Uwrap should be made after careful considerations about the pros and cons of different angles of the Uwrap. Strain data from other specimens are found in Appendix E.

3.4.1.3 Slip profiles obtained from digital image correlation (DIC)

In this section, the slip profiles along the length of the FRP sheet will be presented. The slip profiles on the Uwrap and FRP sheet were measured using a digital image correlation (DIC) technique. To verify the obtained data of the DIC, each data point was compared with the LVDT data. Figure 3-48 compares those two sets of data. Figure 3-48 (a) shows the entire load-displacement graph and Figure 3-48 (b) shows the graph between 0 to 1.5 mm displacement in order to show differences at small loads. The results for other specimens can be also found in Appendix C.

Overall trends were well matched with the LVDT data as shown in Figure 3-48 (a). However, it can be noted that there is some disagreement in measuring slip as shown in Figure 3-48 (b) especially at the small level of load. It was found that an inaccuracy observed from entire specimens in this study was around 0.2 mm margin of error. Therefore, obtained displacements less than 0.2 mm from the DIC technique were not as
accurate as that of the LVDT data set. Maximum error could be reached up to 100% as shown in Figure 3-48 (b) if the actual slip is less than 0.2 mm. It is thought that the error was obtained from the rotation of the specimen itself due to the pulling load. It is further thought that the rotation of the specimens affects the measured displacement by DIC, resulting in disagreements compared to the data of the LVDTs.

In this study, 1704 pixels cover around 150 mm of the specimen. Therefore, one pixel has an approximate size of 0.09 mm. Since the program used for the DIC technique can measure up to one over ten of a pixel, the minimum unit for measuring slip would be 0.009 mm. The larger size of a picture with a sufficiently large subset size would show better results.

Figure 3-48: Comparison of DIC data with LVDT data (PU-MN-3)

Different trends of the slip along the length of FRP sheet were obtained depending on the various geometries of the Uwraps as a result of the parametric study. A total of three different types of specimens were compared to each other. The first type is
the 90 degree specimen. The second type is the 45 degree specimen (PU-45). The third is the corner specimen (PU-CN). All types of Uwraps which were placed at an angle of 90 degree are named the 90 degree specimens except for the PU-CN, which has a corner length. Therefore, 90 degree specimens include all types of specimens except for the PU-45 and PU-CN.

As previously seen, PU-45 specimens did not allow the debonding propagation up to the end of the FRP sheet, and all PU-45 specimens were failed by FRP ruptures. On the other hand, the 90 degree specimens could not prevent debonding propagation; instead, they delayed the debonding propagation. These observations were also found from the measured slip profiles along the length of the FRP sheet.

Figure 3-49 shows the comparison of the slip along the length of the FRP sheet near the debonding load depending on the types of Uwrap. It is interesting to note that all slip data near the debonding load from the 90 degree specimens shows mostly similar trends of the slip along the length of the FRP sheet. At the same time, the slip at the end of the FRP sheet was measured in a range between 0.2 mm and 0.3 mm. This indicates that the slip at the end of FRP sheet is almost reached up to the level of the complete debonding. From the tests for the Mode 2 fracture energy, averaged data of the complete debonding was obtained at the slip of 0.315mm. Therefore, it can be concluded from the DIC technique that the complete debonding was mostly propagated to the end of the FRP sheet. Slip profiles for each specimen and measured displacement using DIC technique are found in Appendix D.

From the previous comparison of the load-displacement curves, the PU-CN was not effectively holding the FRP sheet since it deboned at a much lower load than other
specimens. These previously observed phenomena also could be observed from the slip profiles measured by DIC as shown in Figure 3-49. It can be also observed that the 90 degree Uwrap allows more slip than the 45 degree specimens (PU-45) near the debonding load. At the location of 70 mm, the observed slip of the 90 degree specimens was around 0.65 mm. On the other hand, the observed slip of the 45 degree specimens (PU-45) was 0.31 mm. The 45 degree specimens (PU-45) show a smaller slip even though they failed at a larger load than the 90 degree specimens. The smaller amount of slip profiles is the result of the additional anchoring effect obtained from the 45 degree Uwrap. This indicates how the 45 degree specimens are effectively holding the FRP sheet, preventing the debonding failure compared to normal Uwrap (90 degree).

It can be also found that the slip of the PU-CN is smaller than that of the 90 degree specimens. At the location of 70 mm, the observed slip of the 90 degree specimens was around 0.65 mm. On the other hand, observed slip of the corner specimens (PU-CN) was 0.3 mm, which is a similar value to that of the PU-45. Even though the slip of the corner specimens (PU-CN) is smaller than that of the 90 degree specimens, it doesn’t necessarily mean the Uwrap of PU-CN was a better anchor than that of the 90 degree specimens.

Even though the slip profiles of both the PU-CN and PU-45 specimens are similar, slips obtained from PU-45 and PU-CN were in different debonding mechanisms. A smaller slip of PU-45 is due to the strong anchoring effect. The strong anchoring effect did not allow a large amount of slip along the length of the FRP sheet. On the other hand, a smaller slip of PU-CN is due to the weak anchoring effect. A weak anchoring effect
allows debonding propagation up to the end of the FRP sheet, earlier than that of the 90 degree specimen, resulting in a smaller number of slips.

![Graph showing slip along the length of the FRP sheet]

**Figure 3-49:** Measured slip along the length of the FRP sheet

From the study using the DIC technique, it could be concluded that debonding propagation could be captured from the DIC technique as well as the amount of the slip along the length of FRP sheet. Furthermore, it was found that the 45 degree Uwrap could be effectively used for preventing or delaying the debonding propagation compared to the normal Uwrap (90 degree).

### 3.4.1.4 Comparison of different type of specimens

Obtained results from each pull-out test are compared in this section. Each data previously shown in Table 3-13 was averaged, and the averaged values were compared in terms of the strength enhancement, the ductility, and the efficiency with varied Uwrap geometry. For strength enhancement, the increment of maximum strength due to the addition of Uwrap was calculated. The ductility ratio was calculated as a ratio of
displacement at maximum load to a displacement at complete failure. The Uwrap
efficiency was also calculated. The equation used for calculating the efficiency is shown
in Eq. 3.2.

\[
\frac{\text{Max Load} \times \text{Max Disp.} \times \text{Average Load after drop}}{\text{Disp. after drop} \times \text{Dropped load} \times \text{Amount of Material}}
\]

3.2

Calculated results are normalized based on obtained data from PU-N specimens.
Therefore, the efficiency of the PU-N specimen was set to one.

Figure 3-50 shows the comparison of strength enhancement. Maximum load can
be seen from PU-45D specimens. Most specimens show similar values except for the PU-C
and PU-CN specimens.

![Strength enhancement chart]

Figure 3-50: Comparison of the strength enhancement

The PU-C specimen was the specimen without the Uwrap. For the PU-CN
specimens, it can be seen that maximum load is not increased as much as other types of
specimens, indicating that the anchorage effect would be significantly diminished if the
corner length is long. From this chart, it can be concluded that the Uwrap is a very
efficient anchor to increase the maximum debonding load. It was also found that the PU-MN shows relatively large strengthening effects compared to the other specimens.

Figure 3-51 shows the ductility ratio. In this study, ductility ratio was defined as a ratio of the displacement at the maximum load to the displacement at the complete failure. As expected, the ductility ratio of the PU-C specimens is zero since there was no Uwrap, indicating that it could not hold any load after debonding between the concrete and FRP. The ductility ratio of the PU-45 specimens is also zero because three PU-45 specimens failed by the rupture of the FRP sheet. Among the tested specimens, the PU-T specimens show the largest ductility ratio. However, the strength was not increased as much as an increase in the ductility even though two layers of Uwrap were used. Therefore, it can be concluded that increasing the layers of Uwrap would increase the ductility ratio but would not affect the maximum strength as much as the ductility. It can be recommended that if large ductility is needed, two layers of Uwrap could be an appropriate option for the Uwrap design.

PU-CN specimens also show a large ductility ratio compared with other types. However, as previously explained, PU-CN specimens show smaller maximum load and residual strength compared to other types of specimens. It was mentioned that the PU-CN specimen is the only specimen which has a corner length since the width of concrete block is larger than the width of the FRP sheet. Therefore, it can be concluded that the increased corner length would decrease the strength enhancement but increase the ductility ratio.
Based on developed Eq. 3.2 and the obtained test results, the efficiency was calculated and compared among tests specimens. Figure 3-52 shows the comparison of calculated efficiencies from all types of specimens. Since Uwrap is not mounted to the PU-C specimen, and the FRP sheet was ruptured for the PU-45 before the debonding occurs between the concrete and FRP sheet, efficiencies for both types of specimen were not calculated. Among the tested specimens, it can be seen that only one type of specimen (PU-MN) shows a larger efficiency value than the PU-N specimen. This indicates that other type of specimens tested in this study were not as effective as the PU-N specimens. For the PU-L specimen, bonded length was longer than effective length. For the PU-ST specimen, bonded length was smaller than the effective length. Accordingly, smaller efficiency values were obtained from both types of specimens. Efficiency value is also decreased from the PU-SP specimen. It can be thought that the bad bond area and stress concentration along the shear edge would decrease the efficiency value.
From the PU-SM specimen, it can be found that if the width of the Uwrap is decreased, the efficiency would be decreased. However, it is difficult to conclude that the increased width of the Uwrap will increase the efficiency because the result from the PU-MN shows the highest efficiency value (1.59) even though the width of the FRP sheet was smaller compared to that of PU-N. These opposite results from the PU-SM and PU-MN can be explained as a concept of the effective width length. It is thought that there is effective width length for the Uwrap. Accordingly, if the width of the Uwrap is narrower than the effective width, the efficiency could be decreased. Likewise, if the width of the Uwrap is wider than the effective width length, the efficiency could be also decreased. The best efficiency would be calculated from the effective width of the Uwrap. Therefore, the effective width of the Uwrap should be considered in future studies.

From these observations, it can be concluded that a more efficient Uwrap design should consider the following recommendations.
Following design recommendations of FRP Uwrap could be used for effectively preventing or delaying the debonding propagation between the concrete and the FRP sheet.

1. The corner length should be removed or minimized
2. The corner of the beam should be grinded to remove the sharp edge
3. The side length of the Uwrap should be at least the calculated effective length ($L_e$)
4. The width of the FRP sheet should be the same as the width of the soffit of the concrete beam
5. Many-small Uwraps rather than a single large Uwrap are recommended.
6. Two or three layers of 90 degree Uwrap could be used for obtaining the ductility; however, the maximum load would not benefit from many layers of Uwrap.
7. The tilted geometry of the Uwrap (45 degree) should be used if an increment of maximum debonding load is needed rather than an increment of ductility.

### 3.5 Large scale T-beam test

Based on the obtained results and derived conclusions from the parametric study of the pull-out tests, a large scale T-beam was designed and fabricated. The purpose of the test on the large scale T-beam is to see the effect of the Uwrap on a practical scale. Specific design considerations for the T-beam follow the preliminary tested T-beam
design. However, some modification steps were taken to see the Uwrap effects clearly. The designed T-beam was tested under the four point bending loads. The local strain data in the Uwraps, strain profiles along the length of FRP, load-displacement graphs, and failure mode are presented in this section. Obtained results will be also compared with the developed numerical analysis in Chapter 4.

### 3.5.1 T-beam design

The dimensions of the designed T-beam are the same as those of the preliminary T-beam. However, the rebar and stirrup were modified due to the limitation of the maximum hydraulic machine (445 kN). The moment span in the four-point bending test was also increased to boost the debonding between the concrete and FRP sheet in the shear span.

For tensile reinforcement, the minimum reinforcement limit of AASHTO was used. Instead of using one layer of FRP sheet, a thickened FRP sheet with three layers was used. The increased moment span in the four-point bending test and the thickened FRP sheet were considered for the design steps in order to activate the Uwrap. The specific cross sectional design is shown in Figure 3-53.
Rebar stirrups were spaced at every 10 cm in the shear span. For the moment span, larger spacing (24 cm) was used. The narrow spacing is mostly designed for preventing shear failure of the beam due to the increased maximum applied load of the strengthened T-beam. Specific designs for the stirrups are shown in Figure 3-54.

After fabricating the T-beam, the beam was strengthened by the FRP sheet and Uwrap. This beam was not precracked before the strengthening. The cracking moment of the beam was expected to be much lower than the moment of the rebar yielding with a minimum effect on the overall behavior. Test results could be found in a later section. For FRP, a carbon fiber sheet (SCH-41s) was used. This material is an ICC ER-2103 listed material comprised with Tyfo S epoxy. Specific material properties were already shown in the previous section. Since the three layers of the FRP sheet would be attached to the soffit of the T-beam, sagging could be an issue due to the weight of the thickened three layers during installation. To prevent sagging problem, Tyfo TC is recommended from the manufacturer. However, it is difficult to mix component A and B of Tyfo TC due to
the high viscosity. Alternatively, Cabosil (Silica fume) was used and mixed with the Tyfo S epoxy to increase the viscosity as the manufacturer recommended.

Figure 3-54: Shear stirrup design and longitudinal geometry of the T-beam

Figure 3-55 shows the pictures taken from the mixing the Tyfo S epoxy with the Cabosil and the application of the FRP sheet to the soffit of the T-beam. From the fabrication, it was found that the mixing procedure for making a tacky epoxy could be much easier if the Cabosil is mixed with Tyfo S rather than mixing the two components of Tyfo TC.
For the longitudinal FRP sheet, three layers were used. For Uwrap, 2 layers were used. Two layers of the Uwrap were considered to prevent the stress concentration of the Uwrap at the corner area of the T-beam. Based on ACI debonding calculations, the beam was designed to fail at 191 kN of loading due to the debonding of the FRP sheet. A final design of a fabricated beam is shown in Figure 3-56. A 45 degree angle of Uwrap was selected after consideration of the pull-out test. Accordingly, the corner length was closed to zero as the FRP width was designed to be the width of the web of the T-beam. It should be also mentioned that this Uwrap is only designed for anchor effects. Shear strengthening effect due to the addition of the Uwrap was neglected and not investigated in this study.
3.5.2 Set up for the T-beam test

A test set up will be shown in this section. In order to activate the Uwrap, a wider loading span was selected compared to the preliminary tests as seen in Figure 3-56. A wider loading span would generate the longer maximum moment span, resulting in longer maximum strained zone in the FRP sheet along the length of the T-beam.
The longer maximum strained zone (2.66 m) would activate the Uwrap more efficiently compared to a shorter one (1.33 m) as the strained FRP sheet is pulling the Uwrap. Strain gages were attached to the all Uwraps and to the longitudinal FRP sheet. A total of 16 strain gages were attached to the Uwrap, and 6 strain gages were attached along the length of FRP sheet. One strain gage was attached to measure the concrete

Figure 3-57: Test set up and locations of strain gages
strain at the top in the mid-span. A total of 3 strain gages were attached to the rebar to measure the load of the rebar yielding. Another two strain gages were also mounted to the stirrups. Two LVDTs and one string pot were used to measure the mid-span deflection due to the four-point bending load. The specific test set up and gage locations are shown in Figure 3-57. To prevent the rotation of the beam due to the torsional effect, 4 braces were also fabricated to support both sides of the beam. All the gages and measuring devices were connected to one data acquisition system. During the test, data was collected at a 2 Hz frequency. A constant displacement rate (0.5 mm / min.) was used for the two loading heads.

3.5.3 Load-displacement graph

An obtained load-displacement graph from the test is shown in this section. Data from strain gages from the Uwrap and strain profiles along the length of FRP sheet will be shown in Chapter 4 as those data are compared with numerical results. Therefore, this section will only show the global behaviors, obtained failure mode, and important observations from the designed beam. Figure 3-58 shows a load-displacement graph obtained from the T-beam test.
First of all, rebar was yielded at 119 kN. After the yielding of the rebar, a gradual debonding process occurred. In the laboratory, the debonding sound was captured by observers from the load of 190 kN approximately. It is thought that the partial debonding was started to propagate from the mid span to the supports after the load of 190 kN. The concrete at top of the T-beam did not fail by the compression as the strain data shows 0.00159 as a maximum value. At the load of 305 kN, Uwrap 6 was debonded. After the debonding of Uwrap 6, a small amount of the load was dropped from 305 kN to 300 kN, which is a negligible difference. It is thought that the debonding was propagated to the Uwrap 7 from Uwrap 6 as Uwrap 6 was completely debonded from the concrete as Figure 3-59 shows.

Figure 3-58: Load-displacement graph and failure modes obtained from the flexure test
After the Uwrap is debonded, a few more loads were taken by the FRP sheet, and partial FRP ruptures were observed. The side of the partial FRP sheet was firstly ruptured and the following ruptures occur from the side to the center. Before the complete rupture of the FRP sheet, the test was stopped. When the test was stopped, 60% of the FRP width was still remaining. From this test, it can be known that the designed Uwrap is holding effectively, delaying the debonding process and changing the failure mode from debonding to FRP ruptures. By using the designed anchor, 60% of the increased strength was obtained, and the failure mode was changed from the debonding of the FRP sheet to the rupture of the FRP sheet. A more specific explanation will be presented with the strain data in Chapter 4.
3.6 Summary of experimental works and principal findings

Preliminary studies on the two large scale T beams and the small scale beam were conducted in order to observe the behavior of the Uwrap in the strengthened RC-beam. From the observations of the preliminary study, experimental works in order to obtain basic material properties were planned for the concrete, the concrete-epoxy interface, and unidirectional FRP sheet. Fracture energies of the concrete-epoxy interface in Mode 1 and Mode 2 direction were investigated. Furthermore, an attempt to obtain modified fracture energy under the Uwrap location was conducted. The parametric study of the pull-out test was conducted to observe the Uwrap behavior. Important parameters were selected, and specimens were fabricated based on the selected parameters. Failure mode of pull-out test was dependent to the geometry of the Uwraps. In order to control the failure mode (minimize the stress concentration effects and maximize the anchor effect of the Uwrap), two layers of Uwrap with 45 degree orientation were used for strengthening a large scale T-beam. Finally, a large scale T-beam was fabricated and strengthened based on the results from the parametric study of the pull-out test. The T-beam was tested under the four-point bending set up in order to see the anchor effects of the Uwrap. The principal findings for the experimental programs are as follows.

