ABSTRACT

MIELKE, BRIAN RICHARD. Effective Carbon Fiber Reinforced Polymer Strengthening System for Reinforced Concrete Structures. (Under the direction of Dr. Sami H. Rizkalla).

For the last three decades, fiber reinforced polymer (FRP) material has been used for the repair and strengthening of the country’s failing infrastructure. Even though this technology has been used over these decades for civil infrastructure application, no codes have been developed in the United States, only guidelines, for the use of FRP materials. This technology is designed to offer an economical alternative to the replacement of damaged or deficient structures. These systems are used to increase the capacity of those structures whose current service loads exceed their design load or to repair the damaged ones. The research program reported in this thesis investigates the effectiveness of a new carbon fiber reinforced polymer (CFRP) strengthening system for concrete structures. This program looks to evaluate the effectiveness of the proposed system as affected by the various parameters believed to affect the behavior. These factors include the concrete strength of the existing structures, the fiber reinforcement ratio, the use of an innovative anchorage system, and the effectiveness of using bi-axial fibers versus using uni-directional fibers. The research investigated flexural strengthening of reinforced concrete slabs and beams as well as shear strengthening of beams using a mechanical anchorage system. The research also investigated using the new material for strengthening axially loaded members. The research thoroughly investigated the mechanical properties of the new CFRP material which was used to examine the applicability of the current codes for these new materials. Test results indicate that the proposed CFRP strengthening system provides an increase of the load carrying capacity of the reinforced concrete members strengthened for flexure, shear, or axial compression. Test results also show that, in general, increasing the fiber reinforcement ratio will increases the load carrying capacity of the reinforced concrete members, however, reducing the overall ductility of the member. Test results also indicate that, in general, the proposed CFRP strengthening system is not as effective when it is used for axially loaded high strength
concrete members. Research findings indicate that the use of the specially designed anchorage system for the CFRP U-wraps is extremely effective in increasing the shear capacity of reinforced concrete members. Test results highlight the influence of proper installation of the system as well.
Effective Carbon Fiber Reinforced Polymer Strengthening System for Reinforced Concrete Structures

by
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A thesis submitted to the Graduate Faculty of North Carolina State University in partial fulfillment of the requirements for the degree of Master of Science

Civil Engineering

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Chair of Advisory Committee
DEDICATION

This thesis is dedicated to my parents. Without their unending love, support, and encouragement I would not be the man I am today.
BIOGRAPHY

Brian Richard Mielke was born in Anchorage, Alaska and raised in Raleigh, North Carolina, which he has called home for over twenty years. He began his collegiate career at North Carolina State University, where he graduated Magna Cum Laude in Civil Engineering in 2010. Upon completion of his Bachelor of Science degree, he enrolled in the Structural Engineering and Mechanics Graduate program at North Carolina State University where he worked toward his Master of Science degree under the guidance of Dr. Sami Rizkalla. Upon completion, he intends to pursue a career in structural engineering.
ACKNOWLEDGMENTS

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"It is not the critic who counts, not the man who points out how the strong man stumbled, or where the doer of deeds could have done better. The credit belongs to the man who is actually in the arena, whose face is marred by dust and sweat and blood, who strives valiantly, who errs and comes short again and again, who knows the great enthusiasms, the great devotions, and spends himself in a worthy cause, who at best knows achievement and who at the worst if he fails at least fails while daring greatly so that his place shall never be with those cold and timid souls who know neither victory nor defeat."

-Theodore Roosevelt
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1. INTRODUCTION

1.1. Background
There is an urgent need to develop and implement new technology to repair the country’s failing infrastructure. One of the proposed technologies is the use of Fiber Reinforced Polymer (FRP) materials. These relatively new materials can be used to provide an economical and effective repair and strengthening system alternative to replacing damaged or deficient structures. These systems can be used to increase the capacity of structures when the demand to increase the service loads is required or to repair damage of the structures. The research program reported in this thesis investigates the effectiveness of a specific strengthening system using a specific carbon fiber reinforced polymer material for concrete structures. Research findings will complement updating the current design guidelines published by the American Concrete Institute for FRP Strengthening Systems (ACI 440.2R-08). The system under consideration is designed to strengthen existing concrete structural elements to increase their overall load capacity. Different properties that are evaluated during the research include flexural, shear, and axial compression capacities and structural design procedures. This research program consists of three phases; physical and mechanical properties of the FRP material, testing of members in flexure and shear, and behavior of columns strengthened with the proposed system.

1.2. Objectives
The main objective of this research program is to investigate the effectiveness of the proposed carbon fiber reinforced polymer (CFRP) strengthening system. The system’s effectiveness as a strengthening system was evaluated on the following studies that were conducted:

1) Physical and mechanical properties. The research includes standardized test to determine the physical and mechanical properties of the CFRP material under
consideration. The properties include tensile strength, tensile ultimate strain, and tensile modulus.

2) Reinforced concrete members tested in flexure and shear. The experimental program includes eighteen (18) beams and slabs. The beams share T-shaped cross-section and are designed to fail in both flexure and shear modes while the slabs with a rectangular cross-section are designed to fail in a flexural mode only. Two different concrete strengths and two different strengthening systems are included in the program for both shear and flexure. All specimens were tested in 4-point bending.

3) Reinforced concrete column tested under axial loads. The experimental program includes six (6) columns reinforced with the proposed CFRP strengthening system subjected to axial compression. Varying concrete strength, as well as different strengthening systems was studied. Steel collars were used to prevent premature crushing at the column ends and were cast in place with each specimen to avoid localized failure.

1.3. Organization of Thesis

This thesis consists of six chapters. In addition to this introductory chapter, Chapter 2 presents the background information necessary to understand the objective of this research project. This chapter includes an overview and history of the use of FRP materials in flexural, shear, and axial compression strengthening of reinforced concrete structures. The chapter also introduces different models that were used in predicting the ultimate load carrying capacity of the FRP strengthened reinforced concrete specimens tested in this project.

Chapter 3 presents the experimental program that was implemented for this study. This chapter describes the different reinforced concrete structures that were tested in this program as well as the different FRP strengthening systems used in this study. It also gives details of the test set-up for the reinforced concrete structures and the instrumentation used to capture the behavior of these structures.
Chapter 4 summarizes the material properties of the concrete, reinforcing steel, and CFRP material used in this study.

Chapter 5 reports the test results of the reinforced concrete structures for this study. Conclusions are drawn from these results on the effect of the different CFRP strengthening system on the behavior of reinforced concrete structures.

Chapter 6 summarizes the research findings, presents the conclusions drawn from the study, and presents recommendations for possible future research.
2. LITERATURE REVIEW

2.1. Introduction

The recent Report Card for America’s Infrastructure, published by the American Society of Civil Engineers (ASCE), gave an overall D grade to America’s infrastructure. The World Economic Forum’s 2011 report on national infrastructure dropped the rank of United States’ infrastructure to No. 16 in the world, down from No.6 in 2007. These problems have developed due to structural damage, aging of the infrastructure, and the presence of service loads greater than their original design load carrying capacity. Therefore, the need for an economical and effective solution for the repair and rehabilitation of America’s infrastructure is urgently needed. This chapter briefly summarizes the literature on the use of fiber reinforced polymers (FRP) as construction materials for strengthening of reinforced concrete structures.

The old method used to repair and rehabilitate civil engineering structures was bonding steel plates using epoxy adhesives. This method has proven to be an effective method for repair, but has many drawbacks. Some of these problems are corrosion of the steel, handling due to excessive size and weight, undesirable formation of welds, partial composite action with the concrete, and debonding (Alagusundaramoorthy, Harik, & Choo, 2003). Due to problems associated with using steel plates, there was a call for a more effective strengthening and repair system. FRPs are composite material made up of fibers embedded into an epoxy coating. The most common type of fibers used in civil engineering is aramid, glass, and carbon. The epoxy typically consists of a two part system, a resin and a hardener material. This FRP system is often applied to the prepared surface of the tension zones of the reinforced concrete structure as a repair or strengthening system. The development of FRP materials has yielded several advantages in comparison to the use of steel plates. Some of these advantages include a high strength-to-weight ratio, resistance to electro-chemical corrosion, good fatigue strength, and potential for decreased installation costs and repairs due to lower weight in comparison with steel (Alagusundaramoorthy, Harik, & Choo, 2003). Use of FRP composites for strengthening has shown significant improvement of reinforced
concrete structures in terms of strength, stiffness, and overall performance (Said & Wu, 2008).

2.2. Flexural Strengthening

There are two main techniques used in the flexural strengthening of reinforced concrete structures, externally bonded (EB) and near-surface mounted (NSM). The research work reported in this thesis investigates the use of FRP material as an externally bonded system only. The externally bonded technique is also commonly referred to as wet lay-up; which involves the application of the FRP sheet or grid saturated into resin and attached to the surface of a reinforced concrete structure. An example of a wet lay-up application is shown in Figure 2-1.

![Figure 2-1 Wet Lay-Up Application](image)

When the wet lay-up technique is used, it is important that the concrete surface is prepared prior to application of the FRP and resin to achieve excellent bond. The preparation of the surface typically includes roughing the surface with a diamond-disk grinder or sand-blasting to expose the aggregate and to ensure that there will be a good bond between the FRP
strengthening system and structure. An example of a prepared surface prior to the installation of the FRP strengthening system is shown in Figure 2-2.

![Figure 2-2 Prepared Concrete Surface](image)

It is critical that a good bond is established between the FRP strengthening system and the concrete surface since this technique is considered to be a bond-critical application. A poor bond can lead to premature failures, such as debonding, that can significantly limit the effectiveness of the system. It can also lead to possible brittle failure modes that do not achieve the full strengthening of the FRP system (Carvalho, Chastre, Biscaia, & Paula, 2010).

2.3. Flexure Failure Mechanisms

There are several different failure modes for reinforced concrete structures strengthened in flexure with a FRP system. Reinforced concrete structures not strengthened with an FRP system normally fail in a ductile manner due to yielding of the reinforcing steel followed by crushing of the concrete in the compression zone. The FRP strengthened system is fully utilized when the fibers of the FRP strengthening system rupture prior to failure of the reinforced concrete structure. In this case, the full strength of the strengthening system
would have been developed followed by a ductile failure mode of the reinforced concrete structure. Due to interfacial shear stresses between the FRP strengthening system and the concrete though, a possible failure mechanism could occur due to debonding or delamination of the FRP sheets. Debonding of the strengthening system is characterized as the separation of the cover concrete at the level of tensile steel reinforcement with the FRP system at failure while delamination is characterized by the peeling off of the FRP strengthening system with only a thin layer of mortar paste from the surface of the concrete structure (Gao, Leung, & Kim, 2004). Research has found that high strength concrete may enhance the probability of FRP delamination due to the appearance of more sudden and larger cracks (Arduini, Nanni, & Romagnolo, 2004). From this point forward, all failure mechanisms in which the FRP strengthening system separated from the structure will be referred as a “debonding failure”, regardless of the amount of concrete that broke away with it. Many types of debonding have been studied and will be discussed in this section and are not a desirable failure mode since the full strength of the FRP composite is not developed and often leads to a brittle failure (Ceroni, 2010). These debonding mechanisms can be grouped into three main categories, intermediate-crack debonding, plate-end debonding, and critical diagonal crack debonding. The prominent failure mode seen in this thesis was intermediate crack debonding and therefore will be investigated in more depth than the other two debonding mechanisms.

2.3.1. Plate-End Debonding

Plate-end debonding failure involves the separation of the FRP plate (or sheet) from the concrete surface near the end of the FRP. This type of debonding failure is the most common type of debonding mechanism present and a number of comprehensive studies have been conducted on this type of failure (Teng, Smith, Yao, & Chen, 2003). A sketch of a typical plate-end debonding failure is shown in Figure 2-3. There are two common types of plate-end debonding. One type involves concrete cover separation at the level of tensile steel reinforcement, often due to large curvature of the reinforced concrete structure at failure. The other type of plate-end debonding, as shown in Figure 2-3, involves the delamination of
the FRP composite at the end of the sheet or plate with no concrete cover separation due to large interfacial shear stresses between the strengthening system and the concrete surface.

![Figure 2-3 Plate-End Debonding Failure (Teng, Smith, Yao, & Chen, 2003)](image)

The current analytical approaches to this type of failure are based on nonlinear fracture mechanics. The maximum force at debonding is a function of the mechanical/geometrical properties of the composite and of the Mode II fracture energy of the interface between concrete and FRP (Mazzotti & Savoia, 2009). Mode II fracture energy is associated with the in-plane shear sliding of the concrete structure under loading. This fracture may occur within the interfacial concrete or through the adhesive layer (Said & Wu, 2008).

2.3.2. Critical Diagonal Crack (CDC) Debonding

Critical Diagonal Crack debonding is associated with the rigid body displacement induced by the formation of a critical diagonal crack. This diagonal crack extends virtually through the depth of the beam or slab, from which rigid body displacement in the form of sliding and/or rotation will occur. This type of failure is governed by the shear capacity of the beam or slab without stirrups. A sketch of a typical critical diagonal crack failure is shown in Figure 2-4, where the debonding starts at the root of the critical diagonal shear crack and propagates towards the end of the FRP strengthening system (Oehlers & Seracino, 2004).
The failure mode for this debonding mechanism is very brittle because it is closely related to shear failure and often does not allow the stirrups to be fully engaged, therefore lowering the overall shear capacity of the structure.

