STRENGTHENING DAMAGED REINFORCED CONCRETE BEAMS AND SLENDER COLUMNS USING ULTRA-HIGH MODULUS CFRP PLATES

by

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Abstract

This thesis investigates the application of ultra-high modulus carbon fiber reinforced polymer (CFRP) plates to strengthen damaged reinforced concrete beams and slender columns. In the first phase, two different pre-repair loading histories were simulated in seven 3000x300x150 mm reinforced concrete beams, namely cracking within the elastic range, and overloading in the plastic range. After unloading, the beams were repaired with either high- or ultra-high modulus (210 or 400 GPa) CFRP plates, or a hybrid system, and then reloaded to failure. It was shown that the level of pre-existing damage has an insignificant effect on the strengthening effectiveness and the failure mode at ultimate. The 210 and 400 GPa CFRP of reinforcement ratio \( \rho_f = 0.17\% \) increased the ultimate strength by up to 29 and 51\%, respectively, despite the 40\% lower tensile strength of the 400 GPa CFRP, due to the change in failure mode from debonding to rupture. Doubling \( \rho_f \) of the 400 GPa CFRP to 0.34\% resulted in a 63\% overall gain in flexural strength, only 8\% increase in ultimate strength over \( \rho_f = 0.17\% \), due to change in failure mode from rupture to concrete cover delamination. The beam retrofitted by hybrid CFRP showed remarkable pseudo ductility and warning signs before failure. However, a parametric study revealed a critical balance in proportioning the areas of hybrid CFRP to achieve reliable pseudo ductility. In the beam with \( \rho_f = 0.34\% \), this was achieved using a maximum of 30\% \( \rho_f \) of the 400 GPa CFRP. The second phase of this thesis presents an analytical model developed by modifying the provisions of the ACI 318-08 code and employing the computer software Response 2000, to predict the performance of CFRP strengthened slender reinforced concrete columns. Response 2000 is used to establish the interaction curve while the modified ACI 318-08 code is used to acquire the slender column loading path to failure including the
second order effects. The model predicts that the effectiveness of the FRP strengthening system increases as the slenderness ratio and FRP reinforcement ratio increase.
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Chapter 1: Introduction

1.1 General

The increased deterioration of concrete infrastructure has become a challenge for many modern cities worldwide. Potential solutions range from replacement of the structure to strengthening or rehabilitating with various techniques. The external plate bonding technique is now established as a simple and convenient repair method for increasing the flexural performance and stiffness of concrete elements. The advantage of this technique compared to the replacement of the structural element is that it is not only more economical, but its application can be carried out while the structure is still in use.

Early research into the performance of members strengthened by plate bonding was carried out in South Africa and France where steel plates were applied externally (Fleming and King 1967; Bresson 1971). Over the years, repairing and rehabilitating reinforced concrete beams using steel plates was proven to be an effective method of improving structural performance. In terms of analysis, the several experimental studies verified the validity of the strain compatibility method for the prediction of the failure loads, and concluded that external steel plates can be treated in a manner similar to conventional longitudinal reinforcement (Triantafillou and Plevris 1992). Since the plates are not protected by the concrete in the same way as internal reinforcement, the possibility of corrosion exists which could adversely affect the bond strength, leading to failure of the strengthening system.

Fiber-reinforced polymer (FRP) materials do not suffer from corrosion problems and most of their mechanical and physical properties are superior to those of steel. Furthermore, FRP has higher strength-to-weight and stiffness-to-weight ratios than steel and thus can provide
more strength to the structure and is easier to handle and install. Numerous research studies have already been performed to examine the behaviour of reinforced concrete members strengthened with externally bonded FRP plates. Guidelines have been provided by the ACI Committee 440 outlining the procedures for the design and construction of externally bonded FRP systems for strengthening concrete structures. However, little research has been focused on how the extent of damage will affect the performance of the FRP strengthening system. Also, little research has been conducted on the use of multiple strips of different modulus of FRP to induce a progressive failure. This thesis will investigate both these shortcomings.

Little research has investigated the use of externally bonded FRP plates to strengthen slender reinforced concrete columns. Several research studies have looked at FRP strengthening of slender reinforced concrete columns by wrapping or jacketing. Shaat and Fam (2009) looked at the application of high-modulus FRP plates to strengthen slender steel columns. Due to the success of the application to steel, this thesis will investigate the same application to concrete using an analytical model.

1.2 Objectives

The primary objective of this thesis is to examine the application of CFRP plates of ultra-high Young’s modulus (up to 400 GPa) to strengthen pre-cracked and yielded reinforced concrete beams and the associated failure modes. The thesis also examines the theoretical performance of the application of CFRP plates of ultra-high Young’s modulus (up to 400 GPa) to strengthen slender reinforced-concrete columns. Specific objectives include:
1. To examine and compare the change in deflections and strain levels of pre-cracked and yielded reinforced concrete beams repaired with high (200 GPa) and ultra-high (400 GPa) modulus externally bonded CFRP plates.

2. To examine and compare the change in deflections and strain levels of a pre-cracked beam strengthened with a hybrid or mixed moduli of CFRP strips to induce a progressive failure of CFRP.

3. To determine the performance and strength of each retrofitted reinforced concrete beam when statically tested to failure under four-point bending and compare the results to each other and control tests with no strengthening or damage.

4. To observe the various failure mechanisms of all beams when tested to failure.

5. To develop an analytical model to accurately predict the behaviour of pre-cracked and yielded reinforced concrete beams strengthened with longitudinal CFRP plates.

6. To develop an analytical model to predict the behaviour of slender reinforced concrete columns strengthened with longitudinal CFRP plates.

1.3 Scope

The scope of this thesis consists of experimental investigations and comparisons with control tests and analytical models. The experimental program is designed to address the performance of damaged beams repaired and strengthened with ultra-high modulus CFRP plates. The scope also includes theoretical investigations where analytical models are developed to look at the use of CFRP plates as a method to repair damaged reinforced concrete beams and strengthen slender reinforced concrete columns.

The experimental work was divided into two phases. Phase I was the damaging phase where the beams were loaded to either a load 75% of the yield load or a deflection two times the
deflection at yielding. Phase II was the retrofitting phase where these damaged beams were repaired with varying amounts of high or ultra-high modulus CFRP plates externally bonded to the beams tension face and tested to failure. This study also introduces the concept of hybrid moduli by having CFRP strips of mixed moduli to induce a progressive, gradual failure of CFRP.

The theoretical work was also divided into two phases. Phase I was an analytical model developed to predict the performance of the damaged beams repaired with externally bonded CFRP plates. The model employs the use of the computer software VecTor, FormWorks, and Response 2000. Phase II was an analytical model developed to predict the performance of slender reinforced concrete columns strengthened with externally bonded CFRP plates. This model is based on modifying the provisions of the American Concrete Institute (ACI 2008) and also employs the computer software Response 2000.

1.4 Thesis Outline

Chapter 2: reviews the literature and research on externally bonded CFRP plates to strengthen reinforced concrete members. The general performance of previously conducted experiments is summarized.

Chapter 3: presents the study on the strengthening of damaged reinforced concrete beams using ultra-high modulus CFRP plates. This includes the damaging tests and final tests to failure of the repaired beams. Also included in Chapter 3 is the analytical model used to predict the performance of the repaired beams. The experimental results are used to validate the theoretical results.

Chapter 4: presents the study on a potential analytical model developed to evaluate the effectiveness of ultra-high modulus CFRP plates in strengthening slender reinforced concrete
columns of different slenderness ratios, CFRP reinforcement ratios, and effective length factors \( k \).

**Chapter 5**: provides the conclusions of the studies and presents recommended areas of future research.
Chapter 2: Literature Review

2.1 Introduction

This section introduces some of the background and applications of Fibre Reinforced Polymer (FRP) plates used as an externally bonded strengthening method for reinforced concrete beams and slender reinforced concrete columns.

2.2 General

FRP systems can be used to rehabilitate the strength of a deteriorated structural member, retrofit or strengthen a sound structural member to resist increased loads, or address design or construction errors. The use of FRP to strengthen reinforced concrete structures has been well established as a suitable method and has grown in popularity in recent years. More specifically, using FRP as external reinforcement is an effective and widely used system for structural strengthening and rehabilitation. Externally bonded FRP systems were developed as alternatives to traditional external reinforcing techniques such as steel plate bonding because of their noncorrosive properties and inherent durability. Additional properties which make FRP ideal for structural rehabilitation include their ease of installation and their high strength to weight and stiffness to weight ratios.