1. From the preliminary study, the failure modes of both T-beams were the rupture of the FRP sheet. In this case, the Uwrap was not activated much. On the other hand, the debonding of the FRP sheet followed by several partial debondings of the Uwrap was obtained from the small scale T-beam. Even after the complete debonding between the FRP and the soffit
of the concrete, the Uwrap was holding the FRP sheet, transferring stress to the side of the beam. From the observations, it was found that the Uwrap can be utilized in order to design the ductile failure in the externally bonded FRP system. It was also found that the slip effect between the FRP sheet and the Uwrap after the debonding of the FRP sheet are critical factors to control the overall behavior of the strengthened beam.

2. From the fracture test in Mode 1 direction, it was concluded that the displacement rate is not a sensitive parameter to control the fracture energy. However, it should be noted that the loading head should move slowly enough to minimize a dynamic effect on the specimens.

3. From the fracture test in Mode 1 direction, an addition of the FRP sheet in the three-point fracture specimen could increase the fracture energy significantly. However, it should be noted that the bending moment generated from the Uwrap was also added to the obtained fracture energy, so the actual fracture energy could be lower than this. Accordingly, it can be expected that the fracture energy under the Uwrap in Mode 1 direction could be much larger than that of a normal interface without Uwrap.

4. From the fracture test in Mode 2 direction, it was found that there is residual stress between the concrete and FRP even after the complete debonding between those due to the addition of the Uwrap. It was also found that the maximum shear stress and fracture energy in Mode 2 direction was significantly increased. However, it should be noted that the
bending moment generated from the Uwrap was also added to the obtained fracture energy, so the actual fracture energy could be lower than this.

5. From the material characterization for the unidirectional FRP sheet, it was found from the shear notched beam test on the FRP sheet that one layer of the FRP sheet has a smaller modulus and shear strength compared to that of the stacked layers of the FRP sheet. It is thought that since the fractured area in unit length of the stacked layers of the FRP sheet is larger than the single layer of the FRP sheet, larger test results were obtained from the stacked layers of the FRP sheets.

6. The increased debonding load of the FRP sheet due to the Uwrap is not sensitive to the height of the Uwrap as long as the height is at least a length of effective bond length.

7. The debonding load of the FRP sheet is sensitive to the thickness, corner length, and angle of the Uwrap. The corner length of the Uwrap is a main cause for the decrements in both debonding and residual strength.

8. Within the same amount of Uwrap, three small Uwraps rather than a single large Uwrap increased the both debonding load as well as residual strength, indicating that multiple, small Uwraps are more effective than a large, single Uwrap.

9. From the parametric study, 45 degrees of the Uwrap without the corner length would be the appropriate geometry of the Uwrap in order to maximize the benefit of the anchoring effect.
10. It was found that the digital image correlation technique (DIC) is the appropriate method to measure the slip along the FRP sheet. However, there were some limitations for a continuous recording and the 3D behavior of the specimen. For further study, the high speed data acquisition system for the continuous data collecting from the DIC and two more cameras to capture the 3 dimensional deformation and strain are recommended.

11. It was found that it is difficult to distribute the concentrated heat from the halogen lamp to the entire surface of interest on the specimens evenly. Alternatively, without applying the heat of the halogen lamp, a thermo-camera was used to capture the increased temperature generated from the frictional behavior between the FRP and concrete. The debonding propagation and the stressed zone were captured from this method.

12. From the large scale T-beam test, it was shown that the appropriate Uwrap design increases the failure load significantly compared to the value calculated by ACI debonding equation. Furthermore, the failure mode was also changed from the debonding of the FRP sheet to the rupture of FRP sheet due to the anchor effect of the Uwrap.
Chapter 4
Analytical program

In this chapter, results from the analytical studies will be discussed. First, an outline of the analytical program will be presented. Second, the concept of the developed frictional bond-slip model for the analytical study will be introduced. Last, obtained results of the numerical analysis will be presented and compared with the experimental results described in Chapter 3.

4.1 An outline of the analytical program

Figure 4-1 shows the different analytical models that were developed. It was found from this study that the frictional behavior between the debonded surfaces of concrete and the FRP sheet significantly contributes to the test results. When this frictional behavior was taken into account, numerical models show better convergence. This frictional effect was, therefore, studied and incorporated into a bond-slip model in the Mode 2 direction, resulting in the Mode 2 frictional bond-slip model. This model is used in the cohesive element as an interface property under the location of the FRP Uwrap. The concept and steps taken for the development of the frictional bond-slip model will be explained in detail in the next section.
Based on the existing constitutive models for the concrete and FRP sheet reviewed in Chapter 2 and proposed frictional bond-slip model in this chapter, the numerical study was conducted using the commercially available finite element program ABAQUS 6.8-1. Basic material properties obtained experimentally in Chapter 3 were used for the FE analysis. Three types of FE models were developed.

For the numerical study, first, the pull-out test was modeled in three dimensions (3D). Based on the obtained results from the 3D models, 2D models for the pull-out tests were also developed as a transition work. It should be noted that one of the important objectives for this study is to develop an effective 2D model rather than heavy and
inefficient 3D models. Some steps were taken in order to simplify the 3D models to the 2D models as a transition work.

Developed 2D models of pull-out tests will be verified with both 3D models and experimental results of the pull-out test in order to check whether or not reasonable results were provided by the 2D models.

Last, the large scale T-beam was modeled in 2D using the modeling techniques used for the models of the pull-out tests. The model was verified with the test results. The obtained results from the developed models were also used for observing important mechanisms which could not be observed from the experimental work due to the limitation of the experiments. Parametric studies for the pull-out tests and the large scale T-beam were also conducted using the developed models and, finally, some design recommendations will be provided.

### 4.2 Proposed frictional bond-slip model

In this section, the developed frictional bond-slip model will be explained. It was previously mentioned that the cohesive element will be used for the interface behavior between the FRP and the concrete. However, Mode 1 behavior in compression could not be modeled using the cohesive element in ABAQUS program since the cohesive layer was made not to be damaged under pure compression in ABAQUS. Therefore, the cohesive element could not differentiate the frictional stress generated due to the confining force (compression) from the Uwrap. Accordingly, the bond-slip behavior was
modified and a new bond-slip model is proposed in order to consider the interface frictional behavior and fracture behavior simultaneously.

Figure 4-2 shows the bond-slip behavior between the concrete surface and the FRP sheet. Each number is showing incremental debonding steps. These numbers will be also referred to for the description of Figure 4-4. In the bond-slip behavior between the concrete surface and the FRP sheet, the concrete will be stretched due to the pulling load from the FRP sheet. No confinement effect occurs in this case since there is no Uwrap (Step 1). Accordingly, when FRP is completely debonded (Step 2), no friction will be generated between the concrete surface and the FRP sheet. There is no confinement (Uwrap) effect after an occurrence of the debonding, and no force will be carried by the FRP sheet (Steps 3 and 4).

![Figure 4-2: Illustration of an ordinary interface behavior](image-url)
Figure 4-3 describes the modified bond slip behavior due to the addition of the Uwrap. Each number is showing debonding steps. The earliest step is 0, and final step is 4. Step 1 shows the deformation due to the pulling out load in Mode 1 direction. As the interface is deformed, the reaction stresses from the Uwrap are acting upon the interface, resulting in an increased onset of the debonding stress.

![Diagram of interface behavior with Uwrap]

**Interface with Uwrap**

Figure 4-3: Illustration of the interface behavior with the Uwrap

In Step 2, with the generated stresses from the Uwrap (confinement effect), a frictional behavior is generated. This frictional effect increases as the slip between the FRP sheet and the body of the concrete increases (Step 3 and 4) since the increased strain of the Uwrap due to an increased slip generates the larger reaction stress acting into the interface between the two debonded surfaces. Based on these two different interfacial behaviors, the modified bond-slip behavior under the Uwrap region was identified and
developed. In this study, this model is called the frictional bond-slip model.

These two different debonding procedures are compared in a form of the bond-slip (shear stress-slip) in Figure 4-4. The frictional bond-slip model will be used for the material properties of the interface in the region under the Uwrap locations.

![Figure 4-4: Ordinary bond-slip model and proposed frictional bond-slip model](image)

The shape of the frictional bond-slip model was developed based on observations of the behavior of the pull-out specimen with the Uwrap, as described in Chapter 3. Even though the experimentally obtained results have shown to be very sensitive to the geometries and properties of the Uwrap and the concrete, it was assumed that the interface bond-slip behavior between the concrete and the FRP under the Uwrap would have a similar trend for all cases.

Results from the pull-out tests and modified Mode 2 fracture test support the idea that the maximum shear stress would increase (Step 1). Therefore, it was assumed that
the maximum shear stress (onset of the damage) would be increased as seen in Point 1 of Figure 4-4. These numbers also indicate the steps which were previously shown in Figure 4-3. If the stress is applied to the interface, the interface would be dilated. This dilation would be confined by the Uwrap. Therefore, the shear stress would increase due to the generated confining effect of the Uwrap. This is shown as \( \tau_1 \) in Figure 4-4. On the other hand, it was assumed that a slip at Point 1 would not be changed. The unique physical deformation (slip) of the material at the onset of damage \( (s_1) \) and complete damage \( (s_2) \) would not be changed even though there is an Uwrap effect on the interface.

Results from the pull-out tests and modified Mode 2 fracture test also support the idea that the shear stress \( (\tau_2) \) does not fall to zero stress when a complete debonding occurs between the concrete and FRP sheet (Step 2). After debonding of the FRP sheet, a residual frictional stress would gradually increase up to a new plateau (Step 3 and Step 4). This residual stress \( (\tau_2 \text{ and } \tau_3) \) is considered as a frictional stress due to the confining effect from the Uwrap even after the complete debonding of the interface. These observed trends from experiments were used to develop the new frictional bond-slip model for this study.

This frictional behavior is gradually increased up to the maximum level of the frictional stress, as both Point 3 and Point 4 represent. It can be expected that the increase in slip would cause an increase in strain along the Uwrap, providing larger frictional stresses. This phenomenon can be also well explained with the previously shown Eq. 2.22 in Chapter 2.

To determine those four points of the frictional bond-slip model, the original bond-slip behavior should be obtained experimentally. It should be noted that Points 1, 2,
3 and 4 are geometry dependent in the vertical direction (shear stress). Points 2 and 3 are also geometry dependent in the horizontal direction (slip).

Point 1 of the frictional bond slip model is determined based on the amount of deformation which can be calculated from the Mode 1 and Mode 2 fracture behavior. Due to the load of pulling out in Mode 2 direction, the concrete and epoxy would also stretch in the Mode 1 direction. This deformation creates strains in the Uwrap. The strained Uwrap starts to generate the reaction stress onto the interfaces in the Mode 1 direction. Accordingly, the maximum stress would increase depending on the level of the reaction stress from the Uwrap. A deformation in the Mode 1 direction at the onset of damage in Mode 2 direction was calculated from three variables: A deformation at a complete damage \( w_{1c} \) in the Mode 1 direction, a deformation at the onset of the damage \( w_{2m} \) in the Mode 2 direction and a deformation at complete damage \( w_{2c} \) in the Mode 2 direction. These three variables were obtained experimentally.

The calculated deformation in Mode 1 direction at the onset of the damage in the Mode 2 direction was used to calculate the generated stress from the Uwrap. Then, the amount of deformation was used to calculate the strain along the Uwrap in the fiber direction. Accordingly, Eq. 4.1 is derived for shear stress of the Point 1 \( (\tau_1) \). The derivation of the Eq. 4.1 can be found in Appendix G in detail. As previously explained, the unique physical deformation of the material, slip at Point 1 \( (s_f) \) would not be changed while the level of the stress is changed due to the Uwrap effect.

\[
\left(1 + \frac{4E_{\text{FRP}}t_{\text{FRP}}w_{2m}w_{1c}}{I_b^2 f_i w_{2c} \cos \theta} \right) \tau_{\max}
\]  
4.1
where

\( E_{\text{FRP}} \): Elastic modulus of the Uwrap

\( t_{\text{FRP}} \): Thickness of the Uwrap

\( l_b \): Width of Uwrap which covers the FRP sheet in the soffit of the concrete

\( f_i \): Maximum tensile strength of the interface

\( w_{2c} \): Complete slip in Mode 2 direction

\( w_{2m} \): Slip at onset of damage

\( w_{1c} \): Complete slip in Mode 1 direction

\( \theta \): Angle of Uwrap

\( \tau_{\text{max}} \): Maximum shear stress from ordinary bond-slip curves

In order to determine the location of Point 2 \((\tau_2, s_2)\), it should first be recognized that Point 2 is a transition between the complete damage of the interface and the frictional behavior between two debonded surfaces. A frictional shear stress \((\tau_2)\) would be in the lowest level at this time since the Uwrap has relatively small strain due to the small amount of pull-out displacement (slip). Therefore, a frictional shear stress \((\tau_2)\) in Point 2 would be smaller than the maximum frictional shear stress but larger than zero stress. For this study, this value was taken from the experimental value of the PU-CN specimen of the pull-out tests. Since the PU-CN specimen has a corner length (a difference between the width of the FRP sheet and width of the concrete), the Uwrap was not stretched much.

As shown in Figure 4-5, if the specimen has no corner length, the Uwrap would be
stretched effectively. By contrast, the specimen which has a corner length would not be stretched effectively.

Accordingly, the frictional behavior of the PU-CN was used for estimating the minimum frictional stress ($\tau_2$) between the concrete and FRP sheet. The experimentally obtained frictional stress from the PU-CN specimen was calculated to a value of 3.5 MPa.

This level of the frictional stress ($\tau_2$) would increase up to the level of stress in Point 3 ($\tau_3$) since an increase in slip causes an increase in strain along the Uwrap.

Figure 4-5: Different strains in Uwrap depending on existence of the corner length

![Diagram showing different strains in Uwrap depending on existence of the corner length](image)
providing larger frictional stresses as explained previously. However, it should be noted that there is a limit for the level of strain in the Uwrap. This limit is controlled by either the strain due to the stress concentration at the corner or the strain at the debonding of the Uwrap. If the strain at the Uwrap corner due to stress concentration is smaller than the debonding strain, the Uwrap corner failure would occur, indicating that the maximum strain of the Uwrap will be the strain due to the stress concentration effect at the corner. Once the Uwrap corner fails the frictional effect is removed (Point 4) as the confinement effect also vanishes. Likewise, if the obtained strain at the corner due to the stress concentration effect is larger than the debonding strain of the Uwrap, the Uwrap would be holding the FRP sheet and generating the frictional behavior up until complete debonding of Uwrap. In this case, generated frictional stress due to the Uwrap (Point 3) would be constant until the Uwrap is debonded (Point 4). Once the Uwrap is debonded, the frictional shear stress would be removed (point 4) since the Uwrap is no longer confining the interface. Accordingly, Eq. 4.2 is derived from Eq. 2.21.

\[ v_u = k_1 \rho e E_{FRP} + k_2 f'_c \]

where, \( e = \min\{e, e_d, e_s\} \)

Where

\( k_1 \): Frictional coefficient 1 (0.36)

\( k_2 \): Frictional coefficient 2 (0.088)

\( v_u \): Shear frictional strength

\( \rho \): FRP volumetric ratio to the body of the concrete

\( e \): Strain on the Uwrap
\( \varepsilon_d \): Debonding strain of the Uwrap

\( \varepsilon_c \): Strain due to the effect of stress concentration at the corner

\( E_{FRP} \): Elastic modulus of the FRP sheet

\( f_c \): Concrete compressive strength

As explained previously, Eq. 2.21 was developed by Saenz and Pantelides (2005) in order to estimate the frictional shear strength between the concrete and concrete due to the clamping force of the externally bonded FRP sheet. In their study, the generated frictional stress is between the concrete and the debonded FRP sheet. Accordingly, they used the coefficients \( k_1 \) equal to 0.505 and \( k_2 \) equal to 0.117. For the study presented in this thesis, 3D models of pull-out tests were used to calibrate these two coefficients with the experiments. The best curve fitting was obtained from the values of \( k_1 \) equal to 0.36 and \( k_2 \) equal to 0.088. These values were also used for estimating the frictional shear stress (\( \tau_3 \) and \( \tau_4 \)) between debonded surfaces of the concrete and FRP sheet. Using this modified equation, the level of shear stresses at Point 3 and Point 4 was determined.

The slip at Point 3 \((s_3)\) was experimentally determined. The slip at Point 3 \((s_3)\) is varied depending on the corner length of the Uwrap. As seen from the pull-out experiments, the best anchor effect obtained from the Uwrap would be obtained from the Uwrap which does not have a corner length. The anchor effect was significantly decreased as the corner length is increased. For the Uwrap (PU-N, PU-L), which does not have a corner length (5mm), 2 mm was measured as an amount of slip between Point 2 and Point 3. On the other hand, as seen in the previous chapter, the specimens (PU-CN), which have 64 mm of the corner length, showed a 20 mm slip between Point 2 and Point
3. Therefore, it is an important step to reduce the corner length in order to obtain maximum anchor effects.

The same level of shear stress from the Point 3 ($\tau_3$) was applied to the shear stress in Point 4 ($\tau_4$) since there is a strain limitation for the Uwrap which can be obtained from the debonding between the concrete and Uwrap. If the Uwrap length is long, the slip between Point 3 and Point 4 would increase. If the Uwrap length is small but larger than the effective bond length, the slip between Point 3 and Point 4 would decrease. If the Uwrap length is smaller than the effective bond length, the debonding strain would decrease, and Point 4 would be removed in the graph because the Uwrap would be debonded before it reached up to the maximum frictional stress (shear stress).

These four points were determined to form the frictional bond-slip model and incorporated into cohesive elements under the Uwrap region.

4.3 Three dimensional modeling of pull-out specimens

Previously tested pull-out specimens were first modeled in three dimensions (3D). The finite-element program Abaqus 6.8-1 (e.g. Hibbitt et al. 2007) was used for the numerical analysis. For modeling the concrete epoxy interface, the cohesive element was used in order to take into account both Mode 1 and Mode 2 behaviors. For the FRP and Uwrap, shell elements with Hashin failure criteria were used. Previously introduced material models and the properties for the concrete, interface (cohesive element), and orthogonal properties of the FRP sheet (FRP sheet and Uwrap) were employed during the simulations.
Based on experimental observations on the pull-out test in Chapter 3, the concrete failure was not observed from all tests. To reduce the amount of elements, only the interface regions close to the FRP layer were modeled. Figure 4-6 shows the typical finite element mesh and components of pull-out specimens used for this model.