2.3.3. **Intermediate Crack (IC) Debonding**

Intermediate crack, IC, debonding is the critical form of FRP debonding and therefore is often referred to as the “dominant” form of debonding. IC debonding governs the increase in moment capacity of the structure, governs the ductility of the strengthened section, and is included in the CDC debonding mechanism. In this thesis, the majority of debonding failures observed were attributed to IC debonding. IC debonding usually occurs within the shear span of the beam, initiation at the end of dominant flexural or flexural-shear crack with a high moment-to-shear ratio, and propagates along the direction of decreasing moment (Said & Wu, 2008). A sketch depicting a typical IC debonding failure is shown in Figure 2-5.

![Figure 2-4 Critical Diagonal Crack Failure (Teng, Smith, Yao, & Chen, 2003)](image-url)
There are many analytical models proposed to determine the loads that will lead to IC debonding. However, this type of debonding mechanism is much more complex than the other two debonding mechanisms and there is not a generally accepted theory to describe the failure mechanism (Mazzotti & Savoia, 2009). However, there have been two main approaches taken to detect the occurrence of IC debonding (Mazzotti & Savoia, 2009). One approach taken is to define a value of maximum strain or stress in the FRP reinforcement corresponding to IC debonding. The other approach considers IC debonding to occur when the unbalanced shear force reaches the shear strength between two cracks and is strictly related with the crack spacing between two cracks (Mazzotti & Savoia, 2009). A recent study (Said & Wu, 2008) compiled the major IC debonding guidelines and models for comparison and proposed a failure mechanism model. Some of these models and codes will be presented and used in the evaluation of the beams and slabs tested in this thesis that had an IC debonding failure. The following section discusses selected models that are generally accepted and addresses the varying categories of IC debonding.

2.3.3.1. Japan Society of Civil Engineers (2001) Recommendations

This model was adopted by JSCE (2001) based on the study conducted by Wu and Niu (2000, 2007). It suggests that no debonding of the continuous fiber sheet occurs when the stress, $\sigma_f$, of the continuous fiber sheet at the location of flexural cracking caused by the maximum bending moment in the member satisfies Eq. (1) (Said & Wu, 2008).
\[ \sigma_f \leq \sqrt{2G_f E_f / t_f} \]  \hspace{1cm} \text{Eq. (1)}

where

\[ G_f = 0.644 f'_c^{0.19} \]

where

\( \sigma_f \) = tensile stress of FRP

\( G_f \) = interfacial fracture energy

\( E_f \) = tensile modulus of elasticity of FRP

\( t_f \) = nominal thickness of the FRP sheet

\( f'_c \) = concrete compressive strength

\textbf{2.3.3.2. ACI 440.2R-08}

This model limits the allowable strain in the FRP composites, \( \varepsilon_{fd} \), to prevent IC induced debonding. It was modified from the model proposed by Teng et al. (2001, 2004) and calibrated using average measured values of FRP strains at debonding (ACI 440.2R-08). The strain is limited by Eq. (2).

\[ \varepsilon_{fd} = 0.41 \times \sqrt{\frac{f'_c}{nE_f}} \leq 0.9 \varepsilon_{fu} \]  \hspace{1cm} \text{Eq. (2)}

where

\( \varepsilon_{fd} \) = debonding strain of externally bonded FRP

\( f'_c \) = concrete compressive strength

\( n \) = number of plies of FRP

\( E_f \) = tensile modulus of elasticity of FRP

\( t_f \) = nominal thickness of the FRP sheet
2.3.3.3. Said and Wu (2008)

The model is empirically developed and calibrated against a large database of experimental results that limits the strain in the FRP, $\varepsilon_{deb}$, to avoid premature IC debonding (Said & Wu, 2008). The limiting strain for the Said and Wu formula is given in Eq. (3).

$$
\varepsilon_{deb} = 0.23\left(\frac{f'_c}{E_f t_f}\right)^{0.2} / (E_f t_f)^{0.35}
$$

where

- $f'_c$ = concrete compressive strength
- $E_f$ = tensile modulus of elasticity of FRP
- $t_f$ = nominal thickness of the FRP sheet

<table>
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<th>Eq. (3)</th>
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2.4. Shear Strengthening

The traditional method to increase the shear capacity of reinforced concrete structures was to bond steel plates to the side of structures. These steel plates were used to help to provide resistance and therefore stop the propagation of critical shear cracks. There were many problems associated with this strengthening technique however, including corrosion of steel plates, handling issues due to the heavy weight of the steel plates, and debonding. Therefore, a new reliable and effective shear strengthening technique was explored. The use of CFRP laminates and sheets externally bonded to the reinforced concrete structure was introduced to replace the steel plates. There are three main techniques used to shear strengthen reinforced concrete structures with externally bonded FRP (ACI 440.2R-08). The first technique used in strengthening is to completely wrap the reinforced concrete structure with a FRP system. This shear strengthening technique is the most efficient method since it completely confines the structure and prevents debonding from the concrete surface. It is commonly used for reinforced concrete columns since all sides can be easily wrapped; however it is not practical for typical beams. The second technique used in shear strengthening of reinforced concrete structures is placing the FRP strengthening system on two sides of the structure. This method is commonly used for beams that have only two or three sides exposed for shear
strengthening. This may be the least efficient method for shear strengthening. The third technique of shear strengthening is three sided U-wrapping of the reinforced concrete structure. This method is commonly used for beams that have an integral slab; therefore it is impractical to completely wrap the section. It is more efficient than bonding the FRP strengthening system on two sides but not as efficient as completely wrapping the structure. The research work reported in this thesis investigates the use FRP U-wrap as a shear strengthening system only. An example of shear strengthening using FRP U-wraps is shown in Figure 2-6.

A recent development in shear strengthening of reinforced concrete structures has been to use FRP anchorage with the traditional FRP U-wrap strengthening system. These FRP anchors are used to distribute the stress from the ends of the U-wraps, where high stress concentrations may lead to debonding, back into the structure to the compression zone of the beam. This provides equilibrium of the truss mechanism typically formed within the flexural members to resist the shear. The anchorage system is expected to provide a more efficient FRP U-wrap shear strengthening system that can achieve a higher load carrying capacity. An example of shear FRP anchors are shown in Figure 2-7. The combined FRP U-wrap and shear FRP anchorage system is shown in Figure 2-8.
Figure 2-7 FRP Shear Anchor System

Figure 2-8 Anchored U-Wrap Shear Strengthening System
2.4.1. **Shear Failure Models**

Due to the complexity of the behavior of reinforced concrete structures with the application of shear strengthening, research has been conducted to examine different failure modes that are present in FRP shear strengthening of reinforced concrete structures (Adhikary, Mutsuyoshi, & Ashraf, 2004; Bencardino, Colotti, Spadea, & Swamy, 2005; Pellegrino & Modena, 2006; Chen, Teng, & Chen, 2010; Mofidi & Chaallal, 2011). The two main failure mechanisms experienced in reinforced concrete structures that are strengthened in shear with an FRP system are rupture or debonding. Rupture of the FRP system is the common failure mode for those structures that are completely wrapped because there are no weak edges of the FRP where high stress concentration could lead to debonding. The most common failure mechanism for structures strengthened with a FRP U-wrap system is debonding, normally near the edge of the wrap. Debonding typically occurs when diagonal shear cracks form and propagate such that the concrete can no longer provide shear resistance. At this point, the resistance to shear at these locations is provided only by the FRP reinforcement, leading to high tensile stresses to form in the FRP. These stresses are commonly referred to as interfacial bond stresses, and occur to transfer tensile stresses at the concrete to each side of the shear crack (Adhikary, Mutsuyoshi, & Ashraf, 2004). The interfacial bond stresses that develop can cause the premature debonding of the FRP strengthening system prior to rupture of the FRP.

There are many factors that have been identified as key contributors to the initiation of debonding of FRP in shear (Pvellegrino & Modena, 2006; Mofidi & Chaalla, 2011). One of the factors that influence the contribution of the FRP to shear resistance is the bond model used in modeling the shear distribution in the FRP system. The bond models are used to determine the bond strength that is developed in the FRP that will resist the shear stresses developed in the reinforced concrete member. Many different bond models have been proposed over the years and are often based on the maximum effective length of the FRP U-wrap and the maximum stress that can be developed in the FRP before debonding (Mofidi & Chaallal, 2011). These factors contribute to the shear-slip resistance of the FRP.
strengthening system. Chen, Teng, and Chen (2005) recently proposed a modified shear strength model that is a more realistic representation of the bond area. The bond area is trapezoidal in shape which includes a distance in the compression zone above the crack tip and a distance in the tension zone below the steel reinforcement (Chen, Teng, & Chen 2005). Another factor that influences the contribution of the FRP to shear resistance is the formulation and limitation of the effective strain in the FRP system. This factor often limits the maximum strain in the FRP system below the tensile rupture strain of the FRP, to account for the likelihood of debonding prior to rupturing of the fibers, which is normally the case for members that are not fully wrapped. The effective strain in the FRP is often determined using a type of truss model, with the force in the FRP being equal to the maximum bonding force between the FRP and concrete (Mofidi & Chaallal, 2011). The maximum bonding force is limited by the effective strain in the FRP at debonding. The truss models used in determining the effective strain in the FRP introduce another factor that influences the contribution of the FRP to shear resistance, the crack angle. In most design codes, the assumed crack angle is 45 degrees. This has proven to be a conservative estimation of the angle at which the critical shear crack will propagate from the load point through the member to the support. This angle determines the number of FRP wraps that will be intersected by the critical shear crack, which therefore will contribute to the shear resistance. If this angle is decreased, more FRP wraps will be intersected by the critical shear crack, and therefore increase the FRP’s contribution to shear resistance. Some design codes and models therefore allow the crack angle to be adjusted so that there is a greater contribution of the FRP to shear resistance. Another factor that influences the initiation of debonding is the effective anchorage of the FRP. It has been shown that an effective anchorage of the FRP shear strengthening system can help avoid debonding of the FRP and also allow an increase in the effective strain at failure while decreasing the average interface bond stress between the FRP strengthening system and concrete (Adhikary, Mutsuyoshi, & Ashraf, 2004). The anchorage length is defined as the length beyond which the bond strength remains constant (Mofidi & Challal, 2011). This length allows the distribution of the shear stresses above the critical shear crack to the FRP U-wrap without debonding. Only the FRP whose length is greater
than this effective anchorage length will remain adequately anchored and capable of carrying a shear force (Mofidi & Challal, 2011). This length, along with the effective width of the FRP U-wrap, determines the shear force than can be resisted by the FRP wrap. An effective width of the FRP wrap is often used in models to account for the actual width of the wrap above the critical shear crack, often assumed to be 45 degrees, that has developed an appropriate anchorage length. A figure depicting the bond area, anchorage length, and width are shown in Figure 2-9, with the corresponding equivalent rectangular bonding area, effective anchorage length, and effective anchorage width.

![Figure 2-9 Bond Area, Effective Anchorage Length, Effective Width (Mofidi & Chaallal, 2011)](image)

There have been many models developed to predict the behavior of reinforced concrete structures subjected to a FRP strengthening system. Most of these models consider the total shear strength of a structure to be the sum of the shear resistance of the concrete, steel reinforcement, and FRP system. Because the contribution of the concrete and reinforcing steel to resist shear is widely understood and accepted, the main difference in these models therefore is how the contribution of the FRP system is computed (Chen, Teng, & Chen, 2010;
Mofidi & Chaallal, 2011). Most of the models used in predicting the behavior of the reinforced concrete structures shear strengthened with an FRP system limit the effective strain of the FRP system. Studies have shown that the effective stress level in the FRP reinforcement at shear failure has been much lower than its full tensile strength (Chen, Teng, & Chen, 2010). Therefore, how the effective strain is modeled in the prediction models is vital in predicting the behavior of shear strengthened reinforced concrete structure. A recent study (Mofidi & Chaallal, 2011) has compiled a number of design guidelines for predicting the behavior of reinforced concrete structures strengthened with FRP for comparison and proposed a failure mechanism model. This conceptual model proposed by Mofidi and Chaallal (2011) is based on past research on shear strengthening reinforced concrete structures with FRP materials. This model is based on guidelines and design codes that are already accepted, but was expanded to include all of the influencing factors that the authors believe effect the behavior of the FRP shear strengthening system. These influencing factors include the type of bond model used, the effective strain limit, FRP effective anchorage length, FRP effective width, strip-width-to-spacing ratio, concrete cracking angle, crack pattern, and the effect of transverse steel (Mofidi & Chaallal, 2011). The design code from the “Canadian Highway Bridge Design Code” is based on similar ideology to that of ACI 440.2R-08. There are two major differences between this design guideline and the design guideline presented in ACI 440.2R-08 (Mofidi & Chaallal, 2011). The first major difference is that CSA-S6 uses the variable truss-angle analogy instead of the 45 degree truss-angle used by ACI. The second difference is that they use one formula for the bond reduction factor, $k_2$, rather than two different formulas depending on whether U-wrapping or side bonding is used as ACI does. A recent study (Chen, Teng, & Chen, 2010) was conducted that presented modification to original model presented by Chen and Teng (2003) based on a more accurate and realistic bonded area, making it more accurate. The model examined in this experimental study is the model presented in ACI 440.2R 2008. A brief discussion of the philosophy behind this model as well as the formulation of this model is presented.
2.4.1.1. **ACI 440.2R 2008**