2.3 Strengthening Reinforced Concrete Beams

Bonding steel plates to the tension fibres of reinforced concrete members with adhesive resins was shown to be a viable technique for increasing their flexural strengths (Fleming and King 1967). Experimental research programs into the flexural behaviour of steel-plated beams were also reported by Ladner and Weder (1981), MacDonald (1978, 1982), Swamy et al. (1987), and Hamoush and Ahmad (1990). In terms of analysis, the mentioned experimental studies
verified the validity of the strain compatibility method for the prediction of the failure loads, and concluded that external steel plates can be treated in a manner similar to conventional longitudinal reinforcement (Triantafillou and Plevris 1992). Repairing and rehabilitating reinforced concrete beams using steel plates was proven to be an effective method of improving structural performance and has been used to strengthen many bridges and buildings around the world. However, because steel plates can corrode, leading to deterioration of the bond between the steel and concrete, researchers looked to FRP materials as an alternative.

To overcome some of these limitations that are associated with steel plate bonding, it was proposed in the mid-1980s that FRP plates could prove more adequate than steel plates in strengthening and repair applications (Meier 1987, Kaiser 1989). In Europe, the use of FRP materials as a replacement for steel for structural repair was pioneered at the Swiss Federal Laboratories for Materials Science and Technology or EMPA. Four point loading tests were performed on CFRP strengthened reinforced concrete beams with lengths of 2000 mm (Meier 1987, Kaiser 1989) and 7000 mm (Ladner et al. 1990). Strengthening was achieved through the use of 1.0 mm thick pultruded CFRP laminates with the same epoxy used in earlier steel plating (Hollaway and Leeming 1999). This research led to the first applications of externally bonded FRP systems to reinforce concrete bridges for flexural strengthening (Meier 1987).

In North America, Saadatrnanesh and Ehsani (1989, 1990, 1991) conducted studies where structural concrete beams were strengthened with GFRP plates bonded to the tension face. The results of the tests indicated that a significant increase in the flexural strength can be achieved. In addition, the epoxy bonded plates improved the cracking behaviour of the beams by delaying the formation of visible cracks and reducing crack widths (Saadatrnanesh and Ehsani 1991). An experimental study was undertaken by Chajes et al. (1994) at the University of
Delaware, in which a series of reinforced concrete beams 1120 mm in length were tested in four-point bending to determine the ability of externally bonded composite fabrics to improve the beams’ flexural capacity. The fabrics used were made of aramid, E-glass, and graphite fibres. The external composite fabric reinforcement led to a 36 to 57% increase in flexural capacity and a 45 to 53% increase in flexural stiffness. For the beams reinforced with E-glass and graphite fibre fabrics, failures were a result of fabric tensile failure in the maximum moment region. The beams reinforced with aramid fabric failed due to the crushing of the compression concrete.

Hutchinson and Rahimi (1993, 1996, 2001) conducted tests under the ROBUST project in the United Kingdom. The ROBUST project was the most comprehensive study of an FRP strengthening system where all aspects of materials, design, and analysis were addressed (Hollaway and Leeming 1999). Hutchinson and Rahimi (1996) examined both glass and carbon fibre laminates of different thicknesses and three internal steel reinforcement ratios. The use of GFRP was found to provide significant ductility and reasonable strength enhancement, whilst the increase in strength was greater with CFRP but with less ductility. Hutchinson and Rahimi (2001) as part of the ROBUST project conducted a study where flexural testing of 2300 mm long reinforced concrete beams with bonded external reinforcements took place. The test variables included the amount of conventional (internal) reinforcement and also the type and amount of external reinforcement. The results showed that the stiffness and strength of the beams strengthened with composite plates was substantially increased. The ultimate load capacity of the beams increased by as much as 230% over their unstrengthened counterparts.

Arduini and Nanni (1997) investigated pre-cracked, strengthened reinforced concrete specimens. Parameters in the study included two CFRP material systems, two concrete surface preparations, two reinforced concrete cross sections, and the number and location of CFRP plies.
It was shown that the effect of CFRP strengthening was considerable, but the effect of some of the tested variables was modest. A more recent study by Soudki et al. (2007) looked at reinforced concrete beams strengthened with CFRP sheets and subjected to an aggressive environment. The experimental beams were pre-cracked, repaired with CFRP sheets and subjected to 300 wetting and drying cycles with de-icing chemicals (3% NaCl) to simulate environmental exposure before testing them to failure under four-point bending. Based on the findings of the study, the long-term effectiveness of the CFRP strengthened reinforced concrete in aggressive corrosive environments was established.

These experimental studies present a wide variety of failure modes which Teng and Smith (2002) grouped into six categories as shown in Figure 2.1:

1. Flexural failure by FRP rupture prior to or after tension steel yielding
2. Flexural failure by concrete crushing in compression prior to or after tension steel yielding
3. Shear failure
4. Concrete cover delamination
5. Plate end debonding
6. Intermediate crack debonding

Among the six failures shown in Figure 2.1, the last three (d, e, f) are not found in conventional reinforced concrete beams but are unique to beams with plates externally bonded to the tension face. These modes are referred to as premature debonding failure modes, as they occur before flexural failure of the section (Teng and Smith 2002). The three modes of debonding are classified into two general types: 1) those that initiate at or near one of the plate
ends and then propagate away from the plate end; and 2) those that initiate at an intermediate flexural or flexural-shear crack and then propagate from the crack towards the plate end. The first type is referred to as plate end debonding and the second is referred to as intermediate crack induced debonding. Both types can propagate through either the cement matrix or along the adhesive layer causing cover delamination or FRP plate peeling (Teng and Smith 2002).

The ACI 440.2R-08 Guidelines (ACI 2008) recommend a strain limit to prevent intermediate crack-induced debonding. According to the guidelines, the effective strain in the FRP reinforcement should be limited to the strain level at which debonding may occur, $\varepsilon_{fd}$, as defined by:

$$\varepsilon_{fd} = 0.41 \frac{f_r}{nE_f t_f} \leq 0.9 \varepsilon_{fu}$$  \hspace{1cm} (1)

where $E_f$ and $t_f$ are the elastic modulus and thickness of a single ply of the FRP laminate, $n$ is the number of plies, and $\varepsilon_{fu}$ is the ultimate strain capacity of the FRP laminate. This equation takes a modified form of the debonding strain equation proposed by Teng et al. (2001).

Although strengthening by bonding external FRP plates to the tension face of a damaged reinforced concrete beam is an acceptable and widely used method, further investigation is needed to study the influence of the extent of damage, modulus of the FRP, and amount of FRP on the effectiveness of the strengthening system. In addition, little research is found where beams are repaired with FRP strips of different modulus to induce a progressive failure. Therefore, this study will introduce the concept of a hybrid moduli repair system to induce a gradual failure and thus simulate ductility.


2.4 Strengthening Slender Reinforced Concrete Columns

Strengthening of reinforced concrete columns using FRP jackets or wraps is now a widely accepted technique. Confinement of reinforced concrete columns by means of FRP jackets can be used to enhance their strength and ductility (ACI 2008). Nanni and Bradford (1995) and Toutanji (1999) conducted experimental and analytical studies on the performance of concrete columns externally wrapped or jacketed with carbon and glass FRP sheets subjected to uniaxial compression. Results from both studies show that the external confinement by FRP jackets can significantly enhance the strength, ductility, and energy absorption capacity of the concrete specimens. Both studies also yielded satisfactory predictions of the stress-strain response. Several models that simulate the stress-strain behaviour of FRP-confined compression sections are now available in literature (Teng et al. 2002, De Lorenzis and Tepfer 2003, and Lam and Teng 2003a). The stress-strain model by Lam and Teng (2003a,b) for FRP-confined concrete has been adopted by and can be found in the ACI 4402R-08 guidelines (ACI 2008).