Figure 4-6: Modeling of pull-out specimens
Three different layers, the FRP sheet, the Uwrap and the interface, are generated separately and joined together as one body using a “TIE” command. The TIE command ties two surfaces together for the duration of a simulation. The translational and rotational motions as well as other degrees of freedom are equal for a pair of tied surfaces. Nodes of the refined surface were defined as slave nodes and the nodes in the relatively coarse surface were selected as a master node.

Only half of the specimen was modeled since it is symmetrical. Experimentally obtained material properties are used for the longitudinal FRP sheet, Uwrap and the interface between the concrete and the FRP sheet. Table 4-1 shows the material properties used for this analysis.

Table 4-1: Required material properties for the modeling of the pull-out test

<table>
<thead>
<tr>
<th>Properties</th>
<th>Interface between concrete and FRP (Mode1)</th>
<th>Interface between concrete and FRP (Mode 2)</th>
<th>FRP and Uwrap</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of elasticity, $E$ (GPa)</td>
<td>4.2</td>
<td>__</td>
<td>72.4</td>
</tr>
<tr>
<td>Transverse Modulus of elasticity, $E_t$ (GPa)</td>
<td>__</td>
<td>__</td>
<td>9</td>
</tr>
<tr>
<td>Shear Modulus of elasticity, $G$ (GPa)</td>
<td>__</td>
<td>__</td>
<td>1.66</td>
</tr>
<tr>
<td>Slip Modulus, $K_s$ (MPa/mm)</td>
<td>__</td>
<td>84.6</td>
<td>__</td>
</tr>
<tr>
<td>Tensile strength, $f_t$ (MPa)</td>
<td>3.6</td>
<td>__</td>
<td>879</td>
</tr>
<tr>
<td>Shear strength, $\tau$ (MPa)</td>
<td>__</td>
<td>4.74</td>
<td>36</td>
</tr>
<tr>
<td>Transverse Tensile strength, $f_{tt}$ (MPa)</td>
<td>__</td>
<td>__</td>
<td>38</td>
</tr>
<tr>
<td>Compression strength, $f_c$ (MPa)</td>
<td>32</td>
<td>__</td>
<td>2107*</td>
</tr>
<tr>
<td>Transverse Compression strength, $f_{ct}$ (MPa)</td>
<td>__</td>
<td>__</td>
<td>136*</td>
</tr>
<tr>
<td>Mode 1 Fracture energy, $G_{F1}$ (N/m)</td>
<td>128</td>
<td>__</td>
<td>91600*</td>
</tr>
<tr>
<td>Transverse Mode 1 Fracture energy, $G_{F1}$ (N/m)</td>
<td>128</td>
<td>__</td>
<td>203*</td>
</tr>
<tr>
<td>Mode 2 Fracture energy, $G_{F2}$ (N/m)</td>
<td>__</td>
<td>749</td>
<td>__</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.2</td>
<td>__</td>
<td>0.27*</td>
</tr>
</tbody>
</table>

Values marked with * indicate estimated material properties. Other properties were found experimentally.
Specific element types and interface connections are described as shown in Figure 4-7. The orthotropic properties of the FRP sheet and FRP Uwrap are considered. The fiber direction indicates the strong axis of the FRP sheet. The axis perpendicular to the fiber direction is the transverse axis of the FRP sheet, which is much weaker than the fiber direction in tension. The Uwrap properties are the same as those of the FRP sheet since the Uwrap is also made of the same material as the FRP sheet. For the interface between the concrete and FRP element, cohesive elements were used. This cohesive element contains both Mode 1 and Mode 2 behavior in one element. The specific theory about the cohesive element was explained in Section 2.4.2. Basically, two different fracture behaviors of one material under two different modes, such as Mode 1 and Mode 2, are incorporated into the cohesive element. Mode 2 behaviors in shear direction 1 and shear direction 2 were assumed to be the same. Bottom surfaces of the cohesive elements were fixed to all degrees of freedom. Stress concentration factor for 3D models were not considered because the stress concentration can be captured at the corner area using three dimensional models. However, the developed 2D FE models use the stress concentration factor. It will be explained in later sections.
To overcome the mesh mismatch between the three different layers, the same element size, 2mm by 2mm, was used for the FRP and Uwrap elements. For the interface elements, 2 mm by 2 mm by 1 mm thickness was used. As small sizes of elements were used, local failure and damage propagation along the length of the interface, FRP sheet and Uwrap could be captured. Between the interface element and FRP elements and between the Uwrap and FRP sheet, “TIE” connections were used. All explained models are summarized in Table 4-2. For FRP materials, reduced integration shell element (S4R)

Figure 4-7: 3D finite element model of pull-out specimen
was used. For the interface elements, three dimensional cohesive element (COH3D8R) was used.

Table 4-2: Summary for the 3D finite element model of the pull-out test

<table>
<thead>
<tr>
<th>FRP element</th>
<th>type</th>
<th>Behavior</th>
<th>Number of elements*</th>
</tr>
</thead>
<tbody>
<tr>
<td>FRP sheet</td>
<td>S4R</td>
<td>Elastic-Orthotropic</td>
<td>2125</td>
</tr>
<tr>
<td>Uwrap</td>
<td>S4R</td>
<td>Elastic-Orthotropic</td>
<td>1984</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Interface element</th>
<th>type</th>
<th>Behavior</th>
<th>Number of elements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Corner</td>
<td>COH3D8</td>
<td>Mode 1 + Mode 2 Fracture</td>
<td>128</td>
</tr>
<tr>
<td>Side</td>
<td>COH3D8</td>
<td>Mode 1 + Mode 2 Fracture</td>
<td>1984</td>
</tr>
<tr>
<td>Top</td>
<td>COH3D8</td>
<td>Mode 1 + Mode 2 Fracture</td>
<td>1615</td>
</tr>
</tbody>
</table>

* “Only Normal” models, number of elements could be changed as Uwrap geometry is changed.

The number of elements used to create the specimen of the PU-N (Uwraps placed at an angle of 90 degree) is also shown in Table 4-2. A total of 7,836 elements were used, even though the body of concrete block was not modeled. If the body of concrete was modeled, in this case, the element number would significantly increase. For the 3D models in this study, only the FRP and its interface were modeled since all failure types were obtained in the interfaces between the concrete and the FRP. Unlike the case of models for the pull-out test, for modeling of the beam, a body of the concrete can not be neglected. If the cracks of the concrete body and debonding propagation of the interface from the cracks of the concrete are the focus, the element number will increase even more than the case of the pull-out test, resulting in a huge number of elements which are not in the practical range of the FE analysis. For example, 3D large scale T-beam model with damage material model might require at least 6000000 elements, with more than 400 hours CPU computational time. The University owned high performance computer has
2.6 GHz AMD processors and 32 GB of ECC ram. Alternatively, 2D models were considered since the concrete cracks and debonding propagation from the shear-flexural cracks with the presence of the Uwrap are the focus in this study.

### 4.3.1 Numerical modeling of pull-out specimens

Based on the previously explained developed models, a total of 18 different geometries were considered to model the pull-out test. These 18 different types of models cover all the experimental pull-out specimens. Furthermore, some of the important parameters which were observed in the experimental work such as the corner length, Uwrap orientation, Uwrap thickness, and Uwrap width were selected in order to conduct a parametric study. The list of the developed models is shown in Table 4-3. Boxed regions in Table 4-3 indicate the numerical models without experimental test results. Those are developed for the purpose of the parametric study. One of goals for performing the parametric study is to see whether the numerical results match well with the experimental trends with respect to the addition of different types of Uwrap. Therefore, developed models would be used for both verification purposes and the parametric study. Models were also designed to predict the entire load-displacement behaviors even after the debonding of the FRP sheet in order to see residual strength obtained from the Uwrap anchor effects.
For the parametric study, first, the Uwrap height was selected as a parameter. Five different heights from 25 mm to 127 mm were modeled, and results were compared. The second parameter was the corner length. A 25 mm and 64 mm corner length were modeled to see the effect of the corner length on the models. For the fiber angle of the Uwrap, both 30 degree and 45 degree Uwraps were modeled and compared to the 90 degree Uwrap. The Uwrap thickness from 1 layer (1mm) to 3 layers (3mm) was investigated. The width of the Uwrap is also investigated from 32 mm to 128 mm. For the width of the Uwrap, a three, three 21 mm, two, 32 mm and four 16 mm widths of Uwrap
were modeled in order to study the effect of the number of Uwrap in the same amount of material. The model list for this study is shown in Table 4-3.

First, a comparison with the experimental data will be presented for verification. Second, a parametric study of the selected parameters will be shown and discussed.

4.3.2 FEM verification of the experimental data

This section will show the comparison of the numerical results and experimental results for verification purposes. For verification purposes, load-displacement, slip along the length of FRP sheet, and the strain gage data obtained from experiments are compared to the results of the numerical analysis. The failure modes obtained from experiments and that from analytical models are also compared to each other. Furthermore, stress contours along the entire Uwrap region will be shown and compared with other types of Uwrap.

It was found that developed models can satisfactorily predict the maximum debonding load and residual strength after the debonding of the FRP sheet. However, the transition behavior between before, and after the debonding of the FRP sheet was conservatively predicted by the developed models, which is appropriate for design purposes. First, load-displacement graph at the loading end will be compared. Second, the slip profile along the length will be compared. Last, data obtained from strain gages will be compared with the numerical analysis.
4.3.2.1 FEM verification for the control specimens (No Uwrap)

In this section, the bond behavior between the concrete and FRP without Uwrap was verified with the experimental results (PU-C). First of all, it is an important step to verify the models when there is no Uwrap. This verifies the bond behavior between the FRP sheet and the concrete. After verification with specimens without the Uwrap, the models with the Uwrap will be compared with those of the experimental results.

From the control specimens (PU-C), the load-displacement graph at the loading ends and strain data along the FRP sheet were observed. The displacement was measured from the loading end as explained. For modeling the control specimens, the same modeling techniques, interface elements, and material properties were used. However, the Uwrap was not modeled in the control case since the PU-C specimen does not have Uwrap.

Figure 4-8 shows the comparison between the load displacement graph of the control specimens and that of the models. It can be seen that the load displacement graphs were well predicted. However, the maximum bond strength was overestimated by the models. A higher bond strength compared to the data resulting from the experiments was obtained. One explanation for this is that the condition of the distribution of the aggregate and the shape of the aggregate result in varied bond qualities between the real concrete and FRP sheet. This heterogeneous nature of the bond could be associated with the overestimation of the bond strength, while the models only considered the same bonding quality for the entire interface.
The strain gage data was also compared with the experimental data. The data was obtained only until 0.6 mm of loading end displacement near the debonding occurs. From the strain-displacement graph, it can be seen that the early period of strain was well predicted compared to the near debonding strains. The strain-force graphs were also compared with the experimental results. The overall trends were all well matched with the experimental results except for the maximum strain (maximum debonding load). It should be noted that the strain data was obtained from the specimen which showed the lowest maximum debonding load compared to the other two specimens. The models also overestimate the experimental results most of the time. The reason is the heterogeneous nature of the bond could be associated with the overestimation of the bond strength.

**Figure 4-8:** Comparison of control specimens with FEM results
while the nature of the modeling method considered only the same bond quality for the entire interface. This explains why the maximum load of the numerical analysis is higher than three experimental results. However, the highest maximum load (22.1 kN) obtained from the experiments is close to the result (23.0 kN) of the numerical analysis.

4.3.2.2 Load displacement at the loading end from pull-out specimens

In this section, the obtained results from the models with the Uwrap will be presented. First, load-displacement graphs were compared with experimental data for the pull-out specimens. The load-displacement graph is an important source of information for this study since it explains the delaying or arresting mechanisms of the debonding propagation between the concrete and FRP sheet. The anchor effect due to the presence of the Uwrap could be interpreted by using a load-displacement behavior. Furthermore, the load-displacement data is also related to the Mode 2 frictional bond-slip model of the interface between the concrete and the FRP sheet with the presence of the Uwrap. Figure 4-9 compares the obtained load-displacement graph of the numerical analysis with that (averaged LVDT data) of the pull-out tests.
Figure 4-9: Results of the load-displacement curves compared to the experimental data

Figure 4-9 predicts the behavior satisfactorily even after the debonding of the FRP sheet. For the PU-N specimen, the residual strength after the debonding between the concrete and the FRP is also well predicted. The final failure mode was also well
predicted as the Uwrap was ruptured at the corner due to the stress concentration, which was observed from the experiments.

For the PU-45 specimens, a stiffer load-displacement curve was obtained from the models compared to that from the experiments. As seen in results from the experiments, a rupture of the FRP sheet occurs due to the strong anchor effect of the 45 degree Uwrap. Accordingly, Uwrap failures such as a debonding or rupture at the corner were not obtained from this study. Therefore, the residual strength after the debonding of the FRP sheet could not be presented in the graph of the PU-45 specimens. This also indicates how the Uwrap orientation at an angle of 45 degree can be used to enhance the anchoring effect to the FRP sheet. The maximum load and failure mode were well predicted since the FRP sheet was ruptured near the load of experiments in the numerical analysis.

For the PU-CN specimens, the applied load of the PU-CN was dropped suddenly to a much lower load than that of the other specimens when the complete debonding of the FRP sheet occurred. This amount of load drop in load-displacement curves was also well predicted. In addition, the residual strength before and after the debonding of the FRP sheet was well predicted. Since the PU-CN specimen has a corner length (64 mm), the residual strength after the debonding of the FRP sheet was lower than that of other types of Uwrap. This phenomenon from the corner length was also well predicted. This result also supports that the developed frictional bond slip behavior of the interface at the locations of the Uwrap is the proper method for modeling the anchor effect of the Uwrap.

For the PU-ST specimen, the overall behavior was also well matched with the experimental results. Both experimental results and FE analyses show the earlier failure of the Uwrap compared that of PU-N specimens. The height of the Uwrap was smaller
than that of the PU-N specimens, resulting in earlier debonding compared to that of the PU-N specimens. A debonding of the Uwrap after the debonding of the FRP sheet was well predicted for the PU-ST specimens.

For the PU-T specimen, the overall load-displacement was also well predicted by the models. The effect of the increased thickness was well predicted with the developed models as the increased maximum load and residual strength were observed from the developed models. However, the failure mode was not the same. For the experiments, the FRP sheet was ruptured after a certain frictional behavior between debonded surfaces under the Uwrap. However, the models show the debonding of the Uwrap after a debonding of the FRP sheet. Therefore, the failure mode was not predicted in this case. However, based on the result from the numerical analysis, it was found that the FRP strain at complete debonding of the Uwrap was reached almost up to the failure strain of the FRP sheet (0.012) but before that. Therefore, the failure mode could be the rupture of the FRP sheet or the debonding of the Uwrap depending on the localized effect in the real specimens during the experiment. Therefore, it can be said that the models predict the overall behavior as well as the local behavior of the FRP sheet.

For the PU-MN specimens, the overall load-displacement was also well predicted by the models. However, the residual strength after the debonding of the FRP sheet was underestimated by the models (-9%). From experiments, the residual strength of the PU-MN specimen after the occurrence of the debonding of the FRP sheet was larger than that of the PU-N specimens (+7%). However, the FE analysis showed a similar level of residual strength compared to the PU-N specimens. The increments of the residual strength due to the number of the Uwrap were not brought into the results by the models.
since the frictional area for PU-N and PU-MN are the same; the residual strength after the debonding of the FRP sheet should be a similar value in the numerical analysis. However, the experimental results were not the same but increased as the number of Uwrap increases within the same amount of material. The maximum debonding strength as well as residual strength of the PU-MN was larger than that of PU-N. The two different results from the comparison lead to the conclusion that the frictional behavior could be more effective if a smaller width of Uwrap is used. In other words, intermittently used Uwraps with a small width rather than a single large width of Uwrap are more effective as an anchor. These effects could not be brought into the results from the numerical analysis in this study.

4.3.2.3 Comparison of slip profiles along the FRP sheet

As seen in the previous chapter, experimentally, a slip was measured for the pull-out specimens using the digital image correlation (DIC) technique. The slip profiles along the FRP sheet obtained from the numerical analysis were also compared with obtained data from the DIC measurement of the pull-out specimens.

Slip profiles were obtained just before the occurrence of the debonding between the concrete and FRP sheet. Figure 4-10 compares the experimentally obtained slip profiles by using the DIC technique with the numerical results of the developed models in one graph. The dots indicate the experimentally obtained data points. The lines indicate the obtained results from the numerical analysis. Ninety degrees in the figure indicates
the obtained results from all types of Uwraps at an angle of 90 degree. Accordingly, a 45 degree Uwrap (PU-45) and a corner Uwrap (PU-CN) were considered separately.

![Graph showing slip profiles](image)

a) Slip profile of PU-C and PU-N specimens at 23 kN

![Graph showing slip profiles](image)

b) Slip profile of PU-N specimen at a debonding load

Figure 4-10: Comparison of slip profiles between FEM and DIC measurement (PU-C and PU-N)

It is important to note that the slip profile from the models for the types of the Control (PU-C) and PU-N at the same load shows different profiles. Results from the PU-C model show a larger amount of slip through the length of the FRP sheet compared to that of the PU-N model. This indicates that in the models, the Uwraps were holding the FRP sheet with the frictional effect, resulting in a smaller slip profile compared to that of
the PU-C model. For the slip profile of the PU-N specimen at the debonding load, the overall trends were well predicted; however, the level of slip was somehow smaller than the experimental values. The obtained result from the DIC measurement is approximately 0.15 mm larger than the numerical results. This amount of difference is within the margin of error (0.2 mm) which can be generated by DIC techniques as previously explained.

Figure 4-11: Comparison of slip profiles between FEM and DIC measurement (PU-45 and PU-CN)
Figure 4-11 also compares both the DIC and numerical results. For the PU-45 and PU-CN specimens, slip profiles along the length of the FRP sheet were well predicted. The predicted line for the PU-45 specimens was well aligned with the experimental dotted points. Likewise, the predicted line for the PU-CN specimens was well aligned with the experimental dotted points.

It is interesting to note that the level of the slip profile of the Control (PU-C) specimen at the debonding load and that of the PU-CN specimen at the debonding load are similar to each other. This indicates that the debonding behavior between the PU-C and PU-CN specimens were similar. Accordingly, it can be concluded that the Uwrap with the long corner length could not be used as an anchor. This conclusion is also found and discussed from the experimental results in the previous chapter and verified in this chapter by the comparisons with the numerical analysis.