This design guideline is based on fiber orientation and assumed crack pattern. It is also dependent on the effective strain in the FRP reinforcing, which is dependent upon the concrete strength, wrapping scheme, and the stiffness of the laminate (ACI 440.2R-08). The shear strength is determined by calculating the force in the FRP strengthening system across the assumed crack. The formula for determining contribution of the FRP U-wrap system, $V_f$, to the resistance of shear is given in Eq. (4). All formulas are in SI units.

\[
V_f = \left( A_{fu} f_{fe} (\sin \alpha + \cos \alpha) d_{fu} \right) / s_f
\]

where

\[
f_{fe} = \varepsilon_{fe} E_f
\]

where

\[
\varepsilon_{fe} = \kappa_v \varepsilon_{fu} \leq 0.004
\]

where

\[
\kappa_v = \frac{k_1 k_2 L_e}{11,900 \varepsilon_{fu}}
\]

where

\[
L_e = \frac{23,300}{(nt_f E_f)^{0.58}}
\]

where

\[
k_1 = \left( \frac{f_e'}{27} \right)^{2/3}
\]

\[
k_2 = \frac{d_{fu} - 2L_e}{d_{fu}}
\]

where

- $f_{fe}$ = effective stress in the FRP at failure
- $\varepsilon_{fe}$ = effective strain level in FRP at failure
- $E_f$ = tensile modulus of elasticity of FRP
- $\kappa_v$ = bond-dependent coefficient for shear

**Eq. (4)**
2.5. Axial Compression Strengthening

As the case for shear reinforcing, the traditional method to improve or upgrade the axial compression performance of the reinforced concrete structure is to add additional steel reinforcement. Some of the early methods to improve the performance of reinforced concrete columns were to install a reinforced concrete cage around the pre-existing column or to apply a grout-injected steel jacketing. There were a number of disadvantages to these systems though. Some of those disadvantages included adding weight to the structure, the difficulty of construction, and deterioration of these systems due to corrosion. Therefore, fiber-reinforced polymer systems were introduced to provide axial compression strengthening to reinforced concrete columns. The application of FRP systems to improve the performance of reinforced concrete columns has quickly become preferred over traditional strengthening systems because the FRP system is able to develop greater stresses before failure. By developing greater stresses, the column can achieve larger ultimate axial loads and an increase in ductility. Another benefit of the FRP systems is that they also help prevent corrosion of the reinforcing steel within the column. Recent studies (De Luca & Nanni, 2011; Wang & Hsu, 2007; Hassan, 2003; Esfahani & Kianoush, 2005; Mooty, Issa,
Farag, & Bitar, 2006) have proven that the strength and ductility of reinforced concrete columns can be increased with the addition of FRP. Through the addition of the FRP strengthening system, the compressive concrete strength and the ultimate compressive strain of the core concrete can be increased. By increasing the ultimate compressive strain of the concrete, the ductility and energy absorption of the column will be increased (Esfahani & Kianoush, 2005). There have been two major FRP application methods that have been developed for the axial compression strengthening. The first method is the use of a prefabricated FRP shell jacketing. These shells are fabricated using fiber sheets or strands with resin impregnated before they are installed in the field. They are then bonded with resin or a non-shrink grout/mortar to the surface of the column. A unique aspect of this application technique is changing the shape of the column, often changing a square or rectangular column to a circular or elliptical column. During this process, the FRP jacket is used as permanent formwork during casting of the additional concrete. After the concrete is cured, the FRP jacket provides additional confinement of the newly formed column. The second major method of FRP application is wrapping FRP sheets around the reinforced concrete column. This is the most common method of application and the method used in this study, as shown in Figure 2-10. In most cases, uni-directional FRP sheets are wrapped around the column using a wet lay-up technique. The main fiber direction of the FRP sheets is in the hoop direction, or perpendicular to the longitudinal axis of the column. For both major application methods, the FRP-to-concrete interface is contact crucial. Therefore, the surface of the column needs to be prepared prior to application of the FRP strengthening system to ensure full utilization of the FRP.
2.5.1. **Behavior of FRP Confined Concrete**

The strengthening of existing reinforced concrete columns using traditional methods has been well documented and can be treated like additional confining steel within the column. The behavior of reinforced concrete columns strengthened with FRP systems though behaves differently from traditional reinforcing methods and cannot be modeled using traditional steel.
confined concrete models. The majority of models that have been proposed have assumed that the FRP strengthening system provides a constant confining pressure on the core concrete. They also assume that the axial stress and strain of FRP-confined concrete equal to the axial strain experienced by the core concrete (De Luca & Nanni, 2011). Many of the models assume the stress-strain behavior of FRP confined concrete is bilinear with a distinct softening in a transition zone around the stress level corresponding to the unconfined concrete strength. The initial linear stage is similar to that of unconfined concrete, indicating that FRP has not been activated. FRP confinement is considered passive in nature and therefore only begins to help in the confinement of the core concrete when the volume of the core concrete increases due to dilation. In the second portion of the stress-strain relationship, the FRP confinement is activated. The FRP strengthening system is activated in tension as it restrains the dilation of the core concrete as load increases. At this point, the FRP strengthening system begins to confine the core concrete and induces a triaxial state of stress in the concrete which leads to an increase in the compressive strength of the concrete (De Luca & Nanni, 2011). Research (Esfahani & Kianoush, 2005) has concluded that the subsequent softening of the stress-strain relationship at this point is due to the reduction of the stiffness of the column due to concrete cracking and the expansion of the confined concrete. The stress-strain relationship then follows a second linear state, with the stress of the confined concrete increasing linearly with increasing strain. This behavior will continue until the rupture strain of the FRP is reached, as shown in Figure 2-11. This rupture strain is often less than the ultimate strain determined from direct tensile tests. It is important to note though that in some cases when the amount of FRP confinement is low, the stress-strain behavior may be marked by a descending softening branch before failure. Studies (De Luca & Nanni, 2011; Wang & Hsu, 2007; Esfahani & Kianoush, 2005) have shown that this behavior is often seen in square and rectangular concrete columns. This type of stress-strain behavior is shown in Figure 2-12.
Figure 2-11 Typical Stress-Strain Relationship for Circular Columns (De Luca & Nanni, 2011)

Figure 2-12 Typical Stress-Strain Behavior for Square Columns (De Luca & Nanni, 2011)
2.5.2. Key Parameters in the Efficiency of FRP Confinement

There has been numerous studies on the effects of FRP confinement in circular columns but the amount of research done on square or rectangular columns is much more limited. The limited research on these types of columns has identified a few key parameters to determine the efficiency of the FRP strengthening system. The efficiency of the FRP confinement system is often reduced in columns that are non-circular due to the high stress concentrations at the sharp edges of those columns. At these locations, a “knife edge” effect occurs that leads to the concentration stresses at the corners and premature rupture of the FRP confinement. A recent study (De Luca & Nanni, 2011) has shown that many studies have concluded that the lateral pressure provided by the FRP confinement system is not uniform, rather it is high at the corners of the columns and lower along the faces of the columns. Therefore, a common practice to avoid the stress concentrations at the corner of these non-circular columns is to round the edges to help provide a more uniform stress distribution. A number of research studies (Hassan, 2003; Esfahani & Kianoush, 2005; Wang & Hsu, 2007; Mooty, Issa, Farag, & Bitar, 2006) have shown that rounding the edges of a square or rectangular column can dramatically increase the effectiveness of the FRP strengthening system over those columns without rounded edges. Another key parameter in determining the efficiency of the FRP confinement is the stiffness of the selected FRP system. Many studies (Hassan, 2003; De Luca & Nanni, 2011) have concluded that this is a key parameter governing the enhancement of the confined concrete columns. The stiffness of the FRP system is important because a stiffer system will help in delaying and limiting unstable crack propagation by providing a larger confinement stress on the core concrete. The stiffness of the FRP system also plays a key role in enhancing concrete axial deformation and strength (De Luca & Nanni, 2011). A stiffer FRP system will allow greater stresses to be developed in the core concrete prior to failure by providing a larger constant confining pressure on the column after the initial elastic response of the column. By providing larger stresses, and corresponding axial strains, in the core concrete, the column can experience larger axial deformation as well as a larger confined concrete strength. Another key parameter in the effectiveness of the FRP strengthening system is the number of layers of FRP wrapping, or
thickness of the FRP jacket. Increasing the thickness leads to an increase in the stiffness of the FRP confining system. Recent studies (Hassan, 2003; Wang and Hsu, 2007; De Luca & Nanni, 2011) have shown that by increasing the thickness of the FRP confining system, the load carrying capacity of the column can be increased. It was also seen from these studies that higher axial stress and strain, as well as transverse strain, in the core concrete can be achieved, which leads to a higher load carrying capacity and an increase in the axial deformation capability of the column. The thickness of the FRP confining system has also been seen (Wang & Hsu, 2007; De Luca) to attribute to the softening of the stress-strain curve of a column. For columns that are considered lightly or moderately confined, which is a function of the thickness of the FRP wrap or jacket, a distinct softening branch of the stress-strain curve can be achieved. For the columns that were heavily confined, the softening branch of the stress-strain curve was not observed but rather followed a linear relationship until rupture of the FRP. By increasing the amount of FRP confining the core concrete, there was an increase in the confinement pressure provided by the FRP strengthening system which leads to a greater resistance to the dilatation of the core concrete.

2.5.3. Determining the Strength of FRP Confined Square Columns

There have been many models and guidelines proposed to determine the strength of a rectangular column under pure axial compression (Hassan, 2003; Wang & Hsu, 2007; De Luca and Nanni, 2011, ACI 440.2R-08). The model that was used in this study was the one proposed by ACI 440.2R-08 for noncircular cross sections and is presented in Eq. (5). There is a provision in this code that accounts for the loss in efficiency of confining a noncircular column with FRP, for reasons that were discussed previously. This model provides the formulation of the maximum compressive strength of the concrete that can be obtained through FRP confinement. The maximum confined compressive strength is determined by calculating the maximum lateral confinement pressure that can be applied to the core concrete by the FRP system. This lateral confining pressure is based on the elastic modulus and thickness of the FRP system, the effective strains that can be developed within the FRP, and the shape of the column to which the FRP confining system is applied.
\[ P_n = 0.80 \left[ 0.85 f'_{cc} (A_g - A_{st}) + f_y A_{st} \right] \]

where

\[ f'_{cc} = f'_c \varphi_f 3.3 \kappa_a f_i \]

where

\[ f_i = \frac{2E_f n t_f \epsilon_{fe}}{D} \]

where

\[ \epsilon_{fe} = \kappa_e \epsilon_{fu} \]

where

\[ \frac{f_i}{f'_c} \geq 0.08 \]

where

\[ D = \sqrt{b^2 + h^2} \]

\[ \kappa_a = \frac{A_e}{A_c} \times (b/h)^2 \]

where

- \( f'_{cc} \) = compressive strength of confined concrete
- \( f'_c \) = compressive strength of concrete
- \( \varphi_f \) = FRP strength reduction factor
- \( \kappa_a \) = efficiency factor for FRP
- \( f_i \) = maximum confining presue due to FRP jacket
- \( E_f \) = tensile modulus of elasticity of FRP
- \( n \) = number of plies of FRP
- \( t_f \) = nominal thickness of one FRP sheet
- \( \epsilon_{fe} \) = effective strain level in FRP at failure
- \( A_e \) = cross-sectional area of effectively confined concrete
- \( D \) = diameter of compression member of circular cross section
- \( \kappa_e \) = efficiency factor equal to 0.55 for FRP strain
\[ \varepsilon_{fu} = \text{design rupture strain of FRP} \]
\[ A_c = \text{cross-sectional area of concrete in compression member} \]
\[ b = \text{width of compression face of member} \]
\[ h = \text{overall thickness or height in cross section of member} \]
3. EXPERIMENTAL PROGRAM

The experimental program examines the selected FRP strengthening system for strengthening structural members:

1) Flexural strengthening of reinforced concrete slabs
2) Flexural strengthening of T-section beams
3) Shear strengthening of T-section beams
4) Axial strengthening of columns

The following describes the specimens used for the experimental program, test set-up and instrumentation.

3.1. Slabs for Flexural Strengthening

A total of eight reinforced concrete slabs were tested in this experimental program. The slabs were cast using three different batches of concrete, two batches with a target nominal compressive strength of 4ksi and one batch with a target nominal compressive strength of 8ksi. The nominal dimensions of the reinforced slabs were 13’x2’x6”, and the cross section is shown in Figure 3-1. These reinforced slabs were designed to fail in a flexural mode. All of the slabs were reinforced with five longitudinal #4 steel bars spaced at five inches on center and sixteen #3 transverse bars spaced at ten inches on center.