These design provisions, however, suffer from one important limitation: the effect of column slenderness (i.e. the second-order effect) is not included. Therefore, the design provisions are limited to the design of FRP jackets for short columns for which the second-order effect is negligible. Mirimiran et al. (2001) started the research in this field with concrete-filled FRP tubes (CFFT), which showed that as the slenderness ratio increased, the columns strength rapidly drops. Also on the experimental side, Tao et al. (2004), Fitzwilliam and Bisby (2010) and Bisby and Ranger (2010) tested FRP-confined circular columns with a height-to-diameter ratio up to 20.4. It is shown that FRP wraps increase the strength and deformation capacity of slender columns, although the beneficial confining effects are proportionally greater for short columns (Fitzwilliam and Bisby 2010). Jiang and Teng (2012a) developed a theoretical column
model capable of modelling the behaviour of slender FRP-confined reinforced concrete columns. The limited existing research has nonetheless allowed the following two significant observations to be made: 1) a reinforced concrete column which is originally classified as a short column may need to be treated as a slender column after FRP jacketing and 2) the effectiveness of FRP confinement in enhancing the load-carrying capacity of a reinforced concrete column decreases as the column becomes more slender (Jiang and Teng 2012). This is due to the fact that FRP confinement can substantially increase the axial load capacity of a reinforced concrete section, but its flexural rigidity in the range of confinement-enhanced resistance is much lower than the initial flexural rigidity (Jiang and Teng 2012). Tao and Yu (2008) investigated the performance of slender square reinforced concrete columns strengthened with CFRP jackets subjected to eccentric loading. The test results shows that no obvious strength increase was observed for specimens confined by unidirectional CFRP jackets, while an increase of 22.8% to 61.8% was obtained for those strengthened by bidirectional CFRP jackets. The study showed that the lateral confinement had little effect on strength enhancement but the longitudinal fibres were effective in enhancing the load-carrying capacity of slender reinforced concrete columns subjected to eccentric loading.

Research on the effectiveness of longitudinal FRP plates to strengthen slender reinforced concrete columns is very limited. Shaat and Fam (2009) looked at the application of high-modulus FRP plates to strengthen slender steel columns. The results show that the effectiveness of the CFRP system in increasing the axial strength of the columns increases substantially as slenderness ratios (kL/r) become higher. The results also show that the axial stiffness of the steel columns is also increased due to the application of CFRP; however, the increase is only slightly affected by the slenderness ratios. Due to the success of this application to slender steel columns
by Shaat and Fam (2009) and the lack of research on the same application to slender reinforced concrete columns, this study looks to gain insight on this topic through an analytical model.

**Figure 2.1 – Failure modes of beams strengthened with externally bonded FRP plates (Teng and Smith 2002).**
Chapter 3: Strengthening Damaged Reinforced Concrete Beams using Ultra-high Modulus CFRP Strips

3.1 Introduction

This chapter investigates the application of CFRP plates of ultra-high Young’s modulus to strengthen pre-cracked and yielded reinforced concrete beams. Seven large scale beams of 2900 mm span were tested in four-point bending, including a control unstrengthened beam. The remaining six beams were first loaded to either an equivalent service load level that induces cracking while the beam remains below the yielding point or, assuming a case of overloading beyond yielding, a deflection double that at yielding. After unloading, the beams were strengthened by either ultra-high (400 GPa) or high (210 GPa) modulus CFRP. Two additional parameters were examined, namely the reinforcement ratio, and hybrid moduli by having CFRP strips of mixed moduli to induce a progressive, gradual failure of CFRP. This was quite encouraging to investigate the application of hybrid moduli strips, which is particularly important due to the very brittle nature of FRP.

3.2 Experimental Program

Seven rectangular reinforced concrete beams were fabricated and tested. Cracking was first induced followed by repair using various types of CFRP plates and then the beams were tested to failure. The following sections provide details of the experimental program.

3.2.1 Test Specimens and Parameters

The seven 3000x300x150 mm reinforced concrete beams were identical. The longitudinal reinforcement consisted of top and bottom layers of two 15M steel rebar with clear concrete

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1 This chapter has been submitted for publication as the following journal paper: Richardson, T. and Fam, A. (2013) “The Modulus Effect of Bonded CFRP Laminates used for Repairing Pre- and Post-Yield Cracked Concrete Beams”, ASCE Journal of Composites for Construction, Under review.
covers of 16 mm. The transverse reinforcement included 5 mm diameter steel stirrups spaced at 150 mm throughout the span, except at the ends, where the spacing was reduced to 75 mm. Details and dimensions of test beams are shown in Figure 3.1.

Table 3.1 summarizes the test matrix, including various parameters, namely; level of damage, CFRP reinforcement ratio and CFRP modulus. Specimen B1 represents the control test to failure without any damaging or CFRP strengthening. Specimens B2, B3, and B7 were brought to a damage level within the elastic range by cracking up to a 56 kN, representing 75% of the yield load of control specimen B1 (75 kN). Specimens B4, B5, and B6 were brought to a damage level within the plastic range; represented by cracking up to a deflection level of 26 mm, representing double the yield deflection (13 mm) of B1.

The damaged specimens were then retrofitted with various amounts of the two types of CFRP plates of 210 GPa and 400 GPa Young’s moduli. The CFRP plates were applied to the tension face of the beam and had a consistent bond length of 2730 mm. Specimens B2 and B4 were each strengthened with one strip, representing a reinforcement ratio \( \rho_f = 0.17\% \), of the 210 GPa CFRP, while specimens B3 and B5 were each strengthened with one strip of the 400 GPa CFRP plate, with the same \( \rho_f = 0.17\% \). Specimen B6 was strengthened with two strips (\( \rho_f = 0.34\% \)) of the 400 GPa CFRP plate. Specimen B7 was strengthened with 80% \( \rho_f \) of the 210 GPa CFRP and 20% \( \rho_f \) of the 400 GPa CFRP, which totaled to \( \rho_f = 0.34\% \). Figure 3.1 presents the layout of the CFRP plate arrangements on the tension face of the beam.

### 3.2.2 Materials

Figure 3.2 shows the stress-strain curves of the two types of CFRP plates and steel rebar used in this study. The following is a detailed description of various materials used.

**Ultra-high modulus CFRP plates**: Unidirectional pultruded CFRP plates (Sika CarboDur
UH514) were used. The standard plate of 50 mm wide and 1.4 mm thick was used, except for beam B7, where a narrow strip of 20 mm width was cut from the standard strip in order to provide the desired CFRP reinforcement ratio. The manufacturer reported a longitudinal ultimate tensile strength and elastic modulus of 1450 MPa and 400 GPa, respectively.

**High modulus CFRP plates:** Unidirectional pultruded CFRP plates (Sika CarboDur M514) were used. The standard plate of 50 mm wide and 1.4 mm thick was also used, except for B7, where two narrow strips of 40 mm width were cut from the standard plates in order to provide the desired CFRP reinforcement ratio. The manufacturer reported a longitudinal ultimate tensile strength and elastic modulus of 2400 MPa and 210 GPa, respectively.

**Epoxy resin:** Sikadur 30 epoxy resin was used. The resin is a thixotropic adhesive mortar, based on a two-component solvent free epoxy resin. The mixing ratio was 3:1 of Component A (resin) and Component B (hardener) by weight. The elastic modulus, tensile strength, and shear strength as provided by the manufacturer are 11.7 GPa, 24.8 MPa, and 15 MPa, respectively.

**Concrete:** The concrete used in the beams was specified as 45 MPa at 28 days, with a maximum aggregate size of 19 mm. The cylinder strengths, measured in accordance with ASTM C-39 (ASTM 2010) at the time of beam testing ranged from 40 MPa to 48 MPa.

**Steel rebar:** 15M (16 mm) steel bars were used as longitudinal reinforcement. Tension tests performed in accordance with ASTM A370 (ASTM 2010) showed that the measured yield strength was 460 MPa.

### 3.2.3 Fabrication of Specimens

To prepare the surface of the concrete beam for CFRP application, the beams were sandblasted to provide a roughened surface to promote bonding and then air blasted clean to remove any debris. The CFRP plates were also cleaned with acetone until all the residual carbon...
dust was removed. The epoxy adhesive was then mixed and a nominal thickness of 1.5 mm was applied to the CFRP plates. They were then placed on the concrete surface and pressed down using a roller until the adhesive was forced out on both sides. The epoxy was left to cure for a minimum of 48 hours before testing.

### 3.2.4 Test Setup and Instrumentation

Figure 3.1 shows the schematic of the test setup while Figure 3.3 shows a picture of the test setup taken during testing. The retrofitted specimens were tested in two phases, namely: (a) the cracking/damaging phase of the beam up to the desired load (or deflection) before retrofitting, followed by complete unloading and CFRP retrofitting, then, (ii) the reloading to failure of the same beam. In both phases, the beams were simply supported over a span of 2900 mm and tested under four-point bending with a 500 mm constant moment region. The control beam was loaded monotonically to failure in a single phase. All tests were conducted in stroke control at a rate of 1 mm/min.