4.3.2.4 Comparison of the strain gage data with the experimental data

The strain gage data was also compared with the obtained result from the numerical analysis. Three strain gages were mounted along the length of the FRP sheet as seen in the Chapter 3. The specific locations of the strain gages can be found in Chapter 3. The obtained FEM results are compared with experimental data shown in Section 3.4.1.2. It should be noted that strain gages were mounted to only one specimen per each type of test group. Therefore, the obtained strain data from experiments does not
represent the average value of the specimens. Instead, it represents one test result out of 3 specimens.

**Figure 4-12:** Comparison of strain of FEM results with experimental data
Overall strain data well match with the experimental results for all specimens. However, after the debonding of the FRP sheet, the data show plateau strain compared to the experimental results. As seen in all the curves shown in Figure 4-12, the experimental strain values are gradually increased after the debonding of the FRP sheet, while the strain from the numerical analysis shows almost plateau strain. Especially for the PU-N, PU-T, and PU-MN specimens, the strain value from SG 1 is gradually increased after the occurrence of the debonding of the FRP sheet, while the models show almost constant value of the strain. It can be thought that the local frictional behavior was increased during the test as the generated stress from the Uwrap increases. This increased frictional behavior could be a reason why the strain from the SG1 increased. However, in the developed models, the frictional behavior was assumed to be constant as the maximum frictional stress was estimated based on the debonding strain of the Uwrap or stress concentration factors at the corner of the Uwrap. This is the reason why the strain from the numerical analysis shows almost constant values after the debonding of the FRP sheet.

Furthermore, the strain values also could have fluctuated due to some local effects such as some possible bending strain on the surface or local stress concentration effect due to the uneven debonded surface. However, overall trends of the strain data before and after the debonding of the FRP sheet were well predicted by the developed models.

As seen previously in the strain-displacement data and load-displacement data, the developed models are capable of predicting the behavior before the debonding of the FRP sheet and even after the debonding of the FRP sheet. An important factor in the modeling was the proposed frictional bond-slip model. For the beam analysis, this
frictional bond-slip model will be also used at the interface under the Uwrap locations. A model description for the beam analysis will be also shown in Section 4.3.4.

4.3.3 Stress contour of the Uwrap before the debonding of the FRP sheet

In this section, the stress contour of the Uwrap will be presented. This section will show the stress contour before the debonding of the FRP sheet. Section 3.4 shows the contour of the Uwrap after the debonding of FRP sheet.

The stress contour of the Uwrap could be an important factor in the design of the Uwrap. From the stress contour, some regions in the stress concentration could be identified and, subsequently, a cost effective design could be developed. Accordingly, the stress contour of the Uwrap was investigated by numerical analysis. It is interesting to note that based on the results of the performed FE analysis, the stress contour was significantly changed after the occurrences of the FRP sheet debonding.

![Stress contours of the Uwrap (PU-N)](image)

Figure 4-13: Stress contours of the Uwrap (PU-N)
Figure 4-13 shows the obtained results from the PU-N model. Stress contours for the fiber stress ($\sigma_1$), transverse stress ($\sigma_2$), and shear stress ($\sigma_{12}$) are presented. From the models, it can be seen that a stress concentration in the fiber direction takes place at the upper front area of the corner region. It is interesting that the stress is also transferred to the side of the Uwrap even before the debonding of the FRP sheet. The transverse stress contour in the weak axis of unidirectional Uwrap is also shown in Figure 4-13. Stress concentration for this case, in the transverse direction, takes place at the upper front area of the Uwrap. This contour of the transverse stress in the Uwrap is related to the strain profiles with the longitudinal FRP sheet since the FRP sheet and Uwrap are tied in the models. On the other hand, shear stress was high at the corner area of the Uwrap even before the debonding of the FRP sheet. These observations from the three different stress types show that the maximum shear stress, as well as the maximum tensile stress along the Uwrap, coalesces at a corner region of the Uwrap. Therefore, a corner failure is the most expected failure mode if the bonding capacity of the Uwrap is strong enough. This failure mode was observed from most of the specimens. If the bonding of the Uwrap is not strong enough, debonding occurs. The bonding of the Uwrap is sensitive to the effective bond length (e.g. Chen and Teng 2001), indicating that the height of the Uwrap will control the failure mode such as the rupture of the Uwrap or debonding of the Uwrap.

Figure 4-14 shows the results from the PU-45 model. Failure mode obtained from the PU-45 model is the rupture of the FRP sheet. Therefore, the contour after the debonding of the FRP sheet was not investigated. The stress in the fiber direction is high at the upper front area of the corner region. This is also observed from the PU-N model.
The transverse stress is high at the center front of the Uwrap as seen in Figure 4-13. Obtained stress contours in both the fiber and transverse direction show similar patterns with these of the PU-N. However, it was also observed that the shear stress in the 45 degree Uwrap (PU-45) is different than that in the 90 degree Uwrap (PU-N). From the results of the PU-N, a high shear stress was observed mostly at the front corner area. However, shear stress at the corner is not observed in the PU-45 model. Instead, at the upper front region of the Uwrap, the highest stress was observed. From these observations, it can be concluded that the shear stress is not much generated at the corner region of the 45 degree Uwrap (PU-45). Furthermore, the stress in the fiber direction is more widely distributed on the Uwrap (PU-45) compared to on the PU-N model. These observations lead to the conclusion that the 45 degree Uwrap (PU-45) can significantly reduce the shear stress at the corner, making more fiber in the Uwrap prevent (anchoring) the debonding of the FRP sheet. It is simply thought that Uwrap orientation of PU-45 is more tilted toward the loading direction; a greater anchor effect was obtained from the 45 degree Uwrap. These results are also observed from the experiments in that every PU-45 specimen failed by the rupture of FRP sheet while PU-N specimens failed by the rupture of the Uwrap corner or debonding of the Uwrap.
Figure 4-15 shows the obtained results from the PU-CN model before the debonding of the FRP sheet. The stress in the fiber direction is high at the front of the Uwrap. It is also important to note that the stress of the fiber in the area of the corner length is mostly activated (strained) before the debonding of the FRP sheet. The stress in the transverse direction is showing similar patterns with that of the PU-N model. As explained, strain profiles of this transverse stress are similar to that of the longitudinal FRP sheet since the FRP sheet and Uwrap are tied in these models.

The shear stress of the Uwrap was high at the side boundary of the FRP sheet rather than at the corner region as seen in Figure 4-15 before the debonding of the FRP sheet. This indicates that the shear stress is always developed around the side boundary of the FRP sheet. In the case of the PU-N, the side boundary of the FRP sheet is on the corner region of the concrete block due to the small corner length (5 mm), resulting in the highest shear stress at the corner region. This shear stress is transferred from the side boundary of the FRP sheet to the corner region as the FRP sheet is debonded. The
shifting of the shear stressed region will be explained in the next section and shown in Figure 4-21.

Results from the PU-ST specimen, are similar with that of the PU-N specimen as shown in Figure 4-16. The stress in the fiber direction was high at the front corner area. Both transverse and shear stress show almost the same pattern in PU-N specimens. From this observation, it can be concluded that the short height of the Uwrap (PU-ST) along the side of the concrete block works as an anchor as long as the debonding of the FRP sheet does not occur. After the debonding of the FRP sheet, however, the total displacement to keep the residual strength from the short Uwrap was relatively shorter than that of the long Uwrap (PU-N) since the height of the bonded area of the PU-ST (50 mm) is shorter than that (127 mm) of the PU-N specimens.
Figure 4-17 shows the results obtained from PU-T (Thickened Uwrap by two layers). Results were similar with the PU-N model. However, the stress level was diminished because the thickness of the Uwrap was twice as thick as the Uwrap of the PU-N model. The overall contours show the same pattern with the PU-N specimens before the debonding of the FRP sheet. Even though the stress concentration occurs at the corner of the Uwrap, as seen in the PU-N, the failure mode was different than that of the PU-N. The final failure mode was the debonding of the Uwrap rather than the rupture of the Uwrap since the thickened corner region of the Uwrap can withstand the stress concentration effect at the corners.
Figure 4-18 shows the results obtained from the numerical analysis for the PU-MN model. Interestingly, for PU-MN, the same stress contours with the PU-N model were obtained even though three Uwraps were intermittently used. Fiber stress is high at the front corner of the first Uwrap. The transverse stress is the highest at the front of the first Uwrap, and the level of the stress is gradually diminished as it reaches the back of the third Uwrap. The shear stress is high at the corner of the first Uwrap. From the observations, it can be concluded that an increasing number of Uwraps within the same amount of material does not change stress contour. However, the spacing between the Uwrap was not considered in this study. The small spacing between the Uwrap might not affect its stress contour. In this study, spacing between the Uwrap was 22 mm. Therefore, for a further study, spacing effects should be also considered in order to see the effect of the intermittently used Uwraps.
From the observations of all different types of the Uwrap, it can be concluded that the fiber stress at the front corner area is the highest in the Uwrap. The transverse stress at the front area is always high because of the anchor effect of the Uwrap. The shear stress was generated mostly at the corner region of the Uwrap, indicating that the corner region is in need of shear strengthening. The corner region could be strengthened with more layers of the different angles such as 45 degree. This strengthening will increase the shear capacity at the corner region of the Uwrap.

When the Uwrap is installed at 45 degrees, the shear stress concentration at the corner region was not observed, as seen in Figure 4-14. Furthermore, more fibers were carrying the tensile stress compared to that of the 90 degree Uwrap, indicating that the more load is taken by fibers, the less shear stress was generated. Therefore, it can be concluded that the use of 45 degree Uwrap can be a good design for the anchor effect. In addition, 45 degree Uwrap will prevent the propagation of the shear-flexural crack more
effectively compared to the 90 degree Uwrap because more fibers are tilted perpendicularly to the shear flexural cracks. This indicates that crack development due to the shear load could be arrested or delayed more effectively by the 45 degree Uwrap, resulting in better shear strengthening. Therefore, an appropriate angle of the Uwrap at 45 degree leads to a better anchor effect as well as shear strengthening of the beam. For the 90 degree Uwrap, crack development due to the shear load would be arrested by the shearing of the 90 degree Uwrap. In this case, fibers are not aligned perpendicularly to the shear flexural cracks but the angle between the fiber and flexural cracks are closer to the 45 degrees. Since the fiber in the Uwrap is weak at the shear stress, a 90 degree Uwrap would not be a good design choice for the shear strengthening. Therefore, it can be concluded that the best shear strengthening effect could be obtained from the 45 degree Uwrap.

Theoretically, the angle less than 45 degrees could contribute to a better anchor effect than the 45 degree Uwrap since more fibers are associated with the FRP sheet and aligned to the length of the FRP sheet. An angle less than 45 degrees of the Uwrap would not be a practically applicable geometry in the field. If the practical angle for the field application is considered, the final design recommendation is derived from 45 degree Uwrap, which might be the best geometry selection for both the anchoring effect and shear strengthening based on the results of this study.
4.3.4 The stress contour of the Uwrap after the debonding of the FRP sheet

In this section, the stress contour after the debonding of the FRP sheet is shown. Since anchor effects were still effective even after debonding of the FRP sheet, as seen from the experiments, it is worth examining the numerical results after the debonding of the FRP sheet and discussing the results. Stress contours in the fiber, transverse, and a shear stress contour were also observed.

Figure 4-19 shows the results from the PU-N model after the debonding of the FRP sheet. It can be seen that the corner region of the Uwrap was fractured. At the same time, stress in the fiber direction where no fracture occurred shows the highest value. Matrix failure occurs between the fiber and the fiber at the side of the Uwrap as seen in Figure 4-19. The transverse stress was high at the upper rear region of the Uwrap. This indicates that the matrix failure occurs around the upper front and upper center of the Uwrap except for the upper rear region of the Uwrap.

![Stress Contours](image)

Figure 4-19: Stress contours from the Uwrap, PU-N (Failure mode: Uwrap rupture)
The shear stressed zone after the debonding of the FRP sheet is shown in Figure 4-19. The shear stress was high at the end of the debonding processed zone and the remaining area along the split line (shear failure line) due to shear failure. This indicates that the shear stressed zone after the debonding of the FRP sheet occurs at the end of debonding processed zone and the remaining area along the split line (shear failure line).

Figure 4-20 shows the results from the PU-CN model after the debonding of the FRP sheet. The upper most region of the Uwrap stressed in the fiber direction. This fiber stress becomes diminished at the side of the Uwrap below the level of the corner. The transverse stress for the entire region of the Uwrap was relatively smaller than other types of stress. The matrix failed in tension after the debonding of the FRP sheet. Therefore, the matrix in the transverse direction could not carry the load, resulting in a small stress along the entire Uwrap. The level of the transverse stress is almost the same for the entire region after the debonding of the FRP sheet.

The shear stress was high at the corner area of the Uwrap. However, before the debonding of the FRP sheet, the shear stressed zone was high at the boundary of the FRP sheet rather than at the corner region. This indicates that a highly shear stressed zone was transferred from the boundary of the FRP sheet to the corner area as the FRP sheet is debonded. This phenomenon was also captured by the thermo-camera as seen in Appendix F.
Figure 4-21 shows the shift of the shear stressed region. The shear stressed region of the Uwrap was shifted from the boundary of the FRP sheet to the corner area. The Uwrap region, which overlapped with the boundary of the FRP sheet, was fractured after the shear stressed zone reached the corner.

From this PU-CN specimen, it can be also seen that the amount of slip is much larger than that of the non-corner specimen. This was already observed from the experiment. From the study, it could be concluded that the corner length allows a lot of slip and does reduce the confinement effects from the Uwrap.
Figure 4-21: Shear stress transfer of the PU-CN model

Figure 4-22 shows the results obtained from the PU-ST model. The stress level in the fiber direction after the debonding of the FRP sheet is quite even through the entire region of short Uwrap. Especially, the stress at the end of the debonding zone around the corner of the Uwrap shows higher stressed compared to other areas. The transverse stress also shows higher values at the upper rear area of the Uwrap than in any other area. A reason for this is that the rest of the upper region of the Uwrap failed due to the matrix failure. A highly shear stressed zone is also observed at the end of the debonding processed zone of the Uwrap. As this shear stressed zone was shifted to the end of Uwrap, debonding failure of the Uwrap was obtained as a final failure mode after the debonding of the FRP sheet. If the height of the Uwrap is sufficient (longer than the effective bond length), the corner region of the Uwrap could fracture due to stress concentration as seen from the PU-N. In this case, the debonding failure of the Uwrap was observed from both experiments and the numerical analysis, indicating that the numerical result predicts the failure mode as well.
Figure 4-23 shows the results from the PU-T model (2 layers). Similar patterns were observed with PU-ST. Since 2 layers were used, the rupture of the Uwrap due to the stress concentration at the corner did not occur. Instead, it failed by the debonding of the Uwrap. For the stress in the fiber direction, a similar level of stress for the entire upper region of the Uwrap was observed. This fiber stress gradually decrease at the side of the Uwrap below the level of the corner. The transverse stress was also high at the upper rear region of the Uwrap. From Figure 4-23, a debonded area and a sound area could be distinguished based on the shear stressed zone. Based on the results of the PU-T, it can be also concluded that the shear stressed region after the debonding of the FRP sheet occurs at the end of debonding zone.
A highly shear stressed zone is observed from the debonded end of the Uwrap. This shear stressed zone is gradually shifted toward the end of the Uwrap as the debonding propagation progressed from the corner to the end of the Uwrap.

Figure 4-24 shows the results from the PU-MN. After the debonding of the FRP sheet, fiber stress was high at the corner region as seen in other types of Uwrap, except for PU-45. The transverse stress was high at the upper area of the third (rear) Uwrap. The shear stress was high at the end of debonded zone of the Uwrap as seen previously. These were the most common stress contours after the debonding failure of the FRP sheet.
From the observation of various Uwraps after the debonding of the FRP sheet, it can be concluded that the highest tension stress in the fiber direction are at the corner areas of the Uwrap. The transverse stress is higher at the upper rear region of the Uwrap. This is because the matrix in the rest of the area of the upper region failed by tension stresses. The matrix is always weak in tension compared to the properties of the fiber.

The shear stress is always higher at the end of the debonded area compared to the other regions after the debonding of the FRP sheet. This highly shear stressed region is transferred from the corner region to the bottom end of the Uwrap as the debonding is propagated.

4.3.5 Parametric study on three dimensional (3D) pull-out models

From this section, some parametric studies will be presented. As explained previously, first some important parameters were identified, and each load-displacement
behavior due to the changes of parameters was then investigated. The selected parameters are the same as those in the experimental setup. However, the range for each parameter is extended beyond the tested range within practical sizes for the parametric study of the numerical analysis. Table 4-3 shows the developed models based on the selected parameters. As explained, it was indicated as a box if only analysis was done without experimental work. The specific dimensions are found in Table 4-3.

4.3.5.1 Parametric study: Uwrap length

First, the selected parameter Uwrap length (height of the Uwrap) was investigated from 25 mm (1 inch) to 102 mm (4 inch). Figure 4-25 shows the results from different Uwrap lengths. The load-displacement graph is stopped when Uwrap is completely debonded or ruptured. It is interesting to note that the Uwrap fracture at the corner region was obtained from the Uwrap length of 127 mm. When the length of Uwrap was less than that, it was failed by the debonding of the Uwrap. It was found that the stress concentrations at the corner region and the debonded strains were almost balanced when the Uwrap length was 102 mm. Therefore, if the length of the Uwrap (height of the Uwrap) is more than 102 mm, the Uwrap rupture at the corner region would occur as rather than the debonding of the Uwrap. It should be also noted that the rupture of the Uwrap at the corner region did not occur due to the increased frictional force but due to the damaged fiber with the stress concentration factor.
Another important observed behavior with respect to the length of the Uwrap was the frictional behavior after the debonding of the FRP sheet. The frictional strength after the debonding of the FRP sheet did not change but remained constant due to the nature of the frictional bond-slip model, as previously explained in Section 4.2, but more displacement was obtained as the length of the Uwrap increased. The frictional strength after the debonding is assumed to be constant.

It can be concluded that the Uwrap length could increase the maximum displacement and can also change the failure mode from the debonding of the Uwrap to the rupture of the Uwrap. Furthermore, rupture of the Uwrap at the corner region occurs because of fiber damaged due to the stress concentration.