![Figure 3-1 Cross-Section of Reinforced Slab](image)
Six of the eight slab specimens were strengthened in flexure using a carbon fiber fabric bonded to the tension surface. Out of the six slabs, five slabs were strengthened with a uni-directional (UD) carbon fiber reinforced polymer (CFRP) system, and one slab was strengthened with a bi-directional (BD) CFRP system. The remaining two slabs were not reinforced with the CFRP systems and were used as control specimens to evaluate the effectiveness of the strengthening systems with two different concrete strengths. A detailed list of the reinforced concrete slabs that were tested in this experimental program is given in Table 3-1. The target concrete strength for each specimen and the strengthening system used are indicated in the table. Three different configurations of CFRP strengthening systems were examined with these specimens: a single layer of uni-directional CFRP sheets; two layers of uni-directional CFRP sheets; and a single layer of bi-directional CFRP sheets. All of the CFRP sheets were 12 inches wide; therefore, two sheets were needed per layer to cover the entire bottom surface of each strengthened slab, as shown in Figure 3-2. Table 3-1 also indicates the casting date, testing date, and mark for each specimen. The specimen mark is comprised of a code representing the type of specimen being tested, the target concrete strength, type of strengthening system used (UD or BD, 1 or 2 layers), and the numerically represented cast date.

![Figure 3-2 Cross Section of Flexural Strengthening of RC Slab](image-url)
### Table 3-1 Summary of Flexural Reinforced Slab Specimens

<table>
<thead>
<tr>
<th>Target Concrete Strength</th>
<th>Strengthening System</th>
<th>Cast Date</th>
<th>Test Date</th>
<th>Specimen Mark</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 ksi</td>
<td>None</td>
<td>1/19/11</td>
<td>6/28/11</td>
<td>Slab.4ksi.none.011911</td>
</tr>
<tr>
<td></td>
<td>1 Layer Uni-directional CFRP</td>
<td>1/19/11</td>
<td>6/28/11</td>
<td>Slab.4ksi.1UD.011911</td>
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<tr>
<td></td>
<td>2 Layers Uni-directional CFRP</td>
<td>1/19/11</td>
<td>6/29/11</td>
<td>Slab.4ksi.2UD.011911</td>
</tr>
<tr>
<td></td>
<td>1 Layer Uni-directional CFRP</td>
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<td>7/1/11</td>
<td>Slab.4ksi.1BD.051211</td>
</tr>
<tr>
<td></td>
<td>1 Layer Bi-directional CFRP</td>
<td>5/12/11</td>
<td>7/1/11</td>
<td></td>
</tr>
<tr>
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<td>None</td>
<td>1/19/11</td>
<td>6/30/11</td>
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</tr>
<tr>
<td></td>
<td>1 Layer Uni-directional CFRP</td>
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<td>6/30/11</td>
<td>Slab.8ksi.1UD.011911</td>
</tr>
<tr>
<td></td>
<td>2 Layers Uni-directional CFRP</td>
<td>1/19/11</td>
<td>6/30/11</td>
<td>Slab.8ksi.2UD.011911</td>
</tr>
</tbody>
</table>

### 3.2. T-Beams for Flexural Strengthening

A total of eight T-beams were tested in this experimental program. These beams were designed to fail in a flexural mode. Each T-beam had nominal dimensions of 10 feet long and 16 inches deep, with a web width of 12 inches, and with a flange width of 24 inches and 6 inches thick. The eight flexure T-beams were reinforced with two #4 longitudinal bars in the top and bottom of the web and four #4 longitudinal bars in the flange. They were also reinforced with #3 stirrups in the web spaced at four inches on center. Twelve #3 stirrups were placed in the flange spaced at 12 inches on center. The typical cross section for the flexure T-beams is shown in Figure 3-3.
A detailed list of the flexure T-beams tested in the experimental program is provided in Table 3-2. The table indicates the target nominal concrete strength for each specimen and the type strengthening system that was used. Three different configurations of strengthening were examined using these specimens: a single layer of uni-directional CFRP; two layers of uni-directional CFRP, and a single layer of bi-directional CFRP. All CFRP sheets were twelve inches wide; therefore only one sheet was needed per layer to cover the entire bottom surface of the web of each strengthened T-beam, as shown in Figure 3-4. The table also indicates the casting and testing dates for each specimen, along with a unique specimen mark. The specimen mark indicates the type of specimen, the target concrete strength, the strengthening system provided, and the cast date.
### Table 3-2 Summary of Flexural Reinforced T-Beam Specimens

<table>
<thead>
<tr>
<th>Target Concrete Strength</th>
<th>Strengthening System</th>
<th>Cast Date</th>
<th>Test Date</th>
<th>Specimen Mark</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>4 ksi</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>None</td>
<td>1/19/11</td>
<td>6/28/11</td>
<td>FlexBeam.4ksi.none.011911</td>
<td></td>
</tr>
<tr>
<td>1 Layer Unidirectional CFRP</td>
<td>1/19/11</td>
<td>6/29/11</td>
<td>FlexBeam.4ksi.1UD.011911</td>
<td></td>
</tr>
<tr>
<td>2 Layers Unidirectional CFRP</td>
<td>1/19/11</td>
<td>6/29/11</td>
<td>FlexBeam.4ksi.2UD.011911</td>
<td></td>
</tr>
<tr>
<td>1 Layer Unidirectional CFRP</td>
<td>5/12/11</td>
<td>7/1/11</td>
<td>FlexBeam.4ksi.1UD.051211</td>
<td></td>
</tr>
<tr>
<td>1 Layer Bidirectional CFRP</td>
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<td>7/1/11</td>
<td>FlexBeam.4ksi.1BD.051211</td>
<td></td>
</tr>
<tr>
<td><strong>8 ksi</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>None</td>
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<td>6/29/11</td>
<td>FlexBeam.8ksi.none.011911</td>
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</tr>
<tr>
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<td>6/30/11</td>
<td>FlexBeam.8ksi.1UD.011911</td>
<td></td>
</tr>
<tr>
<td>2 Layers Unidirectional CFRP</td>
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<td>6/30/11</td>
<td>FlexBeam.8ksi.2UD.011911</td>
<td></td>
</tr>
</tbody>
</table>
3.3. T-Beams for Shear Strengthening

The set of T-beams designed to examine the effectiveness of the shear strengthening have nominal dimensions of 9 feet long and 16 inches deep, with a web width of 12 inches, and with a flange width of 24 inches and 6 inches thick, as shown in Figure 3-5. This set of six T-beams was designed to fail in a shear failure mode. These six shear T-beams were reinforced with two layers of three #8 longitudinal bars in the bottom of the web and two #4 longitudinal bars in the top of the web, as well as with four #4 longitudinal bars in the flange. They were reinforced with #3 stirrups in the web spaced at 12 inches on center. The flange was reinforced with #3 stirrups spaced at 12 inches on center. The maximum stirrup spacing suggested by ACI 318 was ignored for these beams so that the effect of the FRP shear strengthening system could be closely examined. Two additional #4 stirrups were also placed at the location of supports. The typical cross-section for these specimens is shown in Figure 3-5.
A detailed list of the reinforced concrete shear T-beams that were tested is given in Table 3-3. The table indicates the target concrete strength for each specimen along with the type of strengthening used. Two different strengthening configurations were examined with the shear T-beams. One of the systems consisted of 12 inch wide uni-directional CFRP U-wrapped sheets placed at 15 inches on center between the locations of supports, as shown in Figure 3-6. The other strengthening configuration used the same U-wraps but with CFRP anchors at the top of each wrap. Each CFRP anchor was placed into a drilled hole at the top of the web. The cross-section of the anchored T-beams is shown in Figure 3-7.
Figure 3-6 Cross Section of CFRP U-Wrap Shear Strengthening System
Figure 3-7 Cross Section of CFRP Anchorage Shear Strengthening System

Table 3-3 also indicates the casting and testing dates for each specimen, along with the specimen mark. The specimen mark represents the type of specimen, the target concrete strength, the strengthening configuration, and the cast date.
### Table 3.3 Summary of Shear Reinforced T-Beam Specimens

<table>
<thead>
<tr>
<th>Target Concrete Strength</th>
<th>Strengthening System</th>
<th>Cast Date</th>
<th>Test Date</th>
<th>Specimen Mark</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 ksi</td>
<td>None</td>
<td>1/19/11</td>
<td>7/12/11</td>
<td>ShearBeam.4ksi.none.011911</td>
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<tr>
<td></td>
<td>Uni-directional CFRP U-Wraps</td>
<td>1/19/11</td>
<td>7/12/11</td>
<td>ShearBeam.4ksi.WrapsUD.011911</td>
</tr>
<tr>
<td></td>
<td>Uni-directional CFRP U-Wraps with Anchors</td>
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<td>7/14/11</td>
<td>ShearBeam.4ksi.AnchoredWrapsUD.011911</td>
</tr>
<tr>
<td>8 ksi</td>
<td>None</td>
<td>1/19/11</td>
<td>7/12/11</td>
<td>ShearBeam.8ksi.none.011911</td>
</tr>
<tr>
<td></td>
<td>Uni-directional CFRP U-Wraps</td>
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<td>7/13/11</td>
<td>ShearBeam.8ksi.WrapsUD.011911</td>
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<tr>
<td></td>
<td>Uni-directional CFRP U-Wraps with Anchors</td>
<td>1/19/11</td>
<td>7/14/11</td>
<td>ShearBeam.8ksi.AnchoredWrapsUD.011911</td>
</tr>
</tbody>
</table>

### 3.4. Columns for Axial Compression Strengthening

A total of six columns were tested in this experimental program. All columns were designed to fail in a compression mode. Each column had nominal dimensions of 44 inches long with a square cross section of 10 inches by 10 inches. The edges of the columns were chamfered at a one inch to one inch slope. The six columns were reinforced with four #5 longitudinal bars and #3 transverse ties spaced at six inches on center. Two 6 inch steel collars were cast in place at the top and bottom of each specimen. The collars were used to prevent premature crushing at the column ends during testing. The typical cross section for the column is shown in Figure 3-8.
Figure 3-8 Cross Section of Columns

A detailed list of the reinforced concrete columns that were tested is given in
Table 3-4. The table indicates the target concrete strength for each specimen along with the type of strengthening used. Two different configurations of CFRP strengthening were examined with these specimens: a single layer of uni-directional CFRP sheets and two layers of uni-directional CFRP sheets. The uni-directional CFRP sheets were 12 inches wide; therefore for each layer three wraps were placed around the column to cover the entire effective wrapping height of 32 inches. There was an effective overlap of 2 inches between the top, middle, and bottom wraps, as shown in Figure 3-9. Each wrap had an overlap length of 6 inches. The cross-section of the columns wrapped with one layer and two layers are shown in Figure 3-10 and Figure 3-11, respectively.

![Figure 3-9 Detail of CFRP Overlap for Columns](image)
Figure 3-10 Cross Section of Columns Strengthened with 1 Layer of CFRP

Figure 3-11 Cross Section for Columns Strengthened with 2 Layers of CFRP
Table 3-4 also indicates the casting and testing dates for each specimen, along with the specimen mark. The specimen mark represents the type of specimen, the target concrete strength, the strengthening configuration, and the cast date.
### Table 3-4 Summary of Axial Compression Reinforced Columns

<table>
<thead>
<tr>
<th>Target Concrete Strength</th>
<th>Strengthening System</th>
<th>Cast Date</th>
<th>Test Date</th>
<th>Specimen Mark</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 ksi</td>
<td>None</td>
<td>3/7/11</td>
<td>9/13/11</td>
<td>Column.4ksi.none.030711</td>
</tr>
<tr>
<td>1 Layer Uni-directional CFRP</td>
<td>3/7/11</td>
<td>9/13/11</td>
<td>Column.4ksi.1UD.030711</td>
<td></td>
</tr>
<tr>
<td>2 Layers Uni-directional CFRP</td>
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<td>9/14/11</td>
<td>Column.4ksi.2UD.030711</td>
<td></td>
</tr>
<tr>
<td>8 ksi</td>
<td>None</td>
<td>3/7/11</td>
<td>9/14/11</td>
<td>Column.8ksi.none.030711</td>
</tr>
<tr>
<td>1 Layer Uni-directional CFRP</td>
<td>3/7/11</td>
<td>9/14/11</td>
<td>Column.8ksi.1UD.030711</td>
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</tr>
<tr>
<td>2 Layers Uni-directional CFRP</td>
<td>3/7/11</td>
<td>9/14/11</td>
<td>Column.8ksi.2UD.030711</td>
<td></td>
</tr>
</tbody>
</table>

### 3.5. Test Setup

All reinforced slabs and T-beams were tested under a four-point loading configuration. A schematic for the test setup for the reinforced slab and T-beam to examine the flexural behavior and shear behavior are shown in Figure 3-12, Figure 3-13, and Figure 3-14 respectively.
Figure 3-12 Schematic of the Slab Test Setup

Figure 3-13 Schematic of the T-Beam for Flexural Strengthening
The actual test setup for the slab is shown in Figure 3-15 and the test setup for the beams is shown in Figure 3-16. All specimens were simply supported with a roller and pin configuration that allowed the slabs to have a 12 feet clear span, a 10 feet clear span for T-beam for flexural strengthening, and a 9 feet clear span for T-beams for shear strengthening. All specimens were loaded with a hydraulic actuator up to failure. A spreader beam was used to distribute the load from the actuator to two HSS steel beams. The beams pressed on ¼” thick neoprene pads that were bearing on the top concrete surface. The beams and pads produced two line loads 2 feet apart and distributed over the width of the specimens.
Figure 3-15 Slab Test Setup
All column specimens are tested in axial compression. A schematic for the test setup for the reinforced column to examine axial compression strengthening is shown in Figure 3-17. The actual setup for the column is shown in Figure 3-18. Load was applied by a compression testing machine with a capacity of 2,000,000 lbs. A cementitious material was applied to the top and bottom surfaces of the column prior to testing to ensure that the applied load was evenly distributed across the cross section of the column.
Figure 3-17 Schematic of the Column Test Setup
All specimens tested in this program were loaded in stages. The slabs and flexure beams were loaded in three stages which were designed to mirror the significant load levels determined in the design. The first stage of loading represents the service loading stage. For this stage, load was applied at a rate of 0.10 in/min until a pre-determined service load was reached. The specimen was then unloaded at a rate of 0.25 in/min. In the second stage the specimen was loaded up to the predicted ultimate load. For this stage, load was applied at a rate of 0.10 in/min until a pre-determined ultimate load was reached. The specimen was then unloaded at a rate of 0.25 in/min. For third and final stage the specimen was loaded up to failure. For this stage, the specimens were loaded at a rate of 0.25 in/min until failure. A schematic for this loading sequence is shown in Figure 3-19.
The shear beams were loaded in two stages. In the first stage the beam was loaded up to the predicted ultimate load. For this stage, the beam was loaded at a rate of 0.10 in/min until a pre-determined ultimate load level was reached. The beams were then unloaded at a rate of 0.25 in/min. The second and final stage the beam was loaded to failure. For this stage, each beam was loaded at a rate of 0.25 in/min until failure. A schematic of the loading sequence is shown in Figure 3-20.