Deflections were measured using linear potentiometers (LPs). LPs were placed at mid-span and in the second phase, additional LPs were placed at 200 mm offsets from mid-span to capture deflections at the ends of the constant moment zone. Strains in the longitudinal steel rebar were measured using 5 mm uniaxial electric resistance strain gauges placed on all tension and compression bars at mid-span. Strains in the CFRP plate were measured at various points along its length using 10 mm uniaxial strain gauges in the longitudinal direction. Strains on the top concrete surface were measured in the longitudinal direction using 30 mm uniaxial strain gauges located at mid-span.
3.3 Experimental Results

Table 3.2 summarizes the test results for all beams, in terms of the yield and ultimate loads, maximum strain in the CFRP, deflections and failure modes. Figures 3.4 to 3.7 present the load-deflection responses of the test beams. Figure 3.8 presents the load-strain responses in steel rebar, CFRP plates and concrete. Figure 3.9 presents the strain distribution along the CFRP plates at various load levels. Figures 3.10 and 3.11 present the cracking patterns in the two stages of loading and failure modes, respectively. The following sections present the test results and discussion.

3.3.1 Effect of Extent of Damage

The effect of pre-repair extent of damage on strengthening performance is shown in Figure 3.4(a) where the load-deflection plots at mid-span of specimens B2 and B4 during the first and second phases of loading (i.e. before and after repair) are presented. Both beams had the same CFRP amount ($\rho_f = 0.17\%$) and modulus of 210 GPa but B2 was cracked in the elastic range, while B4 was overloaded beyond yielding. Also shown as a reference, is the load-deflection plot of control beam B1. The resulting yield loads for specimens B2 and B4 were 100 kN and 104 kN, respectively; a difference of only 3.9%. This represents 33% and 39% increase in yield load. The resulting ultimate loads were virtually identical, 116.4 kN and 115.9 kN, respectively; representing a 28% increase in ultimate capacity. Therefore, although the permanent deflections (Figure 3.4(a)) and cracks (Figure 3.10(a and b)) were much greater in specimen B4, the extent of this pre-repair damage shows little effect on the yield and ultimate loads after repair. Also, at the typical live load deflection limit of span/360 (8 mm) specified in building codes, the corresponding maximum service loads in B2 and B4 are 66.6 kN and 60.5 kN, representing 57% and 52%, of their respective ultimate loads. This reflects the better
enhanced stiffness in the elastic range repair, compared to the plastic range. When subjected to
damage in the plastic range, the deflection of the beam already exceeds this limit before repair.
Therefore, in these extreme cases, there would need to be some mitigation of the deflection such
as jacking of the beam before repair.

Figure 3.4(b) presents the mid-span load-deflection plots of specimens B3 and B5, also
during the first and second phases of loading, along with B1. Both beams had the same CFRP
amount ($\rho_f = 0.17\%$) and modulus of 400 GPa but B3 was cracked in the elastic range, while B5
was overloaded beyond yielding. The resulting yield load for specimen B3 was 120 kN, an
increase of 60%, while specimen B5, although unloaded from a yielded position, did not reach its
upgraded yield load in the second loading phase. This can be seen in Figure 3.8(e) from
reloading of rebar strain gauges, following a linear trend until CFRP rupture. The resulting
ultimate loads were 129.8 kN and 136.3 kN (a difference of 5%), representing a significant 43%
and 51% increase in ultimate capacity, respectively. Therefore, similar to specimens B2 and B4,
the extent of the pre-repair damage shows little effect on the ultimate loads after repair, but
clearly affects yielding load significantly by allowing it to exceed the ultimate load. Also, at the
live load deflection of span/360, the corresponding maximum service loads in B3 and B5 are
71.8 kN and 67.9 kN, respectively, representing 55% and 50%, of their respective ultimate loads.
Again, this demonstrates a slightly better enhanced stiffness in the elastic range than plastic
range.

### 3.3.2 Effect of Modulus

The effect of CFRP modulus on strengthening performance is shown in Figure 3.5 where
the load-deflection plots of retrofitted specimens B2 and B3 with 210 GPa and 400 GPa CFRP
moduli, respectively, are shown. Both specimens had an elastic cracking history. Figure 3.5 also
Chapter 3

compares B4 and B5 with similar plastic damage and retrofitted with CFRP of the two different moduli. All four beams had $\rho_f = 0.17\%$. The ultimate loads of B3 and B5 (400 GPa CFRP) were higher than B2 and B4 (210 GPa CFRP) by 12% and 18%, respectively, despite the fact that the 400 GPa CFRP has a 40% lower tensile strength than the 210 GPa CFRP (Figure 3.2). This is attributed to the change in failure modes as will be discussed. With regard to stiffness, although the CFRP Young’s modulus in B3 and B5 was almost double that of B2 and B4, the enhancement of stiffness was relatively small, 7.8% and 12.2%, in the elastic and plastic ranges, respectively.

Figure 3.6 presents the effect of the hybrid modulus CFRP on strengthening performance by comparing beams B6 with 400 GPa CFRP and B7 with mixed 210 and 400 GPa CFRP moduli. Both beams had a similar $\rho_f = 0.34\%$ and also both beams reached very similar ultimate loads, of only 2% difference (Table 3.2). The concept behind using a combination of CFRP plates of different moduli was to produce pseudo ductility by means of a gradual, progressive failure. As shown in Figure 3.6, specimen B7 showed the desired gradual failure where the ultra-high modulus CFRP plate (400 GPa) ruptured first, accompanied by a slight drop in load, followed by an increase in load at a lower stiffness to a level 11.4% higher than that of the first failure, when the high modulus plates (210 GPa) debonded. Specimen B7 deflected a total 24.5 mm (Table 3.2) (10.2 mm until the tension steel yielded, an additional 6.3 mm until the rupture of the ultra-high modulus CFRP and an additional 8 mm until the remaining high modulus CFRP debonded). Therefore, its ductility factor is 2.4. Specimen B6, on the other hand, showed a total deflection of only 16.1 mm after retrofitting. Although it had a plastic deformation history, it failed before reaching the upgraded yielding load. The fairly linear steep load-deflection curve of B6 (Figure 3.6) does not show any indication of the imminent failure at ultimate.
3.3.3 Effect of CFRP Reinforcement Ratio

Figure 3.7 presents the effect of the CFRP reinforcement ratio by comparing the load-deflection performances of specimens B5 and B6. Both beams experienced the equivalent pre-repair plastic damage and both were repaired with the ultra-high modulus CFRP plates (400 GPa). However, specimen B6 had a CFRP reinforcement ratio of 0.34%, double that of B5. Figure 3.7 shows that doubling the reinforcement ratio resulted in a 63% increase in flexural strength compared to control beam B1, but only 8% increase in ultimate load compared to B5, as the failure mode has changed, which is discussed next. The stiffness was increased by about 19% compared to B5.

3.3.4 Failure Modes

Control specimen B1, which was designed as a typical under reinforced section, failed as expected by yielding of the tension reinforcement, then yielding of compression steel (Figure 3.8(a)) and eventually, concrete crushing. For the CFRP-repaired beams, four failure modes occurred:

(a) The first was observed in specimens B2 and B4 strengthened with the high modulus CFRP (210 GPa), where the CFRP plates debonded from the concrete tension face as shown in Figure 11(a), after yielding of the tension steel reinforcement (Figure 3.8(b and d)). The same debonding failure mode occurred in both cases of elastic pre-repair cracking (B2) and plastic pre-repair damage (B4) at CFRP strains of 5512 and 4432 micro-strain, respectively (Table 3.2 and Figure 3.9(a and c)). These strains represent 39-48% of the CFRP rupture strain. Debonding was initiated at a flexural crack and progressed through the adhesive layer. Traces of concrete were observed on the debonded CFRP plate.

(b) The second failure mode was observed in specimens B3 and B5 strengthened with a \( \rho_f = \)}
0.17% of the ultra-high modulus CFRP (400 GPa), where the CFRP strips ruptured near mid-span as shown in Figure 3.11(b) at strains of 3167 and 3594 micro-strain (Table 3.2 and Figure 3.9(b and d), which are 87-99% of the manufacturer reported ultimate strain. In B3, this occurred shortly after steel yielding (Figure 3.8(c)), whereas in B5, although steel was reloaded from a plastic state, it did not reach the upgraded yielding load.

(c) The third mode of failure was observed in specimen B6 with a $\rho_f = 0.34\%$ of the ultra-high modulus CFRP (400 GPa). Unlike B3 and B5, where the CFRP ruptured, bond failure in the form of concrete cover delaminated occurred in this case as shown in Figure 3.11(c), at a CFRP strain of 2504 micro-strain (Table 3.2 and Figure 3.9(e)), which is only 69% of the rupture strain. This was thought to have occurred due to the increased flexural strength, which triggered shear cracks. Consequently, the shear cracks horizontal propagation at rebar level resulted in weakening the concrete and forcing delamination failure to occur at this horizontal plane.