Figure 4-25: Load displacement graph depending on different Uwrap lengths
4.3.5.2 Parametric study: Corner length

One of the selected parameters of the corner length was also investigated from the 0 mm (0 inch) to 64 mm (2.5 inch). Previously, it was found that the longer corner length is related to the larger amount of slip and minimizing anchoring effects. Figure 4-26 shows the results with respect to the different corner lengths.

Figure 4-26: Load displacement graph depending on different corner lengths

The load-displacement graph ends where the Uwrap debonded or fractured. From the results, it can be found that the maximum load would be decreased at the occurrence of debonding between the concrete and the FRP sheet significantly as the corner length increased. When there was no corner length, maximum load was increased up to 37 kN. However, the models which have 25 mm and 64 mm of the corner length did not increase the maximum strength as much as those with no corner length. The amount of load drop after the debonding of the FRP sheet was also much larger than that of the Uwrap with a
zero corner length (0 mm). This result is strongly associated with the frictional bond-slip model which was developed for this study. Since the Uwrap with the corner length could not generate the frictional behavior between the two debonded surfaces due to the fiber orientation (see the previous section), a larger amount of load was dropped after the debonding of the FRP sheet, and smaller maximum strength was obtained compared to the Uwrap without the corner length. Furthermore, FRP fiber was mostly failed by tension fracture before the frictional strength reached up to the maximum debonding strength of the Uwrap. Therefore, the failure mode for both models of corner length 64 and 25 mm was the Uwrap rupture at the corner area and at the boundary of the FRP sheet, respectively.

4.3.5.3 Parametric study: Number of Uwrap layers

One of the selected parameters, the number of Uwrap layers (thickness), was investigated from 1 layer (1 mm) to 3 layers (3 mm) in this section. It can be expected that the maximum debonding load and residual strength after the debonding of the FRP sheet would increase as the thickness of the Uwrap increased. Figure 4-27 shows the results with respect to various Uwrap thicknesses. For the 1 mm thickness of Uwrap, the corner was fractured as a failure mode as seen previously in the PU-N model. It is interesting to note that the three layers of Uwrap did allow a small amount of slip after the debonding of the FRP sheet. It is thought that the increased shear strength of the Uwrap at the corner region due to the larger thickness did not allow a large slip of the FRP sheet when the FRP sheet was debonded. A better anchor effect and earlier
debonding were obtained from the three layers of the Uwrap compared to that of the other two. In other words, smaller shear strain of the Uwrap at the corner region results in an early debonding at the side region of the Uwrap.

This phenomenon could be also seen in section 4.3.5.4 from the results of the 30 degree and the 45 degree models. Smaller shear strain values at the corner region could increase the anchor effect but reduce the maximum displacement. This phenomenon indicates that controlling the corner region with the thickness, angle and the corner length, will determine the global behavior of the Uwrap effect either, brittle-strong or ductile-weak. In other words, those three factors (thickness, angle and corner length) are the key parameters to control the Uwrap behavior.

Figure 4-27: Load displacement graph depending on different Uwrap thicknesses
### 4.3.5.4 Parametric study: The angle of Uwrap

One of the selected parameters, the angle of Uwrap, was investigated in order to observe the angular effect for the 30 degree, 45 degree, and 90 degree (PU-N) on load-displacement curves of the loading end. Figure 4-28 shows the results from the different angles of the Uwrap. The numerical analysis for the load-displacement graph was stopped when the FRP sheet was ruptured for the 30 degree and 45 degree models. It should be noted that if the thickness of the FRP sheet is increased up to 2 mm or 3 mm, failure mode will be the debonding of the Uwrap for the 30 degree and 45 degree models rather than the rupture of the FRP sheet. For the 90 degree case (PU-N), as previously seen, the FRP sheet was not ruptured before the debonding of the FRP sheet.

![Load displacement graph depending on different Uwrap orientation](image)

As expected, it was observed that the maximum debonding strength was increased as the angle of Uwrap is increased. Stiffer load-displacement graphs were obtained from the tilted angle of the Uwrap (30 degree and 45 degree) compared to those of the normal Uwrap (90 degree, PU-N). However, similar slopes were observed before the onset of the
debonding propagation no matter what the angle of the Uwrap was. It can be concluded that the tilted angle could increase the stiffness of the load-displacement graph after the onset of debonding (see Figure 4-28). In other words, the tilted Uwrap provides a better anchor effect compared to the 90 degree Uwrap.

4.3.5.5 Parametric study: the width of Uwrap

In this section, the effect of the Uwrap width on the load-displacement graph was investigated for the cases of 32 mm, 64 mm, and 128 mm. Figure 4-29 shows the results with respect to the various Uwrap widths.

![Figure 4-29: Load displacement graph depending on different Uwrap widths](image)

A model with 128 mm width failed by the rupture of the FRP sheet before the debonding of the FRP sheet. However, for comparison purposes, an analysis was continued until the Uwrap was debonded. The FRP sheet in the models was modified not to be ruptured. For the cases of 32 mm and 64 mm width of the Uwraps, the fracture of
the FRP sheet did not occur; instead, debonding of the Uwrap was observed from the analysis. As expected, an increase in the width of the Uwrap increases the debonding load of the FRP sheet and residual strength after the occurrence of the debonding of the FRP sheet. Since the increase in the width indicates the increased area under the frictional bond-slip model between the concrete and FRP sheet, increased width increases the debonding and residual strength. In other words, the larger frictional area with a wider width, is involved with the frictional behavior between the two debonded surfaces as the Uwrap is providing pressure on it.

However, the effectiveness could be decreased as the wider Uwrap is used. From experiments, it was observed that the PU-MN specimens showed better strengthening effects as well as a better frictional behavior compared to a single Uwrap when material quantities for both cases are the same. Accordingly, the effectiveness could be increased if the Uwrap width is small. This idea was also supported by Kamel et al. (2000). In their study, it was found that the FRP strain values are considerably larger at the edge than at the center of a sheet.

In the developed models, however, that effect of the width observed from the experiments could not be differentiated decisively as it was assumed that the entire area under the Uwrap locations have the same frictional bond-slip model.

4.3.5.6 Parametric study: the number of Uwrap within the same amount

In this section, the effect of the number of the Uwrap on the load-displacement graph was investigated. A total of the 2, 3, and 4 numbers of Uwraps were modeled, and
the results are compared in this section. The amount of material used was the same for all cases. Figure 4-30 shows the results obtained with respect to the different numbers of Uwrap. From experiments, specimens of the PU-MN showed a 15% larger residual strength than that of the PU-N specimen. In the numerical analysis, the number of Uwraps did not affect the overall load-displacement behavior as long as the amount of materials is the same. It is thought that the developed models are not sensitive to the number of Uwraps as long as the sum of the areas under the Uwraps is the same. It was assumed in the models that the entire area under the Uwrap locations have the same frictional bond-slip model. In the actual case, the frictional bond-slip model could be different depending on the localized stress in the Uwrap. It was found from current study that the edge area of the Uwrap is more stressed compared to the inner region (Kamel et al. 2000). For the experimental cases, it is thought that the frictional bond-slip model could be changed if the Uwrap width is changed. Based on previous experimental observations, a smaller width of the Uwrap (PU-MN) increased the maximum strength (the load at the complete debonding of the FRP sheet) as well as the residual strength since it is generating the frictional behavior effectively between the two debonded surfaces.

On the other hand, it was defined in the models that the entire area under the Uwrap locations has the same frictional bond-slip model. This nature of the modeling method results in non-sensitive results depending on the different numbers of the Uwrap as seen in Figure 4-30.
2.6 GHz AMD processors and 32 GB of ECC ram. Furthermore, a refined 3D large scale T-beam model with damage material model might require at least 6000000 elements, with more than 400 hours CPU computational time. The University owned high performance computer has 2.6 GHz AMD processors and 32 GB of ECC ram. Furthermore, a refined

4.4 Two dimensional (2D) modeling of the pull-out specimens

From the previously developed three dimensional (3D) pull-out models, two dimensional (2D) models were developed. Therefore, simplifications from 3D models to the 2D models were required. A 3D models, especially for the beam strengthened with all the Uwraps and the FRP sheet would not be an effective model since it requires a lot of elements to cover all interface between the concrete and FRP sheet as well as the interface between the concrete and Uwrap. As previously explained, 3D large scale T-beam model with damage material model might require at least 6000000 elements, with more than 400 hours CPU computational time. The University owned high performance computer has 2.6 GHz AMD processors and 32 GB of ECC ram. Furthermore, a refined
element is required for the concrete and interface elements if the cracked region and local behavior of the strengthened beam are the focus rather than the global behavior. In order to cover both FRP-concrete interface and Uwrap-concrete interface, it would require a lot of elements, resulting in a heavy, non-efficient model.

If the coarse element size is used in order to reduce the number of element, concrete shear-flexural cracks in a beam system could not be modeled due to the size of coarse element. As such, 3D modeling is not an efficient modeling approach if the small scale local behavior in the regions of the cracking, debonding and its propagation between the cracked regions are the focus. Therefore, developing an effective model of the pull-out test in 2D is an important step toward the model of the beam for this study. This section will show the developed 2D models and comparisons with the results of developed 3D models of pull-out specimens. Based on the developed method for the 2D models of the pull-out test, 2D models for the strengthened beam system were also developed. Specific details of the strengthened beam system will be shown in a Section 4.5.

Figure 4-31 shows a simplified pull-out model in 2D and a previously seen 3D pull-out models. First, simplification was taken that the portions of the Uwrap attached to both sides of beams have the same dimensions and will have the same behavior. Therefore, the thickness of each Uwrap in 2D has twice the thickness of the Uwrap used in the experiments; likewise second, the stiffness of the modeled interface becomes twice as large. In all the numerical models developed, it was also assumed that there is a perfect bond in the portion of the Uwrap attached to the longitudinal FRP sheet.
Figure 4-31: Simplification from a 3D model to a 2D model

Figure 4-32 shows detailed descriptions of the developed 2D model for the pull-out test. In the 2D models, the cohesive element was selected as an interface between the concrete and FRP sheet. It is only located between the concrete and FRP sheet. For the interface between the lateral side of the concrete block and the Uwrap, a cohesive element was not used in the 2D models since Mode 1 behavior between the lateral side of the concrete and Uwrap does not exist in the 2D models. Therefore, only Mode 2 bond-slip behavior was considered for the concrete-Uwrap connection.

For modeling in 2D, a plain strain element was adopted for the FRP sheet. The interface between the FRP sheet and the concrete was modeled by a 2D cohesive model which has two fracture energies in Mode 1 and Mode 2 directions. For the Uwrap, a plain stress element was used rather than a plain strain element since the Uwrap is relatively thinner than the width of the FRP sheet and the block of concrete. The node of the Uwrap element was connected to the fixed node by a connector. The connector is the link element which has the Mode 2 constitutive law between the concrete and the FRP. The
The proposed frictional bond-slip model was put into the cohesive element under the Uwrap region. Normal interface between the concrete and FRP, ordinary bond-slip model was put into the cohesive element. Stress concentration factor was considered since 2D models cannot generate the stress concentration at the corner area. The calculated stress concentration factors from Chapter 2 were used for the 2D model of the pull out tests.

Figure 4-32: Specific element types and connection in the 2D models
From the 3D models, small variations along the width of the FRP sheet and interface were observed. Therefore, plain strain would be a good simplification applied to the entire FRP and interface elements. Based on the explained methods, 2D models of the pull-out test were developed. The obtained results from the 2D will be compared with those of the 3D models. It will be also compared with the experimental data in the following sections. For the 2D analysis, 90 degree specimens (PU-N) and 45 degree specimen (PU-45) were modeled and compared.

4.4.1 Load displacement graph from the 2D models

In this section, the load displacement graph was compared with both results from the 3D models and the experiments. For the 2D models, the PU-N specimen and 45D specimens were specifically investigated since those two types of specimens would be used for the beam strengthening system in this study.

Figure 4-33 shows the comparisons of the load-displacement graph from the PU-N and PU-45 specimens. Results from the 2D models are compared with those of the 3D models. The overall trends are similar to each other. However, the 2D models seem to predict a slightly lower load with respect to displacement.
Figure 4-33: Comparison of obtained load-displacement graphs of 2D model with 3D models

Figure 4-34 shows the load-displacement graph from PU-N and PU-45 specimens compared to the experimental results. Basically, the 3D models were better at predicting the behavior of the PU-N specimen. In contrast, the 2D models were better at predicting the behavior of the 45 degree specimens (PU-45). However, it can be concluded that both the models are satisfactorily able to predict the load-displacement behavior and also show the strengthening effect and frictional effect due to the addition of the Uwrap. Therefore, it can be concluded that the simplifications taken from the 3D models for the 2D models are adequate steps in order to model the Uwrap effects on the debonding behavior of the FRP sheet.

Figure 4-34: Comparison of Load-displacement graphs of 2D models with experiments
4.4.2 Slip profile along the length of the FRP

From the 3D models and experiments, slip profiles and strain values were also obtained from the specified locations. Accordingly, slip profiles along the length of the FRP sheet obtained from the 2D models were also compared with both the 3D models and the experimental results for the PU-N and PU-45 specimens.

Figure 4-35 shows the results obtained from the 3D and 2D models. From the Uwrap in the PU-N, the same slip profiles were obtained mostly for the entire period. For the 45 degree Uwrap (PU-45), the 2D model is showing a slightly larger slip profile compared to that of the 3D. It is thought that a portion of the 45 degree Uwrap in the 3D model which is attached to the FRP sheet can be modeled with the detailed Uwrap geometry. However, in the 2D model, the detailed geometry along the width could not be modeled since the width of the block is a non-existing axis in the 2D model. Some simplification steps had to be taken for the 45 degree Uwrap of the 2D model. Due to these slight geometric changes, the results of the 45 degree Uwrap from the 2D and 3D were obtained differently. However, it can be concluded that the overall behaviors obtained from both the 2D and 3D models are the same.
Figure 4-35: Slip profiles of the 2D model along the length of FRP compared with the 3D model

Figure 4-36 shows comparisons of the 2D model with the experimental results. Improved slip profiles were obtained from the 2D model for the 45 degree Uwrap. For the 3D model, slip profiles were slightly smaller than the experimental results obtained from the DIC technique.

From the comparison with slip profiles, the 2D models did not show any discrepant results compared to the 3D models and experimental results. These results
support the fact that the simplifications taken from the 3D models and numerical methods of the 2D models are appropriate for this study. Using the developed method for the 2D mode, the larger scale T-beam strengthened with the FRP sheet and the Uwrap will be modeled, and experimental results are compared with the results of the 2D models. The results of the T-beam will be shown in a section 4.5.

4.4.3 Comparison of strain data along the FRP sheet

Strain data along the length of the FRP sheet of the 2D models was also compared with that of both the 3D models and the experimental results. Figure 4-37 shows the comparison between the 2D and 3D models. The overall shapes for the three strain gage data are the same. However, as previously seen, for the 45 degree Uwrap, the obtained results from both the 2D and 3D models show a different behavior by small magnitude. As previously explained, the incapability of the 2D models to take into account the specific geometry along the non-existing axis in the 2D models is thought to be the reason for variations between the 2D and 3D models. However, the overall trends are acceptable, and both models satisfactorily predict the behaviors.
Figure 4-37: Comparison of strains of 2D model with that of the 3D models

Figure 4-38: Comparison of strain of the 2D model with experimental data
4.5 Modeling of the large scale T-beam in 2D

The large scale T-beam is modeled based on the 2D modeling methods which were previously shown and discussed. Two tested T-beams for the preliminary study and one tested T beam for the anchoring effects were modeled. First, two tested beams for the preliminary test were modeled without considering the frictional bond-slip model between the debonded surfaces since the final failure of both beams was the fracture of the FRP sheet before the debonding. Furthermore, the Uwraps were not activated much. In this case, the frictional bond slip behavior is not required.

The last beam was designed for the anchor effect of the Uwrap. Therefore, more layers of the FRP sheet strengthen the soffit of the concrete in order to prevent the fracture of the FRP sheet before the debonding. Accordingly, the frictional bond-slip model after debonding was considered.

In this section, first, detailed modeling specifications for the first two beams (preliminary study: CFRP beam with normal Uwraps and GFRP beam with normal Uwraps) and last beam(CFRP beam with 45 degree Uwraps) with a frictional effect will be explained. Second, the results from two tested T-beams for the preliminary study and the one tested T beam for the anchor effects will be presented. At the same time, results from the numerical analysis will be compared with the experiments for verification. Finally, using the developed T-beam models, a parametric study of the angle of the Uwrap and the frictional behavior on the load-displacement behavior was conducted. Discussions about the presence of a frictional factor between the debonded surfaces will be also presented.
4.5.1 Developed modeling method for the T-beam without friction effect

This section will present the modeling technique for the T-beam in 2 dimensions. The same simplification steps were taken from the 2D models of the pull-out test. The first simplification was that the portions of the Uwrap attached to both sides of beams have the same dimensions and will have the same behavior. Therefore, the thickness of each U-wrap has twice the thickness of the Uwrap FRP sheet used in the experiments; likewise, the stiffness of the modeled interface becomes twice as large. It was also assumed that there is a perfect bond between the Uwrap and the longitudinal FRP sheet. In the previous 2D pull-out models, the concrete was not modeled since the failure was only obtained from the interface and FRP (Uwrap). However, concrete elements and rebar elements have to be taken into account for the models of the beam. For the concrete elements, a damage plasticity model was used. Theoretical background information is found in Chapter 2. Plain strain elements were used for the concrete. The node of the Uwrap was connected to the node of the concrete element by the connector element as seen in the 2D models of the pull-out test. For interface between the concrete and FRP, two different modeling techniques were developed. The fist model uses the damage band developed by Coronado (2006), which is based on the continuum mechanics of the concrete. Accordingly, the cohesive element was not used for this case. It follows the damage plasticity model. Between the concrete and FRP, epoxy was also modeled. Element size used for the concrete was 25.4 mm by 25.4 mm. Element size used for the epoxy was 1 mm by 1 mm. Calculated stress concentration factor from Chapter 2 were used for the 2D model of the preliminary large scale T-beam tests.
Figure 4-39 shows the sketch of the damage band method in detail. The constitutive law for the epoxy follows the elastic-plastic behavior. The damage band (DB) was modeled in the first layer of the concrete element which faces the epoxy. A fracture energy obtained from the concrete-epoxy interface (CEI) was put into this damage band. Therefore, the fracture energy and the onset of tensile failure of the CEI is the key parameter to determine the debonding behavior between the concrete and FRP in this model.