The columns were also loaded in two stages. In the first stage the column was loaded up to the predicted ultimate load. For this stage, the column was loaded at a rate of 0.02 in/min until a pre-determined ultimate load level was reached. The column was then unloaded at a
rate of 0.05 in/min. The second and final stage the column was loaded to failure. For this stage, each column was loaded at a rate of 0.05 in/min until failure. A schematic of the loading sequence is shown in Figure 3-20.

3.6. Instrumentation

During testing of each slab and beam, the applied load, the specimen deflection, and strains at several selected locations were measured. The applied load was measured using a load cell integrated into the hydraulic actuator. Displacements were measured at three locations at the mid-width of each specimen. One location was at the midspan, and the others were directly under each loading point. All deflections were measured using linear
potentiometers. Three strain measurements were taken at the midspan of each flexure specimen. The compressive strain in the concrete was measured on the top surface, the tension strain on the bottom surface of each specimen, and a third strain measurement was taken on the side of each specimen at the level of the internal steel tension reinforcement. All strain measurements were measured using 200mm pi-shaped clip gauges. Figure 3-21 and Figure 3-22 provide a detailed schematic of the instrumentation used for the slab and the T-beam test for flexural strengthening, respectively.

![Figure 3-21 Instrumentation for the Slab](image-url)
During each shear beam test, the applied load, the beam deflection, and strain at several locations were measured. The applied load and beam displacements were measured in the same manner as for the flexure specimens. The strain in the shear specimens was measured at three midspan locations similar to the slab and flexure beams. In addition to these flexure strains, eight additional strain measurements were taken near the end of each specimen, placed horizontally and vertically on each side of the beam web, centered on the outermost U-wraps, as shown in Figure 3-23. All eight of these strain measurements were measured using 100 mm pi gauges.
During each column test, the applied load, the column deflection, and strain at several locations were measured. The applied load was measured using a pressure transducer integrated into the testing machine. The column displacements were measured in the axial direction using linear potentiometers. The strain in the column specimens was measured at four mid-height locations, on two column faces axial strain will be measured and on the other two column faces hoop strain will be measured. Strain measurements will be made using 100 mm pi-shaped clip gauges. A schematic of the instrumentation for the columns is shown in Figure 3-24.
Figure 3-24 Instrumentation for the Columns
4. MATERIAL PROPERTIES

This section summarizes test results of the material properties used in fabrication of the slabs, T-beams, and columns, as well as the FRP material used for strengthening.

4.1. Steel Reinforcement

Material tests were conducted on the A706 steel reinforcement used for the concrete specimens tested in this program. The samples were selected from each batch of reinforcing steel used. Samples were prepared and tested following ASTM A370-10 “Standard Test Methods and Definitions for Mechanical Testing of Steel Products”.

4.1.1. Test Specimens

The specimens tested were cut from full length steel bars sampled from each batch used to produce the large concrete specimens. The steel material specimens tested included #8 bars, which were used as longitudinal reinforcing in the shear beams, #5 bars, which were used as longitudinal reinforcing in the columns, #4 bars, which were used as longitudinal reinforcing in the slabs and flexural beams, and #3 bars, which were used as transverse reinforcing in all of the large concrete specimens. All specimens were cut to length to provide an 8-in gage length. These steel specimens were cut using an abrasive saw.

4.1.2. Test Setup

Prior to testing, an 8-in gage length was marked on each specimen. The diameter of each specimen was measured at two different locations within the gage length using a caliper. Each specimen was loaded until rupture according to ASTM A370-10 using a universal testing machine. An electronic extensometer was attached to the specimen mid-height to measure strain over a 2 inch gage length. A typical specimen loaded in the testing machine is shown in Figure 4-1 prior to testing. Specimens were loaded in tension at a constant rate of 0.0625 in/min until yield. The rate was increased to 0.5 in/min after yield. The applied load, cross-head displacement, and measured strain were recorded for each specimen.
4.1.3. Test Results

Test results including the average yield stress, ultimate load, and maximum stress are reported in Table 4-1. These average values were used in the analysis of the reinforced concrete specimens. The specimens having the letter A in their specimen ID are from the batch of reinforcing steel that was used in the concrete specimens cast on 05/12/11. All other specimens were from the steel used in the concrete specimens cast on 01/19/11. The max stress is the stress associated with the maximum load measured. The yield stress was determined using two different methods. The first method was the autographic diagram method, which is applied when a distinct yield plateau is observed, and was true for most of the #8, #5, and #4 specimens. For this method, the yield stress is the value associated with yield plateau. A typical stress-strain graph for this method is shown in Figure 4-2.
Table 4-1 Average Yield, Max Load, and Stress

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Average Yield Load (kips)</th>
<th>Average Yield Stress (ksi)</th>
<th>Average Max Load (kips)</th>
<th>Average Max Stress (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#8</td>
<td>53.30</td>
<td>67.86</td>
<td>74.12</td>
<td>94.37</td>
</tr>
<tr>
<td>#5</td>
<td>21.68</td>
<td>69.98</td>
<td>29.28</td>
<td>95.43</td>
</tr>
<tr>
<td>#4</td>
<td>12.91</td>
<td>65.77</td>
<td>17.95</td>
<td>91.44</td>
</tr>
<tr>
<td>#3</td>
<td>8.08</td>
<td>73.12</td>
<td>11.71</td>
<td>106.06</td>
</tr>
<tr>
<td>#4-A</td>
<td>13.32</td>
<td>67.85</td>
<td>17.90</td>
<td>91.16</td>
</tr>
<tr>
<td>#3-A</td>
<td>8.22</td>
<td>74.44</td>
<td>11.73</td>
<td>106.18</td>
</tr>
</tbody>
</table>

Figure 4-2 Typical yield plateau stress-strain graph

The second method that was used in determining the yield stress of the specimens was the offset method, which was used when no distinct yield plateau was observed. This method was used for all of the #3 stirrups and one of the #5 bars. This method used the elastic modulus of each specimen to draw a line parallel to the initial stress-strain data offset by a strain of 0.2%. The intersection of this offset line and the initial stress-strain data curve was
determined to be the yield stress of the specimen. Graphical presentation of this method is shown in Figure 4-3.

![Figure 4-3 Typical 0.2% offset stress-strain graph](image)

The measured average rupture strains for the tested specimens are shown in Table 4-2. These average values were used in the analysis of the reinforced concrete specimens. The specimens having the letter A in their specimen ID are from the batch of reinforcing steel that was used in the concrete specimens cast on 05/12/11. All other specimens were from the steel used in the concrete specimens cast on 01/19/11. The rupture strain was determined after failure by using the measured change in gage length and the original gage length.
Table 4-2 Average Rupture Strain

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Rupture Strain (in/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#8</td>
<td>0.19</td>
</tr>
<tr>
<td>#5</td>
<td>0.17</td>
</tr>
<tr>
<td>#4</td>
<td>0.14</td>
</tr>
<tr>
<td>#3</td>
<td>0.14</td>
</tr>
<tr>
<td>#4-A</td>
<td>0.17</td>
</tr>
<tr>
<td>#3-3</td>
<td>0.15</td>
</tr>
</tbody>
</table>

A complete listing of all the steel specimens tested can be found in Appendix B-1 of this thesis.
4.2. Concrete Properties

Material properties of the concrete used in fabrication of the specimens were determined by concrete cylinders made from each batch used in the construction of the large specimens to determine the 28 day compressive strength. In total, five different batches of concrete were used in the construction of the large specimens. These batches include a target 4ksi and 8ksi concrete batch for the slabs and beams used in casting on 01/19/11, a target 4ksi and 8ksi concrete batch for the columns used in casting on 03/07/11, and a 4ksi concrete batch for the beams and slabs cast on 05/12/11. These cylinders were tested in accordance with ASTM C39-01 and ASTM C469-94.

4.2.1. Test Specimens

All cylinders made from the concrete batches were 4x8 inch cylinders. The concrete used to make these cylinders was taken from the middle portion of each batch of concrete.

4.2.2. Test Setup

Prior to testing, the diameter of each cylinder was measured with a caliper by averaging two diameters at right angles to each other. The length of each cylinder was also measured using a caliper. Each cylinder’s weight was recorded to the nearest 0.01 lb. The cylinder was capped with neoprene and loaded by a MTS machine. A load was applied continuously to the cylinder at a rate of 20-25 psi/sec until failure. The maximum load was recorded at failure to determine the compressive strength of the concrete. A cylinder loaded into the MTS machine prior to testing is shown in Figure 4-4.
4.2.3. **Test Results**

The average diameter, area, max load, and 28 day compressive strength of the cylinders for each batch of concrete are given in Table 4-3, Table 4-4, and Table 4-5. The average 28-day compressive strength of each cylinder was determined based on the measured max load.

**Table 4-3 Average concrete strength for beams and slabs cast on 01/19/11**

<table>
<thead>
<tr>
<th>Concrete Batch</th>
<th>Average Diameter (in)</th>
<th>Area (in²)</th>
<th>Max Load (lbs)</th>
<th>Max Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4ksi</td>
<td>3.98</td>
<td>12.45</td>
<td>53,730</td>
<td>4,315</td>
</tr>
<tr>
<td>8ksi</td>
<td>4.01</td>
<td>12.60</td>
<td>82,410</td>
<td>6,540</td>
</tr>
</tbody>
</table>
Table 4-4 Average concrete strength for columns cast on 03/07/11

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Average Diameter (in)</th>
<th>Area (in²)</th>
<th>Max Load (lbs)</th>
<th>Max Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4ksi</td>
<td>3.98</td>
<td>12.42</td>
<td>51,251</td>
<td>4,125</td>
</tr>
<tr>
<td>8ksi</td>
<td>3.98</td>
<td>12.46</td>
<td>99,318</td>
<td>7,970</td>
</tr>
</tbody>
</table>

Table 4-5 Average concrete strength for beams and slabs cast on 05/12/11

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Average Diameter (in)</th>
<th>Area (in²)</th>
<th>Max Load (lbs)</th>
<th>Max Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4ksi</td>
<td>4.03</td>
<td>12.77</td>
<td>63,522</td>
<td>4,975</td>
</tr>
</tbody>
</table>

The results for all the cylinders tested to determine the concrete strength of each cast can be found in Appendix B-2 of this thesis.
4.3. CFRP Material Properties

Material tests were conducted on the CFRP sheets used in the strengthening of the concrete specimens. Both uni-directional (UD) and bi-directional (BD) CFRP sheets were tested. These CFRP sheets were tested in accordance with ASTM D3039.

4.3.1. Test Specimens

Witness panels were produced of each type of fiber sheet and cut into strips having nominal dimensions of 1” x 12”. After the strips of CFRP were cut, two 3 inch long tapered aluminum tabs were bonded to each end of each strip to finish the test specimens. Figure 4-5 demonstrates the coupon making process. A jig and weights were used ensure a proper alignment and a secure bond between the tabs and specimens, as is shown in Figure 4-6. The tabs were allowed to cure for ten days prior to testing. Finished specimens prior to testing are shown in Figure 4-7.