(d) The fourth mode of failure was observed in specimen B7 with the mixed moduli CFRP, where failure occurred in three stages. First the tension steel yielded, followed by the ultra-high modulus strip rupturing at 3562 micro-strain (Table 3.2 and Figure 3.9(f)), and a subsequent drop in load. The load then increased again, followed by debonding of the high modulus strips at a higher load as shown in Figure 3.11(d), at 6045 micro-strain, 54% of the rupture strain.

### 3.4 Analytical Model

This section presents an analytical model that can be used to predict the performance of pre-cracked steel-reinforced concrete beams strengthened with longitudinal CFRP plates. The main purpose of presenting the model is the parametric study that would be carried out after validation, but the model itself is not particularly new. It employs the computer software VecTor2 and FormWorks for the damaging phase and Response 2000 for the CFRP retrofitting
phase. VecTor2 is a nonlinear finite element program for the analysis of two-dimensional reinforced concrete membrane structures, while FormWorks is a preprocessor software that generates input files for VecTor2 and is used to construct the finite element model (Vecchio and Wong 2002). Using the software, any reinforced concrete beam of a specific cross-section can be subjected to cyclic or monotonic loading and the performance can be outputted for every load increment. Therefore, the experimental damaging cycles can be simulated, outputting the load-deflection plots and the residual permanent deflection. In this phase of analysis there is no external CFRP applied to the cross-section.

The second phase involves analysis of the CFRP-retrofitted cross-section. This phase is simulated with Response 2000 (Bentz, 2000), in which CFRP can be indirectly modeled. CFRP was modeled using the linear part of the stress-strain curve of steel, with the appropriate Young’s modulus. As this program does not recognize the existing cracking, the initial stiffer response before cracking is omitted and the cracked stiffness is extended to the onset of reloading. The resulting load-deflection plot from this phase is then superimposed on the load-deflection plot of the first phase such that it starts from the point of permanent deflection upon unloading of the first phase.

The computer programs described above include the general reinforced concrete beam failure modes, namely steel yielding, concrete crushing and shear failure. The two additional failure criteria of CFRP rupture and CFRP debonding had to be added. CFRP rupture is enforced by terminating the sectional analysis once the rupture strain is achieved in the CFRP plate. CFRP debonding is detected once the CFRP strain reaches the value \( \varepsilon_{fd} \) specified by the ACI 440.2R-08 Guidelines (ACI 2008) for intermediate crack-induced debonding, given by:

\[
\varepsilon_{fd} = 0.41 \frac{f_{ct}}{nE_{rt_f}} \leq 0.9 \varepsilon_{fu}
\]  

(1)
where $E_f$ and $t_f$ are the elastic modulus and thickness of a single ply of the FRP laminate, $n$ is the number of plies, $f_{c'}$ is the concrete compressive strength, and $\varepsilon_{fu}$ is the ultimate tensile strain of the FRP laminate.

### 3.4.1 Model Validation

The load-deflection performances of the beams tested in this study have been determined using the model. Table 3.3 summarizes the measured and calculated yield and ultimate loads and Figure 3.12 shows the experimental and theoretical load-deflection curves. The results generally show a good correlation. The percent difference between the predicted and experimental values for the yield loads and ultimate loads range from 2 to 9% and from 1 to 14%, respectively. The model, however, underestimated the permanent deflection upon unloading in the elastic range in beams B2, B3 and B7. This resulted in a shift of the predicted response of the reloading to failure after repair. This problem was of a much lesser extent in beams B4, B5 and B6 unloaded from the plastic range. The theoretical failure modes show consistency with the experimental observations, except for specimen B6 where the model predicts shear failure and the experimental beam failed by concrete cover delamination. However, as indicated earlier, the two failure modes are related since the shear crack propagates horizontally at the rebar level, contributing to the concrete cover delamination.

### 3.4.2 Parametric Study

The model was then used in a parametric study conducted on a beam identical to that used in the experimental program, to investigate: (a) the effect of CFRP reinforcement ratio $\rho_f$ of the 400 GPa CFRP, ranging from 0.09% to 0.43%, and (b) various proportions of cross-sectional areas of the high- and ultra-high moduli of CFRP, 210 and 400 GPa, respectively, ranging from
1:0 to 0:1, for a constant reinforcement ratio of 0.34%. The beam is assumed to be cracked and unloaded in the elastic range, then repaired and reloaded to failure.

Figure 3.13(a) shows the load-deflection responses for beams with various $\rho_f$. The figure shows that as $\rho_f$ increases, the beam gains considerable stiffness and the ultimate flexural load capacity based on CFRP rupture increases, as expected. However, beyond a $\rho_f$ of 0.26%, the flexural strength exceeds the shear capacity, and therefore shear failure governs. Any increase in $\rho_f$ beyond this point would continue to enhance flexural stiffness but not the ultimate strength of the beam.

Figure 3.13(b) shows the load-deflection responses for beams with various proportions of CFRP moduli, including the two extreme cases of 100% of the 210 GPa and 100% of the 400 GPa. As observed experimentally earlier, the figure shows that the ultra-high modulus CFRP fails first by rupture, immediately followed by a load-drop then increase at a lower stiffness until the high modulus CFRP fails by debonding. In order for this concept of pseudo-ductility to be of practical value, the second load peak needs to be at least equal to or higher than the first peak. This is because if failure would occur in real life, it would be due to over loading (i.e. load control, not stroke control). Figure 3.13(b) shows that for this particular beam and $\rho_f$ of 0.34% used, this can be achieved using no more than 30% of the 400 GPa CFRP and not less than 70% of the 210 GPa CFRP. Higher percentages of the 400 GPa CFRP would lead to an ultimate load lower than the first peak load.
### Table 3.1 – Test Matrix

<table>
<thead>
<tr>
<th>Beam</th>
<th>Damage level relative to yield load $P_y$ and deflection $\Delta_s$ of B1</th>
<th>CFRP reinforcement ratio $\rho_f$ (%)</th>
<th>CFRP Young’s modulus $E_f$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>N/A (control)</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>B2</td>
<td>Elastic ($0.75P_y$)</td>
<td>0.17%</td>
<td>210</td>
</tr>
<tr>
<td>B3</td>
<td>Elastic ($0.75P_y$)</td>
<td>0.17%</td>
<td>400</td>
</tr>
<tr>
<td>B4</td>
<td>Plastic ($2\Delta_s$)</td>
<td>0.17%</td>
<td>210</td>
</tr>
<tr>
<td>B5</td>
<td>Plastic ($2\Delta_s$)</td>
<td>0.17%</td>
<td>400</td>
</tr>
<tr>
<td>B6</td>
<td>Plastic ($2\Delta_s$)</td>
<td>0.34%</td>
<td>400</td>
</tr>
<tr>
<td>B7</td>
<td>Elastic ($0.75P_y$)</td>
<td>0.34%</td>
<td>$0.8(210)+0.2(400)$</td>
</tr>
</tbody>
</table>

### Table 3.2 – Experimental Results

<table>
<thead>
<tr>
<th>Beam</th>
<th>Yield $P_y$ ($kN$)</th>
<th>% increase in $P_u$</th>
<th>Ultimate load $P_u$ ($kN$)</th>
<th>% increase in $P_u$</th>
<th>Max. CFRP strain ($\mu e$)</th>
<th>Permanent deflection after damage (mm)</th>
<th>Total deflection at ult. (mm)</th>
<th>Retrofit deflection (mm)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>75</td>
<td>N/A</td>
<td>90.5</td>
<td>N/A</td>
<td>N/A</td>
<td>154</td>
<td>N/A</td>
<td>Yielding $\rightarrow$ Concrete Crushing</td>
<td></td>
</tr>
<tr>
<td>B2</td>
<td>100</td>
<td>33</td>
<td>116.4</td>
<td>29</td>
<td>5512</td>
<td>2.52</td>
<td>23.25</td>
<td>20.73</td>
<td>Yielding $\rightarrow$ CFRP Debonding</td>
</tr>
<tr>
<td>B3</td>
<td>120</td>
<td>60</td>
<td>129.8</td>
<td>43</td>
<td>3167</td>
<td>2.52</td>
<td>18.77</td>
<td>16.25</td>
<td>Yielding $\rightarrow$ CFRP Debonding</td>
</tr>
<tr>
<td>B4</td>
<td>104</td>
<td>39</td>
<td>115.9</td>
<td>28</td>
<td>4432</td>
<td>15.56</td>
<td>34.44</td>
<td>18.88</td>
<td>Yielding $\rightarrow$ CFRP Debonding</td>
</tr>
<tr>
<td>B5</td>
<td>N/A</td>
<td>N/A</td>
<td>136.3</td>
<td>51</td>
<td>3594</td>
<td>15.31</td>
<td>33.22</td>
<td>17.91</td>
<td>CFRP Rupture</td>
</tr>
<tr>
<td>B6</td>
<td>N/A</td>
<td>N/A</td>
<td>147.8</td>
<td>63</td>
<td>2504</td>
<td>16.60</td>
<td>31.71</td>
<td>16.10</td>
<td>Cover Delamination</td>
</tr>
<tr>
<td>B7</td>
<td>117</td>
<td>56</td>
<td>130.1 &amp; 144.9</td>
<td>44 &amp; 60</td>
<td>3562 &amp; 6045</td>
<td>2.49</td>
<td>27.05</td>
<td>24.57</td>
<td>Yield $\rightarrow$ 400CFRP Rupture $\rightarrow$ 210CFRP Debonding</td>
</tr>
</tbody>
</table>