The interface (connector) element links the adjacent concrete element and Uwrap element, as shown in Figure 4-39. The behavior of the interface elements are governed by a Mode 2 bond-slip behavior. This interface element intends to replicate an interface where the in-plane debonding process is solely controlled by a Mode 2 fracture.
Figure 4-40: Cohesive element based FE model (Cohesive Model)

Figure 4-40 shows the cohesive model. The cohesive models do not use a damage band for the concrete epoxy interface; instead, they use a cohesive element as seen in the previous sections. This cohesive element is governed by both Mode 1 and Mode 2 behaviors. For these models, epoxy was not modeled, instead, cohesive elements were placed between the concrete and FRP. For the Uwrap and interface between the Uwrap and the lateral side of the concrete, the same models were used as in the damage band model. However, the cohesive model cannot predict the cracked region. Accordingly, the cracked region between the concrete elements was determined based on results from the damage band models.

Table 4-4 shows the summary of the two developed models. The number of elements, the element type and the material constitutive law are summarized in this table.
Unlike the two previous T beams, a tested T-beam was specially designed for the anchor effects. Three layers of FRP sheet are used in order to see the appropriate anchor effects after the debonding of the FRP sheet. Element size used for the concrete was 25.4 mm by 25.4 mm. Element size used for the interface between the concrete and the FRP sheet was 1 mm by 1 mm. Calculated stress concentration factor from Chapter 2 were used for the 2D model of the large scale T-beam test with 45 degree Uwraps.
Test results as seen in Chapter 3, showed a desirable anchor effect from the developed Uwrap design. This indicates that this beam should be modeled after considering the frictional behavior between the concrete and FRP. The two previous beams were well predicted even without considering the frictional effects since the FRP was fractured much earlier than the activation of the Uwrap and debonding of the FRP sheet. Accordingly, the frictional bond-slip model was developed according to the designed geometry of the Uwrap. The developed frictional bond-slip model was put into the cohesive element as properties in Mode 2 behavior. Figure 4-41 describes the models developed for the analysis of the beam. The same modeling technique is used as seen in the previous models for the two T-beams except the Mode 2 frictional bond-slip model in cohesive elements. Orthogonal properties of the Uwrap were also considered.
4.5.3 Results from the preliminary T-beams

Results from the two T-beams will be presented in this section. Some mesh sensitivity work was performed for the models. Load-displacement graph, strain distribution along the length of the FRP, and local strain data from some of the Uwrap will be compared with the experimental data, and verification works will be presented.

4.5.3.1 Mesh sensitivity

Different mesh configurations were used to investigate if the results obtained from the numerical models are mesh and element size independent. Figure 4-42 shows five variations on mesh configuration for different material elements. Load-displacement curves were obtained from the analysis of the CFRP strengthened beam “continuum model” with different mesh densities. The aspect ratio of the concrete element is one except for Mesh 2 and Mesh 4. Mesh 2 and Mesh 4 use elements twice as height than wide, indicating that the aspect ratio is 2. Results, shown in Figure 4-43, indicate that all 5 mesh variations had similar behavior. All five numerical models were also able to predict FRP rupture failure.
In Abaqus, the characteristic length for planar element is calculated as the square root of the integration point area (e.g. Hibbitt et al. 2007). Accordingly, elements with large aspect ratios like Mesh 2 and 4 showed different behaviors due to the larger aspect ratio compared to one. Therefore, elements that have aspect ratios close to one are recommended. Overall behaviors from Mesh 1, Mesh 3, and Mesh 5 are well converged at the point of the FRP rupture failure.

Figure 4-42: Mesh configurations used to evaluate mesh sensitivity
Among the meshes, Mesh 3 was selected as a mesh configuration to be used in all numerical models for this study. The element size of the concrete will be in the order of two times the maximum aggregate size (13 mm) of the concrete. The cracking and debonding processes along the FRP sheet and the Uwrap could be captured from Mesh 3 since the characteristic length (length scales) is in the order of two to three aggregate sizes (e.g. Bažant 1986, Coronado and Lopez 2006).

Even though results from Mesh 5 were in the converged region, no further mesh refinement was deemed necessary. Mesh 1 was not used since it was considered to be too coarse to show the local damage behavior even though the global prediction is acceptable, as shown in Figure 4-43. For concrete and interface softening behavior, results would be unobjective and would be dependent on mesh size if stress-strain softening behavior is used. Therefore, displacement softening behaviors have to be used for all softening models rather than strain softening behaviors. This spurious mesh

![Figure 4-43: Mesh size sensitivity](image)
sensitivity in the concrete softening model is well described in published papers (e.g. Hillerborg et al. 1976 and Bazant and Planas 1998).

4.5.3.2 Comparison of Load-displacement graph (T beams without friction effect)

In this section, numerical results of the first two T-beams which were tested as a preliminary study will be shown. Obtained analytical results were compared with those of the experimental load-displacement graphs. The two different numerical models show a good prediction of the overall load-midspan deflection response for both FRP strengthened beams. The continuum models, however, overestimated the deflection of the beam at the FRP rupture by 15%.

Figure 4-44: Comparison of load displacement graphs (Cohesive model)

This deficiency in estimating deflections at the ultimate level of the load could be an indication of how these continuum models over predicted the extent of the damage in the concrete beams. The cohesive models show improved displacement prediction (2 % difference) due to use of mixed Mode interfacial elements as well as a refinement in the
concrete crack locations. Results from this Model will be used in Figure 4-49 for comparison with the experimental results. Load results from two numerical models are summarized in Table 4-5. It can be observed that yielding and failure loads were predicted within a 12% margin of error. The cohesive models show a smaller margin of error (+/- 2 to 7%), indicating that it has better capability of estimating the ultimate flexural capacity. Therefore, it can be concluded that the interfacial mixed fracture mode behavior, along with the refinement on crack locations which was used only in cohesive models, is a good modeling approach to predict the global response of FRP strengthened beams.

<table>
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<tr>
<th>Table 4-5: Yielding and ultimate loads of the FRP strengthened beams</th>
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<td>Prediction/Experiment ratio</td>
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<td>Exp (kN)</td>
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4.5.3.3 Comparison of strain profile along the length of the FRP (T beams without friction effect)

Thirteen strain gages were installed on the longitudinal CFRP sheet to evaluate the strain distribution at different loading levels. For the GFRP strengthened beam, a total of 5 strain gages were installed. Figure 4-45 compares experimental and numerical strain profiles along the longitudinal axis of the FRP; experimental values are indicated by
markers. For both strengthened beams, strain distributions are compared at three different levels of applied load: 72 kN, 145 kN, 190 kN for the CFRP beam, 74 kN, 147 kN, 160 kN for the GFRP beam. The selected load levels were chosen to capture the behavior before, near, and after the yielding of the steel rebar.

Figure 4-45: Strain profile along the FRP sheet for both beams
Results show that the strain distributions along the longitudinal FRP sheet are well predicted at every load level for both the FRP strengthened beams. The continuum models and cohesive models showed very similar results; however, it is interesting to note that more strain “fluctuations” are observed in the cohesive models. Figure 4-45 shows a comparison between the experimental and predicted strain values.

The strain fluctuations from the longitudinal FRP sheet appear in the numerical models after the first concrete cracks (cracking load) are predicted. Figure 4-46 shows strain profile before (11 kN) and after (38 KN) the concrete cracks at mid-span.

![Strain profile before cracking load](image1)

![Strain profile after cracking load](image2)

Figure 4-46: Strain profile before and after cracking load of carbon strengthened beam

It can be concluded that stress concentrations appear in the concrete elements after the cracking load, and are reflected on the FRP strain distributions. As indicated in the previous section, continuum models have epoxy layers between the concrete and the FRP, relieving these stress concentrations and redistributing them to the FRP elements. The cohesive models do not have an epoxy layer; instead, it has mixed-mode (Mode 1 and Mode 2) interface elements between the concrete and FRP sheet.
Furthermore, the element size of the concrete-FRP interface in the cohesive model is much smaller than that in continuum models, resulting in a more refined strain profile along the FRP sheet. Therefore, it can be concluded that the cohesive model has the added capability of capturing the effect of stress concentrations around cracked locations. For the CFRP strengthened beam, it was observed that after debonding occurred between the concrete and the longitudinal FRP sheet (at midspan), the strain profile becomes smoother; see strain at 190 kN in Figure 4-45. This “release effect” was not found in the GFRP beam at 160 kN in Figure 4-45, thus indicating that debonding around the concrete cracks has not yet developed at this load level.

4.5.3.4 Behavior of the Uwrap

Strain data from the bonded Uwraps were obtained experimentally and compared with the numerical results. Strain gage locations on the Uwrap for both strengthened beams are shown in Figure 3-4. For the GRFP strengthened beam, strain gages at different heights were attached on selected FRP Uwraps. Results indicated that the two developed models can predict the strain trend of the CFRP Uwrap behavior. Both models show satisfactory results, very likely due to the fact that they do have interface elements. The experimental test of the CFRP strengthened beam did not show Uwrap debonding failures. The CFRP Uwraps were firmly attached even after rupture failure of the longitudinal CFRP sheet.
Figure 4-47: Local strain behavior of CFRP beam

Figure 4-47 shows the comparison of the strain gages 1 and 2 between experimental and numerical strain profiles of the CFRP beam. Experimentally obtained strain levels were below 1000 με (0.1%); strain predictions of the two developed models were also in that range, indicating that the models follow the behavior of the T beam as well as the behavior of the individual Uwrap.
As shown in Figure 4-48, for the GFRP strengthened beam, experimental and numerical strain levels were in the range of 1000 με for SG 3 and in the range of 100 με (0.01 %) for SG 4. It was found that the propagations of the shear-flexural cracks under the Uwraps affect the FRP strain readings. Higher strain values are found in the vicinity of concrete cracks since the numerical data shows this behavior.

4.5.3.5 Cracked pattern of the beams

Stress contours from the continuum models are used to show the effects of the crack location on the level of “activation” of each Uwrap. Figure 4-49 shows the location of the concrete shear-flexural cracks and principal stress contours in FRP Uwraps for
both FRP strengthened beams. These results indicated that the stresses transferred from the longitudinal FRP sheet as well as the location of concrete shear-flexural cracks can significantly change the FRP Uwrap stress contours.

Figure 4-49: Cracked patterns (a) and stress contour of the Uwrap (b)

For both FRP strengthened beams, higher stress values were obtained at locations where shear-flexural cracks form, in particular near the bottom of the longitudinal CFRP sheet. It is interesting to note that for the CFRP strengthened beam, Uwrap 2, near the point load, had the most activation due to the presence of flexure cracks and a larger pulling load from the extended longitudinal FRP sheet. For the GFRP strengthened beam, Uwraps 7 to 9, located in the constant moment region, were activated at the bottom due to the pulling loads from the extended longitudinal FRP sheet. Uwraps 1 to 5, located
in the shear span, have activated areas that match the crack patterns originated by flexural-shear cracks. Uwraps 4 to 6, near the point loads, show a combination of activation at the two locations.

4.5.4 Results from the T-beam with anchor effect

Results from the T-beam with anchor effect will be presented in this section. Load-displacement graphs, strain distributions along the length of the FRP and local strain data from some of the Uwraps will be compared with the experimental data for the verification works. Furthermore, parametric studies of the number of Uwraps and the angle of the Uwraps on the load-displacement graph will be investigated. Conclusions from this study would be presented.

4.5.4.1 Comparison with load-displacement graph

Firstly, obtained load-displacement graphs from the developed models were compared with previously shown experimental data. Experimentally obtained data from the string potentiometer, which was located at mid-span, and averaged data from both load cells were used to draw the load-displacement graph of the experiment. Figure 4-50 shows both FE results and experimental data. Overall trends are satisfactorily predicted. The actual yielding point of the steel rebar seems to be lower than values used for the
models. The yielding load from FE analysis was 150 kN while experimentally it was obtained at 123 kN. Maximum loads obtained from the experiment and FE analysis were 314 kN and 292 kN, respectively.

![Failure modes and failure load calculated based on the ACI440-08.](image)

Figure 4-50: Comparison of load-displacement curves of the model with experimental results

A failure mode and a failure load were also calculated based on the ACI440-08. The failure mode was found to be the debonding of the FRP sheet. Typical test values were used for the estimating debonding of the FRP. For calculating the load of debonding failure, environmental reduction factors were not used. The calculated maximum failure load due to the debonding of the FRP sheet was 199 kN. It was found that the equation estimating debonding failure from the ACI is too conservative if there are Uwraps. As previously seen in results of the pull-out test, the debonding strain was increased with the presence of the Uwrap. Calculated debonding strain using an ACI proposed equation was 0.0056. However, a strain at mid-span obtained from the installed strain gages reached up to the rupture strain of the FRP sheet (0.012), which is twice as large as that from the debonding strain calculated from the ACI equation. It can be thought that the delaying of the debonding is a result of the anchoring effect of the Uwraps. Furthermore, it shows that the ACI equation for the debonding of the FRP sheet does not consider the anchor
effect on the debonding mechanism, which could be obtained from the use of the Uwrap. For further study, developing new equations in the case of the Uwrap is recommended. In order to develop a new debonding equation, many different types of Uwrap (anchor) systems and experimental works are needed and, at the same time, numerical works with the developed methods should be verified.

**4.5.4.2 Comparison with strain profile along the length of FRP**

Seven strain gages were mounted on the longitudinal CFRP sheet to evaluate the strain distribution at different loading levels. Specific locations of the strain gages could be found from the previous Chapter 3 (Figure 3-57). Figure 4-51 shows experimental and numerical strain values along the longitudinal axis of the FRP. Experimental values are indicated by markers.

![Strain profiles along the FRP sheet](image)

Figure 4-51: Strain profiles along the FRP sheet
Strain profiles were obtained at the load level of 89 kN, 178 kN, and 267 kN from the installed strain gages along the length of the FRP. The selected load levels were chosen to capture the strain profiles before and after the steel yielded in tension and before the rupture of the FRP.

Results show that the strain distributions along the longitudinal FRP sheet are well predicted at every load level. It is also interesting to note that strain at non-Uwrap locations shows a plateau, while the strain at the Uwrap locations shows variations on the length of FRP sheet. It is thought that since the area between the Uwrap are already debonded, unbonded behavior between adjacent Uwraps were shown as a plateau in the graph. From a comparison with the experimental data, it can be concluded that the behavior of the FRP sheet at every load level was well predicted with the developed modeling techniques.

4.5.4.3 Local behavior of the Uwrap

Experimentally obtained strain from the Uwraps during the flexural behavior of the strengthened T-beam was compared with results from the developed numerical analysis. Specific locations of the strain gages in the Uwrap could be found from the Figure 3-57. Results indicated that the developed models can predict the strain trend of the Uwrap behavior.

Experimentally, it was observed that the debonding propagated from the mid-span to the anchor zone, and one of the Uwraps was debonded before the FRP was failed by the rupture. This indicates that the transferred load from the strained FRP sheet was
beyond the limitation of the anchor capacity of the Uwrap. The magnitudes of strain in the Uwrap obtained from this test is much larger than in the previous two T-beams, indicating that strain values are more comparable than in the previous case which show a smaller level of strain.

Figure 4-52: Comparison of local strain data from FE analysis with experimental data
Figure 4-52 shows the comparison of the strain data. Experimentally obtained strains from the Uwrap were averaged from both sides to be compared with the results of axis symmetric numerical models. From the experimental observations, Uwrap 4 through Uwrap 7 was activated. It should be noted that Uwrap 6 was debonded due to the strained FRP sheet. Uwraps near the supports showed a small level of strain, which indicates that the Uwrap at those locations are not much activated as an anchor. Uwraps (U5 and U6) near the loading point show a five times larger strain than that of the Uwraps (U1 and U10) near the supports. These trends are well captured by the developed models. Levels of the strain at each different Uwrap are well predicted. The two Uwraps (U1 and U10) which are close to the supports were not accurately predicted by the models. However, overall trends are well predicted. It was observed from the test that Uwrap 6 was completely debonded, and Uwrap 5 was about to be debonded. The rests of the Uwraps were not completely debonded before the rupture of the FRP. On the other hand, in the numerical analysis, the failure mode was the debonding of all Uwraps before the rupture of the FRP. When Uwraps were debonded in the numerical analysis, the strain of the FRP sheet at mid-span was 0.0115. However, a rupture strain in numerical analysis was set to 0.012. This indicates that the test condition was close to the balanced condition between the debonding of the Uwrap and rupture of the FRP sheet. Accordingly, in the numerical models, all Uwraps were debonded before the rupture of the FRP sheet. Experimentally, some Uwraps were debonded, and then the partial rupture of the FRP sheet was observed due to the local stress concentration effect along the FRP sheet before all the Uwrap were debonded.
4.5.4.4 A parametric study of beams with Uwraps

A parametric study was performed to see the effect of Uwrap and developed frictional bond-slip model on the overall load-displacement graph using the developed models. One was modeled without the Uwrap. Therefore, no frictional bond-slip model was put into the cohesive element. The second was modeled with the Uwrap. However, the frictional bond-slip model was not considered for this case. These two developed models are compared with previously obtained numerical results which include the Uwrap and frictional bond-slip model between the concrete and FRP under the Uwrap locations. This parametric study was performed to see the effects of the friction and the Uwrap on the load-displacement curves. Figure 4-53 shows the load-displacement graph from those three models.

![Load-Displacement Graph](image)

Figure 4-53: Parametric study on load-displacement graphs depending on existence of Uwraps and frictional bond-slip model
As expected, without Uwrap, the FRP sheet was debonded much earlier than the models with Uwrap. These results show how the Uwrap in the numerical models are effectively arresting and delaying the debonding propagations. The debonding strain of the model without a Uwrap was 0.006 which is close to the calculated debonding strain (0.0056) based on the equation of ACI 440-08.