Figure 4-5 Tabs bonded to CFRP strips using resin
Prior to testing, each specimen was measured using a caliper and micrometer to determine their actual dimensions. The average of these measurements is reported in Table 4-6 for all tested specimens.
### Table 4-6 Average Dimensions of CFRP Coupons

<table>
<thead>
<tr>
<th>Material</th>
<th>Average Width (inches)</th>
<th>Average Thickness (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uni-directional</td>
<td>0.98</td>
<td>0.042</td>
</tr>
<tr>
<td>Bi-directional</td>
<td>0.96</td>
<td>0.051</td>
</tr>
</tbody>
</table>

4.3.2. **Test Setup**

Each specimen was tested to failure in tension according to ASTM D3039 using a universal testing machine. The specimens were carefully aligned prior to being gripped in the hydraulic jaws of the testing machine. An electronic extensometer was attached to the specimen mid-height to measure strain over a 2 inch gage length. A selected specimen gripped in the machine prior to testing is shown in Figure 5. Specimens were loaded in tension at a constant displacement rate of 0.05in/min to failure.
4.3.3. **Test Results**

Applied load, cross-head displacement, and measured strain were recorded for each specimen. Average peak loads are reported in detail for each specimen in Table 4-7 along with the calculated values of stress at ultimate and elastic modulus. Values for ultimate stress and elastic modulus were calculated by the measured widths and a nominal thickness of 0.02” for each UD and BD specimen. The measured thickness was not used in calculating stress, as it varied significantly depending on the amount of resin used in preparation of a specific sample. As the fibers, not the resin, provide virtually all strength in tension, it is the fiber thickness, which dictates stress. In addition, the nominal fiber thickness is the value used in design. The average values for elastic modulus presented in Table 4-7 were calculated by determining the slope of a best-fit line through the initial linear portion of the
stress-strain curve. These material properties were used in the analysis of the reinforced concrete specimens.

### Table 4-7 Average Peak Load, Ultimate Stress, and Elastic Modulus

<table>
<thead>
<tr>
<th>Material</th>
<th>Average Peak Load (kips)</th>
<th>Average Ultimate Stress (ksi)</th>
<th>Average Elastic Modulus (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uni-directional</td>
<td>4.41</td>
<td>226</td>
<td>13,760</td>
</tr>
<tr>
<td>Bi-directional</td>
<td>4.01</td>
<td>208</td>
<td>13,552</td>
</tr>
</tbody>
</table>

The results for all of the material tests performed on the uni-directional and bi-directional fibers can be found in Appendix B-3 of this thesis.
5. TEST RESULTS

This chapter presents a summary of the test results of the experimental program undertaken to examine the effectiveness of the new CFRP material proposed for strengthening of reinforced concrete structures. The results are for the eight reinforced concrete slabs, eight T-beams, and six concrete columns. The chapter will focus on the behavior of the members strengthened with FRP systems. It will also examine the effectiveness of different FRP strengthening systems on the overall strength, ductility, and mode of failure of the reinforced concrete members.

5.1. Reinforced Concrete Slabs

A total of eight reinforced concrete slabs were tested in this experimental program. The parameters included in the investigation were concrete strength, type of CFRP fabric (uni-directional vs. bi-directional), and number of CFRP layers used. It was found, in general, that the CFRP strengthening system increased the overall load carrying capacity and the stiffness of the strengthened slabs. It was also found that the performance of the slab strengthened with a uni-directional fabric was virtually identical to the slab strengthened with the bi-directional fabric. A summary of the results for the slabs cast on 01/19/11 are given in Table 5-1. The results of the two slabs cast separately to compare the effect of uni-directional and bi-directional CFRP are given in Table 5-2.
Table 5-1 Test Results of the Reinforced Slabs cast on 01/19/11

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Max Load (kips)</th>
<th>% Load Increase</th>
<th>Deflection at Failure (in)</th>
<th>% Reduction in Deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab.4ksi.none.011911</td>
<td>11.56</td>
<td>N/A</td>
<td>9.1</td>
<td>N/A</td>
</tr>
<tr>
<td>Slab.4ksi.1UD.011911</td>
<td>25.30</td>
<td>54</td>
<td>4.0</td>
<td>-56</td>
</tr>
<tr>
<td>Slab.4ksi.2UD.011911</td>
<td>28.20</td>
<td>59</td>
<td>4.11</td>
<td>-55</td>
</tr>
<tr>
<td>Slab.8ksi.none.011911</td>
<td>11.81</td>
<td>N/A</td>
<td>10.24</td>
<td>N/A</td>
</tr>
<tr>
<td>Slab.8ksi.1UD.011911</td>
<td>19.2</td>
<td>38</td>
<td>3.49</td>
<td>-66</td>
</tr>
<tr>
<td>Slab.8ksi.2UD.011911</td>
<td>27.96</td>
<td>58</td>
<td>3.19</td>
<td>-69</td>
</tr>
</tbody>
</table>

Table 5-2 Test Results for Reinforced Slabs cast on 05/12/11

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Max Load (kips)</th>
<th>Deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab.4ksi.1UD.051211</td>
<td>22.81</td>
<td>4.84</td>
</tr>
<tr>
<td>Slab.4ksi.1BD.051211</td>
<td>22.03</td>
<td>4.90</td>
</tr>
</tbody>
</table>

All tests were conducted in stages, completed with failure due to crushing of the concrete in the compression zone or debonding of the strengthening system from the reinforced slabs. The load-deflection behavior of the reinforced slabs, in general, was characterized by a linear relationship until the initiation of first crack, as shown in Figure 5-1. After the initiation of first crack, there was a decrease in the stiffness and the load-deflection behavior followed a non-linear behavior until yielding of the reinforcing steel. Yielding of the longitudinal steel reinforcing was confirmed by the load-strain relationship, as shown in Figure 5-2. After yielding of the reinforcing steel, there was a subsequent decrease in the stiffness and the load-deflection behavior followed a non-linear relationship until failure, as shown in Figure 5-1. The load-deflection behavior of the slabs strengthened for flexure is grouped based on target nominal concrete compressive strength and date of cast as discussed in the following sections.
Figure 5-1 Load-Deflection Behavior for Slab.4ksi.1UD.011911
The load-deflection behavior of the slabs cast on 01/19/11 with the 4ksi target nominal compressive strength concrete is shown in Figure 5-3. The behavior of the strengthened slabs was characterized, in general, by an increase in the stiffness of the slabs strengthened with the CFRP systems due to delay of both cracking of the concrete and yielding of the steel reinforcement. A change of the stiffness occurred after initiation of the first crack and another significant change of the stiffness occurred after yielding of the reinforcement. Yielding of the longitudinal steel was confirmed when examining the load-strain relationship of the slab, as shown in Figure 5-2. There is an obvious loss of ductility of the strengthened slabs in comparison to the control slab when examining the load-deflection behavior. The loss in ductility can be attributed to the premature failure of the slab due to intermediate crack (IC) debonding. The results indicate also that using two layers of CFRP slightly increases the stiffness and overall flexural strength after yielding in comparison to the use of one layer of CFRP. The failure mode for the strengthened 4ksi target nominal strength
reinforced concrete slabs was due to intermediate crack debonding, which will be discussed shortly.

Figure 5-3 Load-Displacement Behavior for 4ksi Concrete Slabs Cast on 01/19/11

The load-deflection behavior of the slabs cast on 01/19/11 with the 8ksi target nominal compressive strength concrete is shown in Figure 5-4. Similarly, the behavior was characterized, in general, by an increase in the stiffness of the slabs strengthened with the CFRP systems due to delay of cracking of the concrete and yielding of the steel reinforcement. The comparison between using one and two layers of CFRP follow the same trend observed in Figure 5-3 and was more pronounced for this case through the increase of stiffness and flexural strength after yielding of the steel reinforcement of the strengthened slabs. Test results indicated that the effect of two layers in comparison to one layer was more effective due to the use of high strength concrete, which provides more compression forces in the compression zone to balance the tensile forces provided by the use of two layers of
CFRP. Test results indicate also that there is also a significant loss of ductility due to the use of CFRP strengthening system in comparison to the control slab. The decrease in ductility can be associated with the premature failure of the strengthened reinforced concrete slabs due to intermediate crack debonding, which will be discussed shortly.

![Figure 5-4 Load-Displacement Relationship for 8ksi Concrete Slabs Cast on 01/19/11](image)

The load-deflection behavior of the slabs cast on 05/12/11 with 4ksi target nominal compressive strength concrete strengthened with uni-directional and bi-directional CFRP sheets are shown in Figure 5-5. The load-deflection behavior of the slabs is almost identical; therefore test results suggest that the presence of the transverse fiber was ineffective.
Figure 5-5 Load-Displacement Behavior for 4ksi Concrete Slabs Cast on 05/12/11

The failure behavior of the control slabs were characterized by large deflection followed by crushing of the concrete in the compression zone, as shown in Figure 5-6.
The typical observed failure mode for most of the strengthened slabs was intermediate crack (IC) debonding of the CFRP. This debonding mechanism was the dominant form of debonding in flexural strengthened reinforced concrete slabs. These cracks are typically located within the shear span, the distance from the point load to the nearest support, and induce high interfacial shear stress that cause FRP debonding in the cases that the induced shear strength exceeds the bond strength. These cracks typically propagate along the shear span in the direction of decreasing moment, as shown in Figure 5-7. It was observed that this type of failure does not penetrate into the aggregate, but rather progresses through the thin mortar-rich layer comprising the surface of the concrete, as shown in Figure 5-8.
Figure 5.7 Intermediate Crack Debonding of FRP Strengthening System
Based on the observed behavior, test results indicate in general that the use of CFRP strengthening systems increased the load carrying capacity. The strengthening system also increases the cracking and yielding loads in comparison to the control specimens. All of the strengthened slabs had a decrease in deflection, consequently the ductility, compared to the control specimens. This reduction in deflection is due to the increase in the reinforcement ratio which results in the reduction of the number and depth of flexure cracks before failure when compared to the control specimens. Figure 5-9 shows the load deflection behavior of all of the tested slabs strengthened with uni-directional fibers cast on 01/19/11, using one and two layers. The behavior of the control slab with target compressive strength of 4ksi and 8ksi are almost identical since both slabs were under reinforced, “tension control,” and have the same steel reinforcement ratio. As discussed, use of two layers of CFRP increases the flexural strength and slightly reduces the ductility, in comparison to the use of one layer of

Figure 5-8 Debonding of FRP along Thin Mortar-Rich Layer
CFRP. The effectiveness of the strengthening system was less pronounced for the 8ksi target compressive strength slab with one layer of CFRP compared to the 4ksi target compressive strength slab with one layer of CFRP. This result could be explained by the premature debonding of the FRP due to the formation of intermediate cracks which propagated quickly through a location of weak bond interface between the slab and CFRP system, as shown in Figure 5-10 and Figure 5-11.

Figure 5-9 Load Deflection Behavior for All Slabs Cast on 01/19/11
Figure 5-10 Formation of Intermediate Crack for Slab.8ksi.2UD.011911

Figure 5-11 Premature Failure of Slab.8ksi.2UD.011911 Due to IC Debonding
For each slab, three 4x8 inch concrete cylinders were tested to determine the concrete compressive strength on the day of testing. Summary of the measured average concrete strengths for three cylinders are given in Table 5-3. Concrete strength for the cast on 05/12/11 is given in Table 5-4. The average compressive strength of the 4ksi and 8ksi target strength cast on 01/19/11 are 5.8ksi and 7.6ksi, respectively. The average of the 4ksi target strength concrete cast on 05/12/11 is 6.00ksi as given in Table 5-4.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Compressive Concrete Strength (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab.4ksi.none.011911</td>
<td>5.7</td>
</tr>
<tr>
<td>Slab.4ksi.1UD.011911</td>
<td>5.9</td>
</tr>
<tr>
<td>Slab.4ksi.2UD.011911</td>
<td>5.6</td>
</tr>
<tr>
<td>Average:</td>
<td>5.8</td>
</tr>
<tr>
<td>Slab.8ksi.none.011911</td>
<td>7.7</td>
</tr>
<tr>
<td>Slab.8ksi.1UD.011911</td>
<td>7.5</td>
</tr>
<tr>
<td>Slab.8ksi.2UD.011911</td>
<td>7.6</td>
</tr>
<tr>
<td>Average:</td>
<td>7.6</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Compressive Concrete Strength (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab.4ksi.1UD.051211</td>
<td>5.9</td>
</tr>
<tr>
<td>Slab.4ksi.1BD.051211</td>
<td>6.1</td>
</tr>
<tr>
<td>Average:</td>
<td>6.0</td>
</tr>
</tbody>
</table>

The models discussed in the literature review for predicting the intermediate crack debonding failure load of a reinforced concrete member flexurally strengthened with FRP material were compared to the results of the reinforced concrete slabs. These models used the material properties of the CFRP and steel reinforcement as discussed in chapter four as well as day of
testing concrete strength, as given in Table 5-3 and Table 5-4. The ultimate loads of the FRP strengthened slabs that experienced failure due to intermediate crack are compared to the intermediate crack debonding models and presented in Figure 5-12 thru Figure 5-17. These results have been normalized with respect to the experimental results, such that a value greater than one indicates that the model is conservative with respect to the test results while a value less than one indicates that the model is unconservative with respect to the test results.