### Table 3.3 – Comparison between experimental and model prediction results

<table>
<thead>
<tr>
<th>Beam</th>
<th>$P_{y,exp}$ ($kN$)</th>
<th>$P_{y,model}$ ($kN$)</th>
<th>% difference</th>
<th>$P_{u,exp}$ ($kN$)</th>
<th>$P_{u,model}$ ($kN$)</th>
<th>% difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>B2</td>
<td>100</td>
<td>95</td>
<td>5.1</td>
<td>116</td>
<td>114</td>
<td>2.1</td>
</tr>
<tr>
<td>B3</td>
<td>120</td>
<td>115</td>
<td>4.3</td>
<td>130</td>
<td>122</td>
<td>6.2</td>
</tr>
<tr>
<td>B4</td>
<td>104</td>
<td>95</td>
<td>9.0</td>
<td>116</td>
<td>114</td>
<td>1.7</td>
</tr>
<tr>
<td>B5</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>136</td>
<td>122</td>
<td>11.1</td>
</tr>
<tr>
<td>B6</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>148</td>
<td>128</td>
<td>14.4</td>
</tr>
<tr>
<td>B7</td>
<td>117</td>
<td>120</td>
<td>2.5</td>
<td>145</td>
<td>130</td>
<td>10.9</td>
</tr>
</tbody>
</table>
Figure 3.1 – Schematic of test specimens.

Figure 3.2 – Stress-strain curves of CFRP and steel.

Figure 3.3 - Test Setup
Figure 3.4 – Effect of pre-repair extent of damage on strengthening performance.

Figure 3.5 – Effect of CFRP modulus on strengthening performance.
Figure 3.6 – Effect of hybrid modulus CFRP on performance.

Figure 3.7 – Effect of CFRP reinforcement ratio on performance.
Figure 3.8 – Load-strain responses of test beams.
Figure 3.9 – Strain distribution along the length of CFRP.
Figure 3.10 – Cracking patterns (dotted=pre repair, solid=post repair): (a) pre-yield damaged beam B2, (b) post-yield damaged beam B4, (c) post-yield damaged beam B6

Figure 3.11 – Failure modes
Figure 3.12 – Comparison of model predicted (black) and experimental (grey) results.
As $\rho_f$ increases, the ultimate load capacity increases until a transition point is reached where the flexural capacity is so strong that the failure mode changes from flexural to shear failure and there is a subsequent drop in load capacity.

A combination exists at approximately 70% high modulus and 30% ultrahigh modulus where the second peak no longer reaches a higher load than the first peak.

Figure 3.13 – Predictions using the analytical model.
Chapter 4: Analysis of Slender Reinforced Concrete Columns Strengthened Using Ultra-high Modulus CFRP Plates

4.1 Introduction

The purpose of this chapter is to investigate the performance of slender reinforced concrete columns strengthened with ultra-high modulus CFRP plates using an analytical model. Shaat and Fam (2009) were able to successfully apply this strengthening technique to slender steel columns and achieve significant strength gain, therefore it was encouraging to investigate the application to slender reinforced concrete columns. The additional challenges in this case are the presence of cracking in tension and concrete nonlinearity in compression, in addition to the typical second order effects and yielding of steel. Almost every effort to date in the area of FRP strengthening of concrete columns has focused primarily on confinement by transverse wrapping of the column. This chapter uses a completely different approach by applying longitudinal ‘very stiff’ CFRP reinforcement, to increase the flexural rigidity $EI$, thereby enhancing the axial load capacity and delaying over all buckling, according to Euler buckling theory. The proposed analysis modifies and combines various existing analytical models and tools in a unique way to address this rather complex problem and assess the effectiveness of this method of strengthening.

4.2 Analytical Model

This chapter introduces an analytical model that can be used to predict the performance of slender RC columns strengthened with longitudinal CFRP plates. The model consists of three steps. The first part employs the computer software Response 2000 to determine the resistance envelope represented by a cross-section interaction curve. In the second step, the loading path to failure of the particular column is determined using a modified equation from the ACI 318-08 that accounts for the contribution of the externally bonded high modulus CFRP plates. Finally
the third step turns the cross-section interaction curve established from the first step into a slender-column interaction curve to determine the resistance of a slender column without the additional second order effects. The model assumes that FRP longitudinal reinforcement is placed symmetrically on both extreme fibres of the column. The reason being, in cases of no zero end eccentricity, the slender column could buckle either side, depending on imperfections and out-of-straightness.

4.2.1 Cross-Section Interaction Curve

The cross-section load-moment interaction curve represents the capacity for the particular RC cross-section. To determine the interaction curve, this model employs the computer software Response 2000. Response 2000 allows analysis of reinforced concrete members subjected to arbitrary combinations of axial load, moment, and shear and is based on the Modified Compression Field Theory by Vecchio and Collins (1986). Two assumptions implicit within Response 2000 are that plane sections remain plane, and that there is no transverse clamping stress across the depth of the beam (Bentz, 2000). Using Response 2000, the cross-section interaction curve can be outputted for any particular cross-section. Wherever the failure path intersects the interaction curve determines the axial load at which material failure of the column will occur and the mode of failure. If the failure path reaches a peak and starts declining before reaching the interaction curve, a stability failure occurs as shown in Figure 4.1 (MacGregor and Wight 2012).

4.2.2 Using Response 2000 to determine the interaction curve

All column information such as cross-section, material data, and reinforcement details is inputted through a graphical interface. In the first step, the basic material properties are inputted and in later steps accurate material data can be inputted. Next, the geometry of the cross section,
the number and type of longitudinal bars, and the type and spacing of transverse steel is selected. This creates the entire cross section and shows it graphically on the screen.

In order to obtain an accurate analysis, proper material models are needed. For concrete, the program automatically assumes a stress-strain curve. The compression softening for normal strength concrete is modeled using the equation proposed by Vecchio and Collins (1986) and tension stiffening using the equations proposed by Bentz (1999). The steel material properties are defined by the elastic modulus, yield strength, strain at strain hardening, rupture strain, and ultimate strength.

Response 2000 does not have FRP as an available material. Therefore, FRP must be simulated by inputting its properties the same way as the steel reinforcement as shown in Figure 4.2. However, Bentz (2000) reports a problem with elastic-to-rupture materials in Response 2000. For materials that display linear elastic behaviour to the point of rupture, Response 2000 will produce very conservative results. The problem is that the program assumes that the strain at a crack must be able to be twice the average strain for the crack check (Bentz 2000). To account for this, Bentz (2000) advises to increase the strain at rupture for the material to twice the measured value and at the same ultimate stress. This gives the material a fake “yield plateau” that reaches to twice the yield strain as shown in Figure 4.3. Therefore, to properly simulate its behaviour, the FRP’s ultimate strength is inputted as both the yield and ultimate strength. The strain at rupture is inputted as the e-strain hardening and the rupture strain in Response 2000 is inputted as a value that is twice as large as the actual rupture strain. The moment-curvature response is then terminated manually once the end of the linear portion of the FRP stress-strain curve is reached. Figure 4.4 shows a screenshot of the program after the column’s cross-section and material properties have been inputted. The assumptions made in this part of the model are:
(a) The FRP is fully bonded to concrete and that debonding is unlikely to occur as in beams, given the high axial compression combined with relatively low bending, and (b) the stress-strain curve of FRP is the same in tension and compression. Although this may not be true, compression failure criterion is set to be by concrete crushing which is at a relatively low strain.