The debonding strain was increased almost up to the rupture strain (0.012) when Uwraps were only considered without the frictional bond-slip model. Accordingly, additional flexural strengthening effects (33%) were obtained from the addition of the Uwraps. When the frictional behavior was considered, the model shows more ductile behavior compared to the model with Uwrap and no friction. The maximum applied load was not much increased from the frictional effect. However, the final failure takes place at a larger mid-span displacement. This indicates that the frictional effects with several numbers of the Uwraps could delay the debonding propagation, resulting in more ductile behavior. The final failure mode was the debonding of the Uwrap after the debonding of the FRP sheet. This failure mode is the ideal failure mode for externally bonded FRP system since the rupture of the FRP sheet and rupture of the Uwrap before debonding of the Uwrap is expected to be a catastrophic failure compared to the debonding of the Uwrap after the debonding of the FRP sheet. Even after the debonding of FRP sheet, the Uwrap is holding the FRP sheet, reducing the brittleness of the failure, resulting in a better ductile behavior. This ductile behavior could prevent one of the major disadvantages of the externally bonded FRP system, brittle failure. This indicates that the safety issue and the conservative nature of FRP design code regarding the externally bonded FRP system (ACI 440-08) could deal with the Uwrap design.
Another parametric study was also performed in order to see the effects the numbers of Uwraps and angle of Uwrap on the load-displacement graphs. A total of 6 different cases were modeled and compared to each other depending on the angle of the Uwrap and the FRP sheet. Details about the 6 different cases are summarized in Table 4-6.

Table 4-6: Developed model description for the parametric study

<table>
<thead>
<tr>
<th>Name</th>
<th>Angle of Uwrap</th>
<th>Number of Uwrap</th>
<th>Layers of FRP sheet</th>
<th>Frictional behavior</th>
</tr>
</thead>
<tbody>
<tr>
<td>No Uwrap</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>1 Uwrap_45</td>
<td>45</td>
<td>1</td>
<td>3</td>
<td>Yes</td>
</tr>
<tr>
<td>3 Uwrap_45</td>
<td>45</td>
<td>3</td>
<td>3</td>
<td>Yes</td>
</tr>
<tr>
<td>5 Uwrap_45</td>
<td>45</td>
<td>5</td>
<td>3</td>
<td>Yes</td>
</tr>
<tr>
<td>1 Uwrap</td>
<td>90</td>
<td>1</td>
<td>3</td>
<td>Yes</td>
</tr>
<tr>
<td>3 Uwrap</td>
<td>90</td>
<td>3</td>
<td>3</td>
<td>Yes</td>
</tr>
<tr>
<td>5 Uwrap</td>
<td>90</td>
<td>5</td>
<td>3</td>
<td>Yes</td>
</tr>
</tbody>
</table>

*NA: not applicable

Specific locations of the Uwrap are shown in Figure 4-54. The same moment span and shear span were used for this parametric study. The distance between adjacent Uwraps are the same and are symmetric about the neutral axis. The specific geometry of the 45 degree Uwrap and distances between Uwraps can be estimated based on the Figure 3-56.
Figure 4-55 shows the obtained results from 45 degree Uwraps. First of all, it shows that an increasing number of Uwraps can delay the debonding propagation of the FRP sheet. However, the models for No-Uwrap and U-1-45 show a similar behavior. This result indicates that if the anchor capacity of the Uwrap is not enough, the Uwrap could not carry the load from the released strain energy generated by the debonding from the FRP sheet. This released energy generated from the debonding of the FRP could be arrested as more anchors (Uwraps) are intermittently installed. That is the reason why U-
1-45 shows the same behavior as the No-Uwrap while U-3-45 and U-5-45 show the increment of the maximum loads and maximum displacements compared to the No Uwrap. Furthermore, the ductile behavior was obtained from both U-3-45 and U-5-45 as the debonding propagation is delayed due to the anchor effect of the Uwraps.

Likewise, Figure 4-55 shows the obtained results from the parametric study on the 90 degree specimen with a different number of Uwraps. It is also found that one number of Uwrap (U-1-90) is not sufficient to carry the released strain energy due to the debonding of the FRP sheet. At the same time, U-3-90 and U-5-90 show an increase in maximum load as well as a delay in the deboning propagation of the FRP sheet.
It should be also noted that the 90 degree Uwrap could allow more slip than the 45 degree Uwrap since 90 degree Uwrap strained in shear whereas 45 degree Uwrap did not. Shear strained 90 degree Uwrap could allow larger displacement compared to the tensile strained 45 degree Uwrap. Therefore, 90 degree Uwrap allows for a much larger displacement. However, the strengthening effects were smaller than that of the 45 degree Uwrap. For the 45 degree Uwrap, tensile strain in the fiber is more active rather than shear strain, and so anchoring effects could be maximized. Therefore, the strengthening effect obtained from the U-3-90 and U-5-90 was less than that from the U-3-45 and U-5-45. However, the ductility of U-3-90 was increased more than that of the U-3-45 as the increase in the maximum displacement was observed.

Based on the performed parametric study, some important conclusions can be derived. First of all, there is a threshold value for the number of Uwrap. As seen in this
study, one anchor was not sufficient to carry the sudden release of the strain from the FRP sheet. For this study, three Uwraps could effectively arrest or delay the debonding propagation of the FRP sheet. Therefore, three could be the threshold number for the Uwrap anchor effect. Secondly, the number of Uwraps installed intermittently was effectively delaying the debonding propagation from the mid-span to the supports. More studies are needed to optimize the design. Thirdly, better ductile behavior could be obtained from the 90 degree Uwrap. However, for maximizing strengthening effects, a 45 degree Uwrap can be the better choice.

4.5.5 Principal findings from the numerical study

In this section, a proposed frictional bond-slip model was introduced. Numerical models were developed to evaluate the effect of the Uwrap on the externally bonded FRP system using the fracture mechanics (Mode 1 and Mode2 fracture energy). FE models of the pull-out test were created in two (2D) and three dimensions (3D). Based on the obtained results from the 3D models in particular pull-out specimens, more efficient 2D models were developed.

Based on the results from the numerical study, the effect of the interfacial modeling in the global and local response of the externally bonded FRP system was assessed. These findings were used to successfully predict the behavior of a large scale FRP strengthened beam. The following conclusions were derived from the numerical study.
- The developed frictional bond-slip model accurately predicts the bond behavior between the concrete and the FRP under the Uwrap.

- The experimentally observed increase in the maximum load and frictional behavior of the FRP sheet after the debonding between the concrete and the FRP could be captured by the use of the proposed frictional bond-slip model.

- It was found that based on the results of the performed FE analysis, the stress contour of the pull-out specimens significantly changed after debonding of the FRP sheet. The stress contour on the Uwrap could be used for the effective Uwrap design in the externally bonded FRP system.

- An increase in the Uwrap length could increase the maximum slip between the body of the concrete and the FRP sheet, and it can also change the failure mode from the debonding of the Uwrap to the rupture of the Uwrap.

- It can be concluded that the tilted Uwrap orientation such as 45 degree Uwrap (PU-45) could increase the slope of the load-displacement graph before the onset of debonding.

- Optimizing the corner region with the thickness, orientation, and the corner length, will lead to the effective anchor (Uwrap) design.

- An effect of the Uwrap width observed from the experiments could not be differentiated decisively from the numerical analysis as it was assumed that the entire area under the Uwrap locations have the same frictional bond-slip model.

- All developed numerical models for the T-beam were able to predict the sequences of the experimental failure modes, e.g. yielding of the steel
reinforcement followed by debonding of the FRP sheet.

- The numerical models that consider Mode 2 and mixed Mode fracture interfaces (Cohesive model) showed a better prediction capability for the entire load-deflection range of the strengthened T-beams.

- Strain distributions in FRP sheets are well predicted by all the numerical models developed in this study. The use of mixed-mode interfacial elements (Cohesive model) adds the capability of capturing the effects of the stress concentrations around the cracked locations in the concrete soffit.

- FRP stress contours, obtained from the numerical models, showed that the level of stresses from the longitudinal FRP sheet, as well as the location of concrete shear-flexure cracks, can activate larger regions of the Uwrap.

- The equation estimating debonding failure from the ACI was found to be too conservative especially in the case of Uwrap strengthening.

- In the modeling of the T-beam with anchor effect, additional flexural strengthening capacity (33\%) was obtained due to the addition of the Uwraps.

- When the frictional behavior was considered, the FE models shows a more ductile behavior of the T-beam compared to the models with Uwraps and no frictional interface.

- If the installed Uwrap has less capacity than the threshold value defined by the analysis, the Uwrap can not carry the load from the released strain energy generated by debonding of the FRP sheet, resulting in a no-anchor effect. Defining threshold value should be also accompanied with experimental results.
• The ACI equation for the debonding of the FRP sheet does not consider the anchor effect possibly obtained from the use of the Uwrap. For further study, developing new equations in the case of the Uwrap is recommended.
Chapter 5
Conclusions and recommendations

Experimental and numerical procedures were proposed to characterize the effect of the Uwrap in the performance of externally bonded FRP system (EBFS). Based on several conducted experiments, the frictional bond-slip model was developed and used for predicting the behavior of the Uwrap in the performance of externally bonded FRP system (EBFS). Accordingly, the anchor effect obtained from the Uwrap system was evaluated and used for the design of a large scale T-beam strengthened by an FRP sheet. An increase in the debonding strain of the FRP sheet due to the addition of the Uwrap in the test of the T-beam indicates how the anchor effects of the Uwrap enhance the strength of the T-beam and prevent the debonding propagation.

The developed FE models based on the frictional bond-slip model also predict the increased strength of the beam and the local behavior of the Uwrap. The effectiveness of the proposed numerical method has been validated by experimental results, indicating that the cohesive element used for the concrete-FRP interface and the frictional bond-slip model proposed in this study, which consider the region under the Uwrap, are the principal factors needed to predict the bond behavior of FRP system. Conclusions and recommendations derived from this study are as follows.
Conclusions

Two key factors were found pertaining to the externally bonded FRP system with the Uwrap. The first factor is the stress concentration at the corner. The stress concentration at the corner region of the Uwrap was the main cause of the rupture failure of the Uwrap. If the stress concentration at the corner region is too high, rupture of the Uwrap occurs. By contrast, if the stress concentration factor is small, debonding of the Uwrap occurs. Therefore, stress concentration at the corner region should be considered in order to predict the Uwrap behavior. The stress concentration could be minimized as the corner of the beam is grinded.

The second factor is the frictional effect between the debonded surfaces under the region of the Uwrap after the onset of the debonding. This factor should be also considered in order to characterize and evaluate the anchor effect of the Uwrap in the FRP strengthened beam. The frictional effect between the concrete and FRP under the region of the Uwrap increases the maximum stresses in Mode 1 and Mode 2 directions as well as the fracture energy of the concrete-FRP interface. This improved performance of the interface as the Uwrap generates frictional effects between the FRP sheet and the concrete.

Based on these two key factors, a frictional bond-slip model was proposed in this study in order to predict the anchor effect of the Uwrap. The developed FE analysis with the proposed frictional bond-slip model well predicts the bond behavior under the Uwrap locations. An increase in the maximum load and frictional behavior of the FRP sheet after the debonding were well captured by the use of the frictional bond-slip model.
The developed FE models for strengthened T-beams that consider a Mode 2 and mixed Mode fracture interfaces (Cohesive model) showed a better prediction capability for the entire load-deflection range. The use of the cohesive model adds the capability of capturing the effects of the stress concentrations around the cracked locations in the concrete soffit in the beam analysis.

It was found from the developed FE model of pull-out specimens that the stress contour of the Uwrap is significantly changed depending on the occurrence of the debonding between the concrete and FRP sheet. Before the debonding of the FRP sheet, the longitudinal stress at the front corner area of the Uwrap has the highest value. After the debonding of the FRP sheet, the highest longitudinal stress in tension is at the entire corner region of the Uwrap. The highest shear stress was generated at the corner region of the Uwrap even before the debonding of the FRP sheet.

From the FE models of the T-beams, there was a threshold value for the capacity of the Uwrap based on the obtained results of the FE analysis. If the installed number of Uwrap has less capacity than the threshold value, the Uwrap could not carry the load from the released strain energy generated by debonding of the FRP sheet, resulting in a no-anchor effect.

**Recommendations**

The digital image correlation technique was used for measuring the slip along the length of the FRP sheet and was found to be an effective non-contact, full displacement measuring technique. However, the continuous data point could not be obtained due to the limitation of the data processing speed of the equipment. Furthermore, an overturning
rotation of the pull-out specimens due to the pulling load generated erroneous results at some edge areas of the pictures. To obtain a better image correlation, the data acquisition system for high speed image processing should be equipped, and overturning rotation due to the pulling load should be minimized.

From the shear test of the unidirectional FRP sheet, it was found that one layer of the FRP sheet has a smaller modulus and shear strength compared to that of the stacked layers of the FRP sheet. It is thought that an increase in the fractured path in the stacked layer increases the modulus as well as shear strength of the FRP sheet. Further study is needed to investigate the relation between the strength and the number of layers.

From the developed models of the pull-out specimens, the effect of the width observed from the experiments could not be differentiated decisively from the developed numerical analysis. It is thought that FRP strain values are considerably larger at the edge than at the center of a sheet. This effect was not included in the models, indicating that the developed models are not sensitive to the width of the Uwrap. The frictional bond-slip behavior for the large width of the Uwrap has to be developed and evaluated after taking into account the effect of the Uwrap width.

The frictional bond-slip behavior was developed based on the observed phenomenon and results from the pull-out tests in this study. The parameters related to the equations in order to determine point 1 through point 4 of the frictional bond-slip behavior are needed to be calibrated with the available experimental results from others. At the same time, more beams should be also tested with various types of the FRP Uwrap in order to develop proper design guidelines for the anchor and shear effect of the Uwrap.
The verification of the frictional bond-slip model for the various types of the FRP Uwrap should be conducted as a future study.

Accordingly, a greater number of test results and verification of the model are required steps to reach the design guidelines and design equations for the Uwrap as an anchor system. The developed design equations of anchor effect should be also incorporated with current design equations for the shear strengthening of the RC beam.

As a further study, developing of a prototype of the Uwrap-anchor system is recommended. If the prototype is developed for most cases of the Uwrap-anchor system, the fabrication cost from the factory and design cost could be minimized, resulting in a more economical design of the anchor system. Based on the developed models and proposed frictional bond-slip model, a prototype of the Uwrap anchor could be explored.

Regarding the nature of safety issues related to the externally bonded FRP system, integrating an early warning system for passive safety is recommended. Since the use of the Uwrap can delay the debonding propagation of the concrete-FRP interface, an early warning system can help the reliability issues of the structures strengthened with FRP laminate and Uwraps. Use of the health monitoring system including FBG (Fiber Bragg Grating) sensing (e.g. Wang et al. 2007) or using OTDR (Optical Time-Domain Reflectometer) technique especially for highly uneven strain field (e.g. Hou et al. 2009) could be explored.
Bibliography


ACI Committee 440. (2008). Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures. ACI 440.2R-08, American Concrete Institute


dependence of mode I fracture behaviour in carbon-fibre/epoxy composite
laminates” Composites Science and Technology, 58(3-4), 591-602.

materials.”Composites Part A: Applied Science and Manufacturing, 38(11), 2333-
2341.


deformation measurements by digital image correlation.” Proceedings of SPIE –
The International Society for Optical Engineering, Vol. 6341, Speckle06:
Speckles, From Grains to Flowers

Flexural Strengthening of RC Beams Damaged by Corrosion of Tension Rebar.”,
The Third International Symposium on Non-Metallic (FRP) Reinforcement for
Concrete Structures, Sapporo, Japan, vol. 1, 435-442.

dams.” Earthquake Eng Struct Dyn 27(9), 937–56.

on the failure of geometrically similar concrete beams strengthened in shear with

sheets/plates bonded to concrete.” Engineering Structures, 27(6), The Occasion of
MEGA 2003, 920-937.


continuous fiber composite laminate.” Engineering Fracture Mechanics 42(3),
543-561

Mattock, A. H. “Shear friction and high-strength concrete.” (2001) ACI Structural
Journal 98(1), 50-59.


Experimental investigation of smart FRP-concrete composite beam with embedded FBG sensors


Appendix A

Softening behavior of three point bending test

Figures included in this section respectively show the load-LVDT and load-CMOD behavior of plain concrete (PC) and concrete epoxy-interfaces (CEI) tested under the both CMOD control and stroke control for this study.

Figure A-1: cmod_2

Figure A-2: cmod_3
Figure A-3: cmod_4

Figure A-4: cmod_5

Figure A-5: cmod_7
Figure A-6: cmodIE_1

Figure A-7: cmodIE_2

Figure A-8: Stroke_1
Figure A-9: Stroke_2

Figure A-10: Stroke_3

Figure A-11: Stroke_4
Figure A-12: Stroke_5
Appendix B

Mode 1 fracture tests with FRP sheet

Normal fracture energy could be used for the area where Uwrap is not present. However, the fracture energy of the interface under the Uwrap locations in Mode 1 direction could be different (larger) than that of the interface without an Uwrap. It was already observed from the previous splitting tensile test that the maximum tensile strength could be increased due to the addition of the FRP sheet. Similarly, increased concrete fracture energy is expected from the addition of the FRP sheet. The purpose of this test is to see the effect of the FRP sheet on the fracture energy.

To see the effects of the Uwrap on the fracture energy in Mode 1 direction, a three-points bending test on the notched beam with the FRP sheet (SCH41s) was designed and performed. It should be noted that the concrete batch for this test group is different than that for the ordinary fracture test.

Figure B-1 describes how the FRP sheets were attached to the beams. The length of the FRP sheet was changed to see the effect of the length of the Uwrap. The same geometry of the beam and test set ups were used. The test was conducted under the stroke control because the previous test results indicates that the stroke control is also appropriate method to measure the fracture energy of the CEI as long as the loading head moves in slow manner (0.05 mm/min).
Obtained results are shown in Table B-1. Four different types of specimens were tested such as the plain concrete, concrete-epoxy interface (CEI), CEI, short FRP sheet (100 mm) and CEI, and long FRP sheet (200 mm). From the tests, normal fracture energy for the plain concrete and CEI were 104 N/m and 95 N/m, respectively.