![Figure 5-12 IC Debonding model predictions normalized to measured results for Slab.4ksi.1UD.011911](image)

Figure 5-12 IC Debonding model predictions normalized to measured results for Slab.4ksi.1UD.011911
Figure 5-13 IC debonding model predictions normalized to measured results for Slab.4ksi.2UD.011911

Figure 5-14 IC Debonding model predictions normalized to measured results for Slab.4ksi.1UD.051211
Figure 5-15 IC Debonding model predictions normalized to measured results for Slab.4ksi.1BD.051211

Figure 5-16 IC Debonding model predictions normalized to measured results for Slab.8ksi.1UD.011911
These results indicate that the IC debonding model proposed in ACI 440.2R-08 and the model proposed by Said & Wu predict very similar results when compared to the experimental test results. Both of these models, in general, closely predict the failure loads due to intermediate crack debonding for the reinforced concrete slabs with 4ksi target concrete strength. The model proposed by the Japanese Society of Civil Engineers is not as accurate in predicting the failure loads due to intermediate cracks for the reinforced concrete slabs with 4ksi target concrete strength and consistently predict conservative failure loads for these slabs. The model used in ACI 440.2R-08 and the model proposed by Said & Wu are not as accurate for the reinforced concrete slabs with 8ksi target concrete strength as they both give unconservative failure loads compared to the test results. The model proposed by the Japanese Society of Civil Engineers though gives very accurate predictions for the reinforced concrete slabs with 8ksi target concrete strength. A summary of the predicted maximum load for the 4ksi and 8ksi strengthened slab is given in Table 5-5 and Table 5-6, respectively, for each IC debonding model discussed as well as a predicted maximum load when debonding is not considered. All of these values are compared to the experimental results.
Table 5-5 Experimental Results vs. Predictions for 4ksi Slabs

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Experimental Result (kips)</th>
<th>ACI 440 – capacity</th>
<th>ACI 440 - IC debonding</th>
<th>Said &amp; Wu - IC debonding</th>
<th>JSCE - IC debonding</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab.4ksi.none.011911</td>
<td>11.56</td>
<td>1.13</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Slab.4ksi.1UD.011911</td>
<td>25.30</td>
<td>1.12</td>
<td>1.12</td>
<td>1.12</td>
<td>1.35</td>
</tr>
<tr>
<td>Slab.4ksi.2UD.011911</td>
<td>28.2</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.05</td>
</tr>
<tr>
<td>Slab.4ksi.1UD.051211</td>
<td>22.81</td>
<td>0.99</td>
<td>0.99</td>
<td>0.99</td>
<td>1.21</td>
</tr>
<tr>
<td>Slab.4ksi.1BD.051211</td>
<td>22.03</td>
<td>0.96</td>
<td>0.96</td>
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Table 5-6 Experimental Results vs. Predictions for 8ksi Slabs

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<th>Specimen ID</th>
<th>Measured Result (kips)</th>
<th>ACI 440 – capacity</th>
<th>ACI 440 - IC debonding</th>
<th>Said &amp; Wu - IC debonding</th>
<th>JSCE - IC debonding</th>
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</table>
5.2. T-Beams Strengthened for Flexure

A total of eight (8) reinforced concrete T-beams, designed to fail in flexure, were tested. The parameters considered were concrete strength, type of CFRP fabric (uni-directional vs. bi-directional), and number of CFRP layers. It was found that the CFRP strengthening system increased the overall load carrying capacity and the stiffness of the T-beams. Further, it was found that the performance of the T-beam strengthened with a uni-directional fabric was almost identical to the T-beam reinforced with the bi-directional fabric, as they have almost identical ultimate loads and deflections. A summary of test results of the tested reinforced concrete T-beams is given in Table 5-7 and Table 5-8 cast on 01/19/11 and 05/12/11, respectively. The results in Table 5-8 are given to compare the effect of using uni-directional or bi-directional CFRP sheets.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Max Load (kips)</th>
<th>Load Increase over Control (%)</th>
<th>Deflection at Failure (in)</th>
<th>Reduction in Deflection from Control (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FlexBeam.4ksi.none.011911</td>
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<td>37</td>
<td>1.21</td>
<td>-69</td>
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<tr>
<td>FlexBeam.4ksi.2UD.011911</td>
<td>44.24</td>
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<td>0.95</td>
<td>-76</td>
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<tr>
<td>FlexBeam.8ksi.none.011911</td>
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<td>FlexBeam.8ksi.1UD.011911</td>
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<td>26</td>
<td>1.16</td>
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<td>53.21</td>
<td>46</td>
<td>1.12</td>
<td>-79</td>
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### Table 5-8 Test Results for Reinforced Flexure T-Beams cast on 05/12/11

<table>
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<th>Specimen</th>
<th>Max Load (kips)</th>
<th>Deflection</th>
</tr>
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<td>FlexBeam.4ksi.1UD.051211</td>
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<td>46.14</td>
<td>1.50</td>
</tr>
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</table>

All T-beams were tested in stages until failure due to crushing of the concrete in the compression zone or debonding of the strengthening system from the reinforced T-beams. The typical load-deflection behavior of the reinforced T-beams, in general, was characterized by a linear relationship until the initiation of first crack, as shown in Figure 5-18. After the initiation of first crack, there was a decrease in the stiffness and the load-deflection behavior followed a non-linear behavior until yielding of the reinforcing steel. Yielding of the longitudinal steel reinforcing was confirmed by the load-strain relationship of the beam, as shown in Figure 5-19. After yielding of the reinforcing steel, there was a subsequent decrease in the stiffness and the load-deflection behavior followed a non-linear relationship until failure. The load-deflection behavior of the T-beams strengthened for flexure is grouped based on target concrete compressive strength and date of cast as discussed in the following sections.
Figure 5-18 Load-Deflection Behavior for FlexBeam.4ksi.2UD.0111911
The load-displacement behavior of the reinforced concrete T-beam cast with 4ksi target nominal compressive strength on 01/19/11 and strengthened with one layer and two layers of CFRP are shown in Figure 5-20 in comparison to the control specimen. There was an increase in the stiffness of the T-beams strengthened with the CFRP systems due to the delay of cracking of the concrete and yielding of the steel, as shown in Figure 5-20. Test results indicate that using two layers of CFRP slightly increased both the strength and stiffness and reduced the ductility of the strengthened T-beams because the additional layer of CFRP is able to provide a greater tensile force to balance the compressive force in the concrete prior to failure. The addition of a second layer also increased the overall reinforcement ratio of the beam, causing an increase of the stiffness and subsequent decrease in deformation of the beam compared to using one layer of CFRP. The results also indicate that there is significant loss of ductility when using the CFRP strengthening system which can be attributed to the
premature failure of the T-beams from intermediate crack debonding, as shown in Figure 5-21.

**Figure 5-20** Load-Displacement Behavior for 4ksi Flexure T-Beams Cast on 01/19/11
The load-displacement behavior of the reinforced concrete T-beams cast with 8ksi target nominal compressive strength concrete on 01/19/11 and strengthened with one layer and two layers of CFRP are shown in Figure 5-22 in comparison to the control specimen. The behavior indicates an increase in stiffness of the T-beams strengthened with the CFRP systems due to the delay in cracking of the concrete and yielding of the steel reinforcement, as shown in Figure 5-22. For the high strength concrete, the effect of the use of two layers is more pronounced, as was the case for the reinforced slabs. The use of high strength concrete provides a more effective use of the two layers since the CFRP provides high tensile forces to react with the presence of the high internal compression forces of the concrete for the high strength concrete. The results also indicate that there is significant loss of ductility when using the CFRP strengthening system which can be attributed to the premature failure of the T-beams from intermediate crack debonding, as shown in Figure 5-21.
The load-deflection behavior of the T-beams cast on 05/12/11, with a 4ksi target nominal compressive strength concrete to compare the behavior of uni-directional and bi-directional CFRP sheets, is shown in Figure 5-23. The load-deflection behavior of the T-beams is virtually identical. Therefore, test results suggest that the presence of transverse fiber is ineffective because they do not increase the longitudinal reinforcement ratio of the T-beam, which is the main influence on the load-deflection behavior.
The observed failure mode for the control specimens was crushing of the concrete in the compression zone, as shown in Figure 5-24, while the typical observed failure mode for most of the strengthened specimens was debonding of the CFRP caused by intermediate cracks, as shown in Figure 5-25. This debonding mechanism was the dominant form of debonding in flexural strengthened reinforced concrete T-beams. These cracks are typically located within the shear span, the distance from the point load to the nearest support, and induce high interfacial shear stress that cause FRP debonding in the cases that the induced shear strength exceeds the bond strength. These cracks typically propagate along the shear span in the direction of decreasing moment, as shown in Figure 5-26. It was observed that this type of failure does not penetrate into the aggregate, but rather progresses through the thin mortar-rich layer comprising the surface of the concrete.
Figure 5-24 Typical Failure for Flexure Control T-Beams
Figure 5-25 Typical Failure for Strengthened Flexure T-Beams

Figure 5-26 Propagation of Intermediate Cracks through the Shear Span for Flexure T-beams
Based on the observed behavior, the results indicate in general that the use of CFRP systems for strengthening increased the load carrying capacity. The strengthening systems also increase the cracking and yielding loads in comparison to the control specimens. The increase in the stiffness of the strengthened T-beams in comparison to the control specimens is caused by the presence of the CFRP system which increases the overall reinforcement ratio of the beam causing an increase of the stiffness. All of the strengthened T-beams had a decrease in deflection, consequently the ductility, compared to the control specimens. This reduction in deflection is due to increase of the reinforcement ratio which results in reduction of the number and depth of the cracks before failure when compared to the control specimens. Figure 5-27 shows the load deflection behavior of all the tested flexure T-beams strengthened with uni-directional fibers using one and two layers of fibers. The behavior of the control T-beam with target compressive strength of 4ksi and 8ksi is almost identical since both T-beams were under reinforced “tension controlled” and have the same reinforcement ratio. As discussed, the use of two layers of fibers increases the flexural strength and slightly reduces the ductility in comparison to the use of one layer of fibers.
For each T-beam, three 4x8 inch concrete cylinders were tested to determine concrete compressive strength at the day of testing. Summary of the measured average concrete strengths for three cylinders are given Table 5-9. Concrete strength for the cast on 05/12/11 is given in Table 5-10. The average compressive strength of the 4ksi and 8ksi target compressive strength concrete cast on 01/19/11 was 5.6ksi and 7.0ksi, respectively. The average compressive strength of the 4ksi target compressive strength concrete cast on 05/12/11 was 6.0ksi, as given in Table 5-10.
The models discussed in the literature review for predicting the intermediate crack debonding failure load of a reinforced concrete member flexurally strengthened with FRP material were compared to the results of the reinforced concrete slabs. These models used the material properties of the CFRP and steel reinforcement as discussed in chapter four as well as day of testing concrete strength, as given in Table 5-9 and Table 5-10. The ultimate loads of the FRP strengthened flexure T-beams that experienced failure due to intermediate crack are compared to the intermediate crack debonding models and presented in Figure 5-28 thru Figure 5-33. These predictions have been normalized with respect to the measured results, such that a value greater than one indicates that the IC debonding model is conservative with respect to the test results while a value less than one indicates that the model is unconservative with respect to the test results.
Figure 5-28 IC Debonding models normalized to measured results for FlexBeam.4ksi.1UD.011911

Figure 5-29 IC Debonding models normalized to measured results for FlexBeam.4ksi.2UD.011911
Figure 5-30 IC Debonding models normalized to measured results for FlexBeam.4ksi.1UD.051211

Figure 5-31 IC Debonding models normalized to measured results for FlexBeam.4ksi.1BD.051211
Figure 5-32 IC Debonding models normalized to measured results for FlexBeam.8ksi.1UD.011911

Figure 5-33 IC Debonding models normalized to measured results for FlexBeam.8ksi.2UD.011911
In general, the models used to compare test results of the flexure T-beams strengthened with FRP were fairly accurate in predicting the IC debonding failure loads. For the flexure T-beams with 4ksi target strength concrete, the ACI 440.2R-08 IC debonding model gave the most consistent results when compared to the test results while the Said & Wu model and the model proposed by the Japanese Society of Civil Engineers were more inconsistent when compared to the test results. All of the models, in general, where fairly accurate in predicting the failure loads for the flexure T-beams. The ACI 440.2R-08 debonding model and the model proposed by Said & Wu were unconservative for the all the T-beams while the model proposed by the Japanese Society of Civil Engineers was conservative when compared to the test results. A summary of the predicted maximum load for the 4ksi and 8ksi strengthened flexure beam is given in Table 5-11 and Table 5-12, respectively, for each IC debonding model discussed. This table also presents the predicted maximum load carrying capacity for each specimen when debonding was not considered. All of these values are compared to their respective test results.