Using the software, the M-N interaction curve can be developed. The display from Response 2000 after solving for the interaction curve is shown in Figure 4.5. Response 2000 draws plots over the depth of the column. Response 2000 uses two control plots depending on the type of loading. In Figure 4.5 the control plots are the M-N interaction and moment curvature plots. At any particular failure point on the interaction curve, Response 2000 generates another nine plots.

**Cross Section:** Darker regions of the cross section indicate uncracked concrete. Longitudinal reinforcement and stirrups are drawn dark red if they’re on the yield plateau, bright red if strain hardening, and dark and bright green for yielding in compression.

**Longitudinal Strain:** The linear longitudinal strain confirms the assumption that plane sections remain plane. Maximum and minimum strain values can be obtained from these plots.

**Crack Diagram:** This plot shows the estimated crack pattern as well as the crack widths. For cases where part of the concrete is crushing, this section is drawn in pink.

**Longitudinal Reinforcement Stress:** This plot shows the average longitudinal stress in the reinforcement along the cross-section.

**Longitudinal Reinforcement Stress at a crack:** This local value includes the effects of the shear on the crack and principal tension.
**Longitudinal Concrete Stress**: This plot shows the stress in the concrete along the depth of the cross section.

**Internal Forces**: The Internal Forces plot shows the force and location of the compressive and tensile forces in the cross section. Note that the tensile force arrow may not come directly from the steel location due to the concrete tensile force component and the FRP.

**N+M**: The N+M plot shows the moment and axial force drawn simply as arrows.

Given these plots, the behaviour of the column along its interaction curve can be analyzed.

### 4.2.3 Slender Column Failure Path

To determine the failure path of slender columns, the design provisions of ACI 318-08 provide an approach referred to as the “Moment Magnification Procedure” to account for the secondary effects (ACI Committee 318, 2008). The method states that compression members shall be designed for a factored axial force $P_u$ and the factored moment amplified for the effects of member curvature $M_c$ as

$$M_c = \delta_{ns}M_2$$  \hspace{1cm} [1]

where $M_2$ is the end moment, the moment magnifier $\delta_{ns}$ is taken as

$$\delta_{ns} = \frac{c_m}{1-P_u/P_c} \geq 1.0$$  \hspace{1cm} [2]

and the critical buckling load $P_c$ is taken as

$$P_c = \frac{\pi^2EI}{(kt_u)^2}$$  \hspace{1cm} [3]
where $l_u$ is the unsupported length and $k$ is the effective length factor. The stiffness $EI$ shall be taken as

$$EI = \frac{(0.2E_c I_g + E_s I_{se})}{1 + \beta_{dns}}$$  \[4\]

where $E_c$ and $E_s$ are the modulus of elasticity of the concrete and steel, respectively, $I_g$ is the gross moment of inertia, $I_{se}$ is the moment of inertia of the steel, and $\beta_{dns}$ is the ratio of the maximum factored axial dead load to the total factored axial load.

**4.2.3.1 Contribution of FRP to Column Strength**

In order to account for the FRP contribution, modifying Equation [4] to include the FRP stiffness is proposed, as shown in the following equation:

$$EI = \frac{(0.2E_c I_g + E_s I_{se} + E_f I_f)}{1 + \beta_{dns}}$$  \[5\]

where $E_f$ and $I_f$ are the modulus of elasticity and moment of inertia of the FRP, respectively. It is assumed here that $E_f$ is equal in tension and compression. Using these equations, the loading path to failure on the interaction curve, as shown in Figure 4.1, for a particular column strengthened with FRP plates can be determined.

**4.2.4 Slender-Column Interaction Curve**

To better discuss the effects of variables on slender column strength, it is convenient to turn the cross-section interaction curves into slender-column interaction curves. Line $O-B_1$ in Figure 4.6 shows the load-maximum moment curve for a column of given slenderness and end eccentricity and is determined using the above equations. As stated previously, the column fails when the failure path intersects the cross-section interaction curve at $B_1$. At the time of failure, the load and moment at the end of the column are given by point $A_1$. Therefore, if this process is
repeated using the equations above and for different end eccentricities, the FRP strengthened slender-column interaction curve can be obtained. For the dimensions of the known column’s cross-section, the material properties, slenderness, and end eccentricity, the axial load at which the slender column will fail can be determined.

4.3 Verification of the Model

To verify the analytical model, the test data from Gajdosova and Bilcik’s (2013) experiments on full scale CFRP-strengthened slender RC columns was used. Gajdosova and Bilcik (2013) tested slender columns with the relatively new retrofit method of near surface mounted (NSM) longitudinal CFRP strips. The full scale specimens with rectangular cross-sections (210 x 150 mm) were tested to failure under eccentric compressive loading. The total lengths of the specimens were 4100 mm. They were symmetrically reinforced with 2 x 4 10 mm diameter longitudinal bars and stirrups of 6 mm diameter at a spacing of 150 mm were utilized. The columns were strengthened by the near surface mounted reinforcement method, in the form of CFRP strips (1.4 x 10 mm) mounted into grooves (three on each of the longer side of the cross-section) in the concrete cover. The CFRP has an ultimate strength and Young’s modulus of 2500 MPa and 168 GPa, respectively. Figure 4.7 outlines the details of the test setup, specimen geometry, and strengthening configuration.

The theoretical performance of these CFRP strengthened slender columns, including both the predicted interaction curve as well as the predicted loading path to failure, were determined using the procedures described in Section 4.2. Figure 4.8 compares the theoretical performance with the experimental loading path of the test performed by Gajdosova and Bilcik (2013). The experimental column failed at an axial load of 321 kN while the model predicted a failure load of 255 kN; a 21% difference. Therefore, the model while not quite accurate predicted conservative
results. Gajdosova and Bilcik (2013) reported that the CFRP strengthened slender columns tested in the experimental investigation failed in buckling. This can be seen in Figure 4.8 where the tangent of the experimental failure path turns horizontal just before it reaches the interaction curve, indicating a stability failure. On the other hand, although the theoretical failure path comes very close to horizontal, the model still predicts a material failure where the tension steel yields and the concrete crushes, before tensile rupture of the CFRP.

The inaccuracy of the proposed model could be due to numerous reasons. The model is a very simplified approach where the stiffness of the FRP is merely added to Equation 4 to produce Equation 5. However, Equation 4 is based only on the relations between concrete and steel. Therefore, to simply include the stiffness of the FRP, as done in Equation 5, neglects the impact the FRP may have on the rest of the equation; the relationship between the concrete, steel, and FRP may be different. Therefore, further validation for the proposed model is needed to adequately assess the model’s accuracy. Limited validation data is available, however, and is the reason only one set of test data was used in this thesis.

4.4 Parametric Study

The predicted performance of RC columns of varying slenderness ratios strengthened with different amounts of ultra-high modulus CFRP strips (400 GPa) were determined using the above procedures. Variables kept constant in this study included the cross-section dimensions of the column, internal longitudinal steel reinforcement, the transverse reinforcement, the concrete strength, and the eccentricity of the applied load. The cross-sections of the columns analyzed had a consistent depth and width of 150 mm and 300 mm, respectively. The longitudinal reinforcement consisted of four 15M steel rebar with clear concrete covers of 16 mm. The transverse reinforcement of each specimen included 5 mm diameter ties spaced at 150mm and
75mm. The concrete strength was 50 MPa and the axial load was applied at an eccentricity of 19.5 mm, which is the minimum value according to the ACI 318-08. The parameters analyzed in this study included:

(a) the slenderness ratio or column length,

(b) the FRP reinforcement ratio, and

(c) the effective length factor $k$.

Table 4.1 summarizes the test matrix, parameters, and resulting maximum predicted axial loads and failure modes. The cross-section interaction curves obtained from Response 2000 with the slender column failure paths can be found in Appendix A.

Figure 4.9 shows the original cross-section interaction curves with the load paths of slender columns of lengths 2000 mm, 3000 mm, and 4000 mm. Also shown on Figure 4.9 are the corresponding slender column interaction curves for the particular column length and the method used for developing them. As can be seen in the figure, the peak loads based on the failure load path and slender interaction curves are the same.