<table>
<thead>
<tr>
<th>Type</th>
<th>Specimen</th>
<th>Fracture Energy (N/m)</th>
<th>FRP length (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain concrete</td>
<td>PC</td>
<td>104</td>
<td>0</td>
</tr>
<tr>
<td>Concrete-epoxy interface</td>
<td>CEI-1</td>
<td>93</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>CEI-2</td>
<td>96</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>average</td>
<td>95</td>
<td></td>
</tr>
<tr>
<td>Concrete-epoxy interface</td>
<td>CEISF-1</td>
<td>270</td>
<td>100</td>
</tr>
<tr>
<td>+ Short FRP sheet</td>
<td>CEISF-2</td>
<td>321</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>average</td>
<td>296</td>
<td></td>
</tr>
<tr>
<td>Concrete-epoxy interface</td>
<td>CEILF-1</td>
<td>1475</td>
<td>200</td>
</tr>
<tr>
<td>+ Long FRP sheet</td>
<td>CEILF-2</td>
<td>1761</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td>average</td>
<td>1618</td>
<td></td>
</tr>
</tbody>
</table>

For the specimens with the FRP sheet, obtained fracture energy increased as expected. For the CEI with a short length of FRP sheet (100 mm), the average value of fracture energy was 296 N/m. For the CEI with a long length of FRP sheet (200 mm), 1618 N/m was obtained as fracture energy. It is thought that the additional fracture
energy from the interface between the FRP and the side of the concrete beam was added to the fracture energy of the interface.

From this test, it was concluded that the addition of the FRP sheet could increase the fracture energy significantly by seventeen times which is significant changes. However, it should be noted that the bending moment generated from the FRP sheet was also added to the obtained fracture energy, so the actual fracture energy could be lower than this and to get pure fracture energy, a further study should be associated with this test results. Accordingly, it can be expected that the fracture energy under the Uwrap in Mode 1 direction could be much larger than normal interface without the Uwrap due to the additional fractured area of the interface between the FRP sheet and the side of the concrete beam. However, the increment could be different depending on the geometric and material properties of the Uwrap and concrete. Observations from this tests and obtained results support the proposed frictional bond-slip model, which was presented in Chapter 4.
Appendix C

Load-displacement graph of the pull out specimens

In this section, the load-displacement graphs obtained from the pull-out tests would be shown. Data point obtained from the digital image correlation technique would be also shown in the graph as dots.

Figure C-1: PU-C-1
Figure C-2: PU-C-2

Figure C-3: PU-C-3
Figure C-4: PU-N-1

Figure C-5: PU-N-2
Figure C-6: PU-N-3

Figure C-7: PU-CN-1
Figure C-8: PU-CN-2

Figure C-9: PU-CN-3
Figure C-10: PU-L-1

Figure C-11: PU-L-2
Figure C-12: PU-L-3

Figure C-13: PU-MN-1
Figure C-14: PU-MN-2

Figure C-15: PU-MN-3
Figure C-16: PU-SP-1

Figure C-17: PU-SP-2
After FRP sheet debonding, Uw rap debonded

Figure C-19: PU-ST-1
Figure C-20: PU-ST-2

Figure C-21: PU-ST-3
Figure C-22: PU-SM-1

Figure C-23: PU-SM-2
Figure C-24: PU-SM-3

Figure C-25: PU-T-1
Figure C-28: PU-45-1

Figure C-29: PU-45-2
Figure C-30: PU-45-3
Appendix D

Slip profile along the length of FRP sheet using DIC method

In this section, the slip profile along the length of FRP sheet will be shown. Some of these data was already shown in Chapter 3 and compared with the numerical analysis in Chapter 4. In this section, slip profile from each specimen is shown. The slips were measured by the digital image correlation technique. However, due to the overturning rotation of the specimen from the pulling load, edge part of the image was not well predicted. However, the overall trends and obtained slip profiles at the Uwrap region were well measured. Two figures are shown for each specimen. Top figures show the strain profile before the debonding. Bottom figures show that after the debonding of the FRP sheet. From some of the specimens, the slip after the debonding was not measured since the camera position was changed when the sudden debonding failure of the FRP sheet occurred. Before the debonding of the FRP sheet, slip profiles will be shown at each selected level of the load. After the debonding of the FRP sheet, a slip profiles at the occurrence of the debonding will be only shown.
Before the debonding

Figure D-1: PU-N-1

Before the debonding

Figure D-2: PU-N-2
Figure D-3: PU-N-3
Before the debonding

After the debonding

Figure D-4: PU-CN-1
Figure D-5: PU-CN-2
Figure D-6: PU-CN-3
Before the debonding

After the debonding

Figure D-7: PU-L-1
Figure D-8: PU-L-2
Before the debonding

Figure D-9: PU-L-3
Before the debonding

After the debonding

Figure D-10: PU-MN-1
Before the debonding

After the debonding

Figure D-11: PU-MN-2
Before the debonding

After debonding

Figure D-12: PU-MN-3
Before the debonding

After the debonding

Figure D-13: PU-SP-1
Before the debonding

After the debonding

Figure D-14: PU-SP-2
Before the debonding

After the debonding

Figure D-15: PU-SP-3
Before the debonding

After the debonding

Figure D-16: PU-ST-1
Before the debonding

After the debonding

Figure D-17: PU-ST-2
Before the debonding

After the debonding

Figure D-18: PU-ST-3
Figure D-19: PU-SM-1
Before the debonding

Figure D-20: PU-SM-2

Before the debonding

Figure D-21: PU-SM-3
Figure D-22: PU-T-1
Before the debonding

After the debonding

Figure D-23: PU-T-2
Figure D-24: PU-T-3
Before the debonding

Figure D-25: PU-45-2

Figure D-26: PU-45-3
Appendix E

Load-strain graph of the pull out specimens

In this section, load-strain graphs from the pull-out tests will be presented. Strain gages were only attached to the one representative specimen from same types of specimens. Specific locations of the strain gages could be found in Figure 3-46 in Chapter 3.

Figure E-1: PU-C-1
Figure E-2: PU-N-3

Figure E-3: PU-CN-1
Figure E-4: PU-L-3

Figure E-5: PU-MN-2
Figure E-6: PU-SP-3

Figure E-7: PU-ST-3
Figure E-8: PU-SM-3

Figure E-9: PU-T-3
Figure E-10: PU-45-3
Appendix F

Results obtained from the thermo-camera

To capture the debonding propagation along the length of FRP sheet, a thermo-camera was used. The name of the selected model is FLIR™ SC660. Maximum thermal sensitivity of this camera is about 0.045 K (Kelvin). It has also 307,000 individual temperature measurement pixels. An initial attempt of capturing a debonding propagation using the thermo-camera started from the idea that different material has different thermal conductivity. The thermal conductivity of air and concrete are about 0.025 W·K /m and 1.7 W·K /m, respectively. If the debonding occurs at the concrete-epoxy interface (CEI), the air would permeate between the FRP and concrete. Since the air and concrete thermal conductivity are different, temperature changes (cooling down process) would be also different. Accordingly, debonded regions and debonding propagation are expected to be captured using a thermo-camera. In order to capture debonding propagation using a thermo-camera, initially, heat from two 500 W halogen lamps was applied to increase the temperatures of the surface of the FRP sheet. However, it was difficult to distribute a concentrated heat evenly from the halogen lamp to the entire surface of interest on the specimens as shown in Figure G-1. Uneven heat distribution can be found from the surface of interest. Accordingly, temperature changes were also not effectively captured during the test. It is thought that heat generated from the frictional behavior of the debonded surfaces between the concrete and FRP could increase the temperature of the FRP sheet, eliminating the cooling down process which can be obtained from thermal
conductivity. Therefore, it was decided that the halogen lamp would not be used. Instead, a thermo-camera was used to capture the increased temperature generated from the frictional behavior between the FRP and concrete without applying the heat of a halogen lamp, and it was found that measuring temperature without a halogen lamp was a better method in order to observe the debonding propagation more clearly than the method with the halogen lamp.

Figure F-2 shows the results obtained from the PU-T-3 specimens. Figure F-2 shows the load-displacement graph of the loading end with the observed debonding propagation at selected points.

Generally speaking, it was observed that the Uwrap was holding and delaying the debonding propagation. Stiffness was changed when the propagation of the debonding was reached up to the Uwrap region. After that, the stiffness was not changed anymore, and the load was linearly increased up to maximum load. When the debonding was
reached up to the end of FRP sheet, a sudden drop of the load was observed. A final failure mode after the debonding of the FRP sheet was the rupture of the FRP sheet due to the stress concentration. These ruptured areas were also observed from the thermo-camera as shown in the fifth picture of Figure F-2. It is thought that since the fractured area of the FRP sheet generates heat due to a sudden release of the strain energy, the thermo-camera easily captured the increased temperature at the local position of the fractured area in the FRP sheet. A thermo-camera can also detect the highly shear stressed zone as shown in the fifth picture of Figure F-2. Since the Uwrap is holding the FRP sheet in a transverse direction after debonding between the concrete and FRP sheet, shear stresses were generated around the corner of Uwrap due to the shearing effect between the side and bottom of the Uwrap. This phenomenon can be also clearly seen in Figure F-4.
Figure F-3 shows the observation from the test of the PU-CN-3 specimen. From the first and second picture, it was also clearly seen that when debonding was propagated to the Uwrap, stiffness in the load-displacement graph was changed as seen in Figure F-2. It is also interesting to note that the stiffness of PU-CN-3 was changed at the debonding load (20 kN) of the PU-C specimen, which does not have the Uwrap.
Even after it reached up to bond capacity of the PU-C specimen, load was increased up to the maximum load (29.4kN). Therefore, it is proven from the thermo-camera that load was increased since the Uwrap was effectively holding the FRP sheet. After the debonding of the FRP sheet, a stress transfer was captured as shown in the fourth picture of Figure F-3. That picture clearly shows how the stress of the Uwrap is transferred from the FRP location to the corner region of the Uwrap. Due to the longer corner length, a debonding propagation of the Uwrap in the transverse direction from the location of FRP sheet to the corner region of the Uwrap was observed. At the end, the Uwrap rupture at the corner of the concrete block was seen as shown in the fifth picture.
of Figure F-3. From these observations, it was found that the corner length allows larger
displacement as debonding is propagated from the location of the FRP sheet to the
corner region of the Uwrap.

Figure F-4 shows the observation from the PU-L-2 specimen. As previously
seen, the debonding propagation was delayed due to the addition of Uwrap. When the
propagation was reached to the region of the Uwrap, stiffness was also changed. After
that, load was gradually increased up until the propagation was reached up to the end of
the FRP sheet. After the complete debonding between the FRP and concrete, stress was
transferred to the corner of the Uwrap as the fifth picture of Figure F-4 shows. In this
case, stress was transferred earlier than the PU-CN-3 since the corner length is much
shorter than the PU-CN-3. The thermo-camera captured a highly shear stressed zone at a
corner region after the debonding of the FRP sheet. It shows how the Uwrap is holding
the FRP sheet and the location of the highly stressed zone. Even though the corner area
was damaged by a shear deformation (mostly matrix is damaged), it still can deliver the
force from the FRP sheet to the side of the Uwrap since fibers are not fractured and still
connect the FRP sheet to the side of the Uwrap. Finally, this load carrying capacity of the
Uwrap after a debonding of the FRP sheet was decreased to zero as the fiber of the
Uwrap corner was fractured, or as the Uwrap was debonded from the side of concrete
block. In this case, rupture of the Uwrap at corner area was observed due to the stress
concentration.
By using a thermo-camera, debonding propagation and highly stressed zone of the FRP sheet and Uwrap were observed. It was also clearly observed that the debonding is delayed when it reached up to the Uwrap location by the thermo-camera. The failure mode of the Uwrap and the region of the ruptured Uwrap were also observed by the thermo-camera.

However, debonding propagations could not be distinguished from some of the specimens since the temperature differences were very small. To observe differences
between a debonded area and a sound area, a calibration of the thermo-camera and a
better sensitivity which is larger than 0.045 K is needed.

It was also found that the heat by application of halogen lamp was not a necessary
step for a preparation of the surface as the heat is not well distributed with normal set up.
Accordingly, the unbalanced heat distribution on the surface was observed. Therefore, a
sophisticated method for applying the heat evenly on the entire surface of interest should
be developed. However, if discerning the debonded area from the sound area is the
purpose of the test rather than observing continuous debonding propagation, the halogen
lamp could be used so that it can capture the debonded area by temperature changes due
to differences between the thermal conductivities of the concrete and air.
Appendix G

Calculation of effective bond length ($L_e$) and derivation of Equation 4.1

In this Appendix, some calculations for effective bond length would be shown and derivation of Equation 4.1 would be explained.

G.1 Calculation of effective bond length ($L_e$)

Effective bond-length for 1 layer, 2 layers and 3 layers of FRPs based on Chen and Tang’s equations were calculated. The used equation is shown in Eq. G.1

$$L_e = \sqrt[4]{\frac{E_p t_p}{f_c}}$$  \hspace{1cm} G.1

Where,

$E_p$ : Young’s modulus of the FRP sheet (MPa),

$t_p$ : Thickness of the FRP sheet (mm),

$f_c$ : Compressive strength of the concrete (MPa)

Calculated effective bond lengths for 1 layer, 2 layers and 3 layers are 105 mm, 149 mm and 183 mm respectively.
G.2 Derivation of Equation 4.1

The Eq. 4.1 was developed to estimate the maximum shear stress of the concrete-FRP interface due to the addition of the Uwrap. As explained in Chapter 4, in order to estimate the maximum shear stress under the Uwrap, a deformation in the Mode 1 direction at an onset of damage in Mode 2 direction should be first calculated from three variables as follows.

\( w_{1c} \): A deformation at a complete damage in the Mode 1 direction

\( w_{1m} \): A deformation in Mode 1 direction at the onset of the damage in the Mode 2 direction

\( w_{2m} \): A deformation at the onset of the damage in the Mode 2 direction

\( w_{2c} \): A deformation at complete damage in the Mode 2 direction.

These three variables can be found in Figure G-1.

Figure G-1: Force-separation behavior of interface under Mode 1 and Mode 2 loads
It was assumed that a ratio of a deformation at the onset of the damage \( w_{2m} \) to a deformation at the complete damage \( w_{2c} \) in the Mode 2 direction multiplied by the deformation at a complete damage \( w_{1c} \) in Mode 1 direction is a deformation \( w_{1m} \) in the Mode 1 direction at the onset of damage in Mode 2 direction as shown in Eq. G.2.

\[
w_{1m} = w_{1c} \frac{w_{2m}}{w_{2c}} \tag{G.2}
\]

This calculated deformation in Mode 1 direction at the onset of the damage in the Mode 2 direction is used to calculate the generated stress from the Uwrap.

In order to calculate the generated stress due to the Uwrap, this deformation in Mode 1 direction will generate strain in the Uwrap as shown in Figure G-2.

The strain \( \varepsilon \) along the width of the Uwrap \( l_b \) due to the deformation \( w_{1m} \) can be calculated as Eq. G.3. The width of the Uwrap \( l_b \) can be found from Figure G-2.

\[
\varepsilon = \frac{2w_{1m}}{l_b} \tag{G.3}
\]
It can be assumed that the strain ($\varepsilon$) along the width of the Uwrap ($l_b$) will acting upon the concrete-FRP interface, generating the stress along the width of the Uwrap. The force ($F$) can be calculated with the thickness of the Uwrap using both the Eq. G. and Eq. G. as shown in Eq. G.4.

$$F = 2E_{FRP}E_{FRP} = \frac{4E_{FRP}W_{1m}}{l_b}t_{FRP} = \frac{4E_{FRP}t_{FRP}W_{1c}W_{2m}}{l_bW_{2c}}$$  \hspace{1cm} G.4

Where

$F$: Force acting upon the concrete-FRP interface due to the Uwrap

$E_{FRP}$: Elastic modulus of the Uwrap

$t_{FRP}$: Thickness of the Uwrap

$l_b$: Width of Uwrap which covers the FRP sheet in soffit of the concrete

Using the equation Eq. G.4, the average increase in a normal stress ($\sigma_{inc}$) due to the Uwrap acting on the concrete-FRP interface can be estimated as shown in Eq. G.5.

$$\sigma_{inc} = \frac{F}{l_b} = \frac{4E_{FRP}t_{FRP}W_{1c}W_{2m}}{l_b^2W_{2c}}$$  \hspace{1cm} G.5

This stress ($\sigma_{inc}$) is normal stress acting on the interface due to the deformation of the Uwrap. In order to estimate the increase in shear stress ($\tau_{inc}$) due to the deformation of the Uwrap using maximum shear stress ($\tau_{max}$), the ratio of the increased normal stress ($\sigma_{inc}$) to the maximum tensile stress ($f_t$) was used to as shown in Eq. G.6

$$\tau_{inc} = \frac{\tau_{max}}{f_t} \sigma_{inc} = \frac{4E_{FRP}t_{FRP}W_{1c}W_{2m}}{l_b^2f_tW_{2c}}\tau_{max}$$  \hspace{1cm} G.6
Therefore, increased maximum shear stress ($\tau_{inc,max}$) could be calculated as shown in Eq. G.7.

$$
\tau_{inc,max} = \tau_{max} + \tau_{inc} = \left(1 + \frac{4E_{FRP}I_{FRP}W_{1c}W_{2m}}{l_b^2 f_t W_{2c}}\right)\tau_{max} \tag{G.7}
$$

If the Uwrap orientation ($\theta$) is considered, the equation for increased maximum shear stress ($\tau_{inc,max}$) could be rewritten as Eq. G.8

$$
\tau_{inc,max} = \tau_{max} + \tau_{inc} = \left(1 + \frac{4E_{FRP}I_{FRP}W_{1c}W_{2m}}{l_b^2 f_t W_{2c}}\cos \theta\right)\tau_{max} \tag{G.8}
$$
VITA

Jae Ha Lee

Education:

Ph.D, Civil engineering (Structure), Pennsylvania State University, UP, PA

M.S, Civil Engineering (Structure), Pennsylvania State University, UP, PA

B.E, Architectural Engineering (Structure), Handong Global University, S.Korea

Work Experience:

The Pennsylvania State University, UP, PA

Research Assistant (2005-2010)

Selected Publications:

Lee, Jae Ha, Lopez, Maria M., Use of Mixed Mode Fracture Interfaces for the Modeling of Large Scale FRP Strengthened Beams, Journal of Composites for Construction (in press)

Gullapalli, Anusha, Lee, Jae Ha, Lopez, Maria M., Bakis, Charles E. Effect of Sustained Loading and Temperature on the FRP-Concrete Bond Performance. Transportation Research Record (Dec. 2009)

Lee, Jae Ha, Lopez, Maria M. and Bakis, Charles E., Slip Effects in Reinforced Concrete Beams with Mechanically Fastened FRP Strip. Cement and Concrete Composites (Aug. 2009)