The effect of the slenderness ratio on performance is shown in Figures 4.10, 4.11, and 4.12. Figure 4.10 presents the slender-column interaction curves of strengthened columns of three different slender lengths (2000 mm, 3000 mm, and 4000 mm) and compares them to their unstrengthened curves. The strengthened columns in Figure 4.10 have FRP reinforcement ratios of 0.39%. Figure 4.11 presents the short-column interaction curve for strengthened columns of short length 990 mm. As you can see in both Figure 4.10 and 4.11, the strengthened columns have a greater capacity for axial load and moment compared to the unstrengthened columns and the increase in relative strength is more prominent in the longer columns. Figure 4.12 shows the
variation of the percentage increase in axial strength with length or slenderness ratio. The figure shows increases of 7.7%, 19.6%, 32.1%, and 36.4% for columns with slenderness ratios of 22, 44, 67, and 89 (990 mm, 2000 mm, 3000 mm, and 4000 mm) respectively and FRP reinforcement ratios of 0.39%. This strengthening technique has been proven to be very effective at resisting bending moments as in beams. Therefore, as the slenderness ratio increases, the secondary moment increases and this strengthening technique becomes more effective.

The effect of the FRP reinforcement ratio on performance is shown in Figure 4.13. Figure 4.13a presents the short-column interaction curve of strengthened short columns with four different FRP reinforcement ratios (0.39%, 0.77%, 1.16%, and 1.54%) all of length 990 mm. Figure 4.13b-d present the slender-column interaction curves of strengthened slender columns with four different FRP reinforcement ratios (0.39%, 0.77%, 1.16%, and 1.54%) and three different column lengths (2000 mm, 3000 mm, and 4000 mm). Figure 4.14 shows the variation of the percentage increase in axial strength with FRP reinforcement ratio. As expected, the model predicts the effectiveness of the FRP strengthening system increases as the FRP reinforcement ratio increases. In addition, Figure 4.14 shows that the FRP strengthening system has a much greater effect on the slender columns (2000 mm, 3000 mm, and 4000 mm) as opposed to the short column (990 mm). Also indicated in the longer slender column curves (3000 mm and 4000 mm) shown in Figures 4.13c and d, is a critical point where the column reaches its buckling load and its failure mode changes from a material to stability failure. Therefore, for smaller loading eccentricities the maximum axial load will be limited by stability failure.
Table 4.1. Summary of predicted results.

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<th>Set</th>
<th>Col.</th>
<th>L (mm)</th>
<th>kl/r</th>
<th>ρf (%)</th>
<th>E_f (GPa)</th>
<th>k</th>
<th>Predicted P_max (kN)</th>
<th>%age inc. in P_u</th>
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Figure 4.1 – Material and stability failures (MacGregor and Wight 2012).

Figure 4.2 - Materials input page.
Figure 4.3 – Fake FRP stress-strain curve inputted into Response 2000.

Figure 4.4 – Input screenshot.
Figure 4.5 – Sectional Analysis.

Figure 4.6 – Slender column interaction curve (MacGregor and Wight 2012).
Figure 4.7 – Test setup, specimen geometry, and strengthening configuration from full scale tests by Gajdosova and Bilcik (2013)

Figure 4.8 – Cross-section interaction curve with the experimental (Gajdosova and Bilcik 2013) and predicted slender column failure paths.
Figure 4.9 – Cross-section interaction curves and the corresponding slender interaction curves.
Figure 4.10 – Slender interaction curves for unstrengthened and strengthened columns of length 2000 mm, 3000 mm, and 4000 mm.

Figure 4.11 – Short interaction curve for unstrengthened and strengthened columns of length 990 mm.
The model predicts the effectiveness of the CFRP system will increase considerably as slenderness ratios (kL/r) become higher.

Figure 4.12 – Effect of slenderness on performance.
Figure 4.13 – Short (990 mm) and slender (2000 mm, 3000 mm, and 4000 mm) interaction curves for strengthened columns of different FRP reinforcement ratios.
Figure 4.14 – Effect of FRP reinforcement ratio on performance.
Chapter 5: Conclusions

5.1 Performance of damaged reinforced concrete beams strengthened using ultra-high-modulus CFRP strips

In this study, steel-reinforced concrete beams (150 x 300 x 3000 mm) were pre-cracked and strengthened with high and ultra-high modulus CFRP plates and tested under four-point loading. Generally, the study showed great potential for this technique. An analytical model has been developed by using the computer software VecTor, FormWorks, and Response 2000 to predict the load deflection performance during the cracking and retrofitted phases. The following conclusions are drawn from the experimental and analytical investigations:

1. The level of pre-existing damage has insignificant effect on strengthening effectiveness and failure mode at ultimate; however, CFRP modulus has a significant effect. The 210 and 400 GPa CFRP of reinforcement ratio $\rho_f = 0.17\%$ increased ultimate strength by 29 and 43%, respectively, for beams pre-cracked in the elastic range and by 28 and 51% for beams overloaded to the plastic range.

2. The better ultimate strength enhancement of 400 GPa CFRP, despite its 40% lower tensile strength than the 210 GPa CFRP, is attributed to the change in failure mode to CFRP rupture, rather than premature debonding.

3. The 210 GPa CFRP increased yield load ($P_y$) by 33 and 39% for beams pre-cracked at the elastic and plastic range, respectively. However, the 400 GPa CFRP increased $P_y$ by 60% at elastic range but did not reach the upgraded $P_y$ at the plastic range.

4. The 400 GPa CFRP with $\rho_f = 0.17\%$ enhanced flexural stiffness in the elastic and plastic ranges by only 8% and 12%, respectively, which is only slightly better than the 210 GPa CFRP.
5. Doubling the $\rho_f$ of the 400 GPa CFRP from 0.17 to 0.34% resulted in only an 8% increase in ultimate strength (but to an overall gain of 63%); due to change in failure mode from rupture to concrete cover delamination. The stiffness increased by 19%.

6. The beam retrofitted by hybrid modulus CFRP of 80% 210 GPa and 20% 400 GPa of the $\rho_f = 0.34\%$ showed remarkable pseudo ductility and warning signs before ultimate failure. The 400 GPa CFRP ruptured at a load representing 44% strength gain, followed by reduction in stiffness until the 210 GPa debonded at a load representing 60% strength gain.

7. There is a critical balance in proportioning the areas of hybrid moduli CFRP to achieve reliable pseudo ductility. In the beam with $\rho_f = 0.34\%$, this was achieved using a maximum of 30% $\rho_f$ of the 400 GPa CFRP. Any higher percentage would lead to a final ultimate load lower than the first failure peak load.

5.2 Analytical model of slender reinforced concrete columns strengthened using ultra-high modulus CFRP plates

In this study, an analytical model was developed by modifying the provisions of the American Concrete Institute (ACI 2008) and employing the computer software Response 2000 to predict the performance of slender RC columns. The following conclusions are drawn:

1) An analytical model to predict the performance of FRP strengthened slender RC columns is developed by: a) using Response 2000 to find the cross-section interaction curve, b) modifying the ACI 318-08 provisions to get the slender column failure path, and c) converting the cross-section or short column interaction curve into a slender column interaction curve.

2) The model predicts that the effectiveness of the FRP strengthening system increases substantially as the slenderness ratio increases.
3) The model predicts that as the FRP reinforcement ratio increases, the effectiveness of the FRP strengthening system will increase, more so for higher slenderness ratios.

4) The model shows that at a greater slenderness ratio and a smaller loading eccentricity, stability failure or column buckling may limit the maximum axial load.

5.3 Future Research

Future research should be conducted to support the conclusions reached in these tests and verify the analytical model developed to predict the performance of pre-cracked and yielded reinforced concrete beams strengthened with ultra-high modulus FRP plates. FRP plates with a wider variety of elastic modulus, thickness, and bond length should be tested to establish a design procedure for a wide range of damaged beams. In addition, FRP strengthened beams with varying steel reinforcement ratios should also be investigated. Continued studies investigating the concept of hybrid moduli plates should also be conducted to further develop the idea and overcome FRP’s inherent disadvantage of failing abruptly without warning.

Experimental tests should be conducted to continue the research on the application of FRP plates to strengthen slender reinforced concrete columns and verify the analytical model developed in this paper. Possible parameters in future experimental tests should include: slenderness ratio, FRP reinforcement ratio, bond length, end conditions and effective length factor. Furthermore, a wider variety of FRP reinforcement types and configurations such as FRP jackets, Near-surface mounted (NSM) strips or NSM bars should also be investigated.
References

ACI Committee 318 (2008). “Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (318R-08).” Farmington Hills: American Concrete Institute.


References


Appendix A: Extra Figures for Chapter 4

Figure A.1 – Cross-section interaction curve for unstrengthened and strengthened columns of length 2000 mm with the slender column failure paths.
Figure A.2 – Cross-section interaction curve for unstrengthened and strengthened columns of length 3000 mm with the slender column failure paths.

Figure A.3 – Cross-section interaction curve for unstrengthened and strengthened columns of length 4000 mm with the slender column failure paths.