### Table 5-11 Experimental Results vs. Predictions for 4ksi Flexure Beams

<table>
<thead>
<tr>
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<th>Test Result (kips)</th>
<th>Ratio of Measured/Prediction</th>
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<tbody>
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<td></td>
<td>ACI 440 - capacity</td>
<td>ACI 440 - IC debonding</td>
</tr>
<tr>
<td>FlexBeam.4ksi.none.011911</td>
<td>26.46</td>
<td>1.26</td>
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<td>FlexBeam.4ksi.1UD.011911</td>
<td>41.83</td>
<td>0.77</td>
</tr>
<tr>
<td>FlexBeam.4ksi.2UD.011911</td>
<td>44.24</td>
<td>0.50</td>
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<tr>
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<td><strong>Standard deviation:</strong></td>
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### Table 5-12 Experimental Results vs. Predictions 8ksi Flexure Beams

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<th>Specimen ID</th>
<th>Test Result (kips)</th>
<th>ACI 440 - capacity</th>
<th>ACI 440 - IC debonding</th>
<th>Said &amp; Wu - IC debonding</th>
<th>JSCE - IC debonding</th>
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<tr>
<td>FlexBeam.8ksi.none.011911</td>
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<td><strong>1.11</strong></td>
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<td><strong>Standard deviation:</strong></td>
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<td><strong>0.05</strong></td>
<td><strong>0.03</strong></td>
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5.3. T-Beams Strengthened for Shear

A total of eight (8) reinforced T-beams that were designed to fail in shear were tested. The parameters investigated were the concrete strength and effect of using anchorage for the U-wraps compared to traditional U-wraps for shear strengthening. In general, test results indicate that the CFRP strengthening system for shear increased the overall shear load carrying capacity of the strengthened T-beam. Further, it was found that the performance of the T-beam strengthened with anchored CFRP U-wrap increases the load carrying capacity of the T-beam when compared to the CFRP U-wrap without anchorage. A summary of test results for the beam strengthened for shear are given in Table 5-13.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Max Load (kips)</th>
<th>Load Increase over Control (%)</th>
<th>Deflection at Failure (in)</th>
<th>Increase in Deflection from Control (%)</th>
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<td>16</td>
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<tr>
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<td>9</td>
<td>0.76</td>
<td>16</td>
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</table>

All tests were tested in stages until failure. The load-deflection behavior of the reinforced T-beams, in general, was characterized by a linear relationship until the initiation of first crack, as shown in Figure 5-34. After the initiation of first crack, there was a decrease in the stiffness and the load-deflection behavior followed a non-linear behavior until yielding of the
transverse reinforcing steel. Yielding of the transverse reinforcement was confirmed by the load-strain behavior of the beam, as shown in Figure 5-35. After yielding of the transverse reinforcing steel, there was a subsequent decrease in the stiffness and the load-deflection behavior followed a non-linear relationship until failure. The load-deflection behavior of the T-beams strengthened for shear is grouped based on target concrete compressive strength and date of cast as discussed in the following sections.

Figure 5-34 Load-Deflection Behavior for ShearBeam.4ksi.WrapsUD.011911
The load-deflection of the T-beams with target 4ksi target nominal compressive strength concrete strength with wraps and anchored wraps are shown in Figure 5-36. The results indicate that the presence of the strengthening systems increased both the strength and deflection at failure in comparison to the control beams. The increase in measured deflection at failure can be associated with the higher load capacity achieved by the T-beam in comparison to the control. The increased load capacity can be related to the increase in shear capacity of the T-beam due to the CFRP U-wraps. The results also indicate that there was an increase in the stiffness post-yield of the transverse reinforcement. The increase in the stiffness is due to the resistance of shear deformation of the CFRP U-wraps analogous with an increase in load. There was also an increase in the stiffness of the T-beam strengthened with the anchored U-wraps due to the resistance of shear deformation of the anchored CFRP U-wraps corresponding with an increase in load. The flexural load-strain behavior clearly
shows that not all the longitudinal steel reinforcement had yielded before failure, as evident in the linear behavior of the load-strain relationship shown in Figure 5-37. This figure also indicates that the yielding strain had been reached prior to failure, suggesting that the extreme fiber of the first layer of steel may have begun to yield prior to failure of the beam.

Figure 5-36 Load-Deflection Behavior for 4ksi Shear T-Beams Cast on 01/19/11
The load-deflection of the T-beams with target 8ksi target nominal compressive strength concrete strength with wraps and anchored warps are shown in Figure 5-38. The results indicate that the effect of the CFRP U-wraps as well as the anchored CFRP U-wraps was ineffective for the 8ksi T-beams. The ineffectiveness of the U-wraps and anchors could have been associated with the premature debonding of the U-wraps, leading to the premature propagation of the critical shear crack from the load point to support, as shown in Figure 5-40 and Figure 5-41, respectively. Some of the issues of premature debonding can be traced to fabrication of the strengthening system. During installation, some of the U-wraps had scotch tape left on the ends of the wrap, causing a poor bond to from between the CFRP and concrete at this location. The scotch tape was used when the FRP sheets were cut to their required lengths prior to installation; the scotch tape ensured that the fibers were cut evenly and in a straight line. During testing, the shear cracks were able to propagate through this
“weak bond” section of the CFRP strengthening system, limiting its effectiveness. Evidence of the ineffectiveness of the strengthening systems is shown in Figure 5-38, where only a slight increase in the stiffness of the T-beam post-yield of the transverse reinforcement is present. This behavior suggests that there was poor bond between the wraps and concrete, possibly due to the fabrication issues, as discussed previously. The load-strain behavior of the 8ksi clearly indicates that the longitudinal steel reinforcement had yielded prior to failure of the specimen, as shown in Figure 5-39.

![Figure 5-38 Load-Deflection Behavior for 8ksi Shear T-Beams cast on 01/19/11](image)
Figure 5.39 Typical Load-Strain Behavior of Longitudinal Steel for 8ksi Shear Beam
Figure 5-40 Premature Debonding of 8ksi U-Wrap System due to Tape
The observed failure mode for the control specimens was shear failure due to the formation of a critical shear crack at one of the applied load points that propagated toward the support, as shown in Figure 5-42. The failure mode for the strengthened specimens was also due to a critical shear crack. Failure of the specimens that were strengthened with CFRP U-wraps was due to debonding of the U-wrap from the side of the web where the main shear crack propagated through to the support, as shown in Figure 5-43 and Figure 5-44. The specimens strengthened with the anchored CFRP U-wraps failed due to rupturing of the anchors at the location of the main shear crack near the support and the subsequent debonding of the U-wrap from the side of the web which allowed the propagation of the critical diagonal shear crack, as shown in Figure 5-45 and Figure 5-46.
Figure 5-42 Typical Failure for Control Shear T-Beam

Figure 5-43 Typical Shear Failure for U-Wrap Shear Strengthened T-Beam
Figure 5-44 Typical Debonding Failure of U-Wraps

Figure 5-45 Typical Failure for Anchor U-Wrap Shear Strengthened T-Beam
Based on the observed behavior, the results indicate in general that the use of CFRP systems for strengthening increased the load carrying capacity. The strengthening systems also, in general, increased the loads required to induce cracking of the concrete and yielding of the transverse reinforcing steel in comparison to the control specimens, as shown in detail in Appendix A. The increase in the stiffness of the strengthened 4ksi T-beams in comparison to the control specimens are caused by the CFRP system resisting the shear deformation caused by loading. Test results indicated that there was a greater increase in the stiffness post-yield for the specimens strengthened with the anchored U-wraps. This behavior can be explained by the presence of the anchors, which delayed the debonding of the U-wraps and ruptured prior to debonding of the U-wraps. Therefore, they experienced higher loads and deflections compared to the specimens strengthened with the U-wraps. All of the strengthened T-beams had an increase in deflection compared to the control specimens. The increase in deflection is due to the capability of the beam to carry more load due to the presence of the CFRP strengthening which was able resist greater shear forces before debonding of the U-wraps or
rupture of the anchorage. Figure 5-47 shows the load deflection behavior of all the tested shear T-beams strengthened with CFRP U-wraps and anchored CFRP U-wraps. As discussed, use of the anchored U-wrap system increases the shear strength and the deflection in comparison to the use of U-wraps only. Test results indicate also that use of high strength concrete increases the shear resistance of the control beams due to increasing the capacity of the compression strut in the shear mechanism.

![Figure 5-47 Load-Deflection Behavior for all Shear Beams Cast on 01/19/11](image)

For each tested beam, three 4x8 concrete cylinders were tested to determine the concrete compressive strength at the day of testing. Summary of the average concrete strengths are given in Table 5-14. The average compressive strength of the 4ksi and 8ksi target compressive strength concrete cast on 01/19/11 was 5.7ksi and 7.1ksi, respectively.
The model discussed in the literature review for predicting the failure load for a reinforced concrete member shear strengthened with FRP material were compared to the results of the reinforced concrete slabs. The model used the material properties of the CFRP and steel reinforcement as discussed in chapter four as well as day of testing concrete strength, as given in Table 5-14. A summary of the predicted maximum load compared to the experimental results is given in Table 5-15 and Table 5-16 for the 4ksi and 8ksi shear strengthened T-beams, respectively.

### Table 5-14 Compressive Concrete Strength for Shear Beams Cast on 01/19/11

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<th>Specimen</th>
<th>Compressive Concrete Strength (ksi)</th>
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</tr>
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</tr>
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</tr>
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### Table 5-15 Experimental Results vs. Prediction for 4ksi Shear Beams

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<th>Specimen ID</th>
<th>Measured Result (kips)</th>
<th>Measured/ACI 440 Prediction</th>
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</thead>
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<tr>
<td>ShearBeam.4ksi.none.011911</td>
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<td>1.46</td>
</tr>
<tr>
<td>ShearBeam.4ksi.WrapsUD.011911</td>
<td>181.57</td>
<td>1.55</td>
</tr>
<tr>
<td>ShearBeam.4ksi.AnchoredWrapsUD.011911</td>
<td>204.21</td>
<td>1.74</td>
</tr>
<tr>
<td>Average:</td>
<td>1.58</td>
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</tr>
<tr>
<td>Standard deviation:</td>
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</tbody>
</table>


Table 5-16 Experimental Results vs. Prediction for 8ksi Shear Beams

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Measured Result (kips)</th>
<th>Measured/ACI 440 Prediction</th>
</tr>
</thead>
<tbody>
<tr>
<td>ShearBeam.8ksi.none.011911</td>
<td>178.53</td>
<td>1.83</td>
</tr>
<tr>
<td>ShearBeam.8ksi.WrapsUD.011911</td>
<td>188.18</td>
<td>1.50</td>
</tr>
<tr>
<td>ShearBeam.8ksi.AnchoredWrapsUD.011911</td>
<td>195.21</td>
<td>1.55</td>
</tr>
</tbody>
</table>

Average: 1.63
Standard deviation: 0.18

These results indicate that the model presented in ACI 440.2R-08 is very conservative when predicting the contribution of the FRP strengthening system for shear resistance. The results also indicate that the contribution of the stirrups and concrete in resisting shear are also very conservative in the model, as evident in the large discrepancy between the predicted values for the control specimens compared to the experimental results. A possible explanation to these results is that the critical shear crack angle was less than 45 degrees in the experiment, as assumed in the ACI model, such that more stirrups contributed to the shear resistance than was predicted. The assumption of the critical shear crack angle of 45 degrees would also affect the model’s prediction of the contribution of the FRP to the resistance of shear, as it would under predict the number of U-Wraps that contributed to the shear resistance of the beam as well.
5.4. Reinforced Concrete Columns

A total of six (6) reinforced columns were tested in this experimental program. The parameters investigated were concrete strength and number of CFRP layers used for strengthening. Test results indicate, in general, that the CFRP strengthening system increased the overall load carrying capacity and the stiffness of the strengthened columns. Summary of test results for the columns cast on 03/07/11 are given in Table 5-17.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Max Load (kips)</th>
<th>% Load Increase</th>
<th>Axial Strain at Max Load (in/in)</th>
<th>% Increase in Axial Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column.4ksi.none.030711</td>
<td>566</td>
<td>N/A</td>
<td>0.0030</td>
<td>N/A</td>
</tr>
<tr>
<td>Column.4ksi.1UD.030711</td>
<td>652</td>
<td>13</td>
<td>0.0041</td>
<td>38</td>
</tr>
<tr>
<td>Column.4ksi.2UD.030711</td>
<td>756</td>
<td>25</td>
<td>0.0132</td>
<td>346</td>
</tr>
<tr>
<td>Column.8ksi.none.030711</td>
<td>1001</td>
<td>N/A</td>
<td>0.0032</td>
<td>N/A</td>
</tr>
<tr>
<td>Column.8ksi.1UD.030711</td>
<td>1046</td>
<td>4</td>
<td>0.0045</td>
<td>43</td>
</tr>
<tr>
<td>Column.8ksi.2UD.030711</td>
<td>1051</td>
<td>5</td>
<td>0.0043</td>
<td>36</td>
</tr>
</tbody>
</table>

All tests were conducted in stages, ending with failure due to crushing of the core concrete or rupture of the strengthening system as well as buckling of the longitudinal rebar, as shown in Figure 5-48.
The typical load-deflection behavior of the strengthened columns, in general, was characterized by a linear relationship until yielding of the transverse reinforcement, as shown in Figure 5-49. Due to the passive nature of the transverse reinforcement, the confinement pressure increased linearly until the transverse reinforcement reached its yield strength. The transverse reinforcement provides passive confinement because the confining pressure reacts only to the dilation of the core concrete. The confining pressure provided by the transverse steel reinforcement increases linearly up to yielding of the transverse reinforcement. At this stage, the transverse steel reinforcement provided constant confinement to the core concrete equivalent to the yield strength of the transverse reinforcement. Yielding of the transverse reinforcement was confirmed when examining the measured load-strain relationships of the axial compressive and transverse tensile strains of the column at mid-height, as shown in Figure 5-50. Presence of the CFRP provides additional confinement and increases the load.
carry capacity of the columns. The behavior indicates a decrease of the stiffness after yielding of the transverse reinforcement since the elastic modulus of the CFRP, which is activated at this stage, is less than that of the transverse steel causing the a non-linear behavior, as shown in Figure 5-49. The additional load carrying capacity of the column after yielding of the transverse steel reinforcement is a function of the FRP only. The load-deflection behavior of the column is non-linear until yielding of the core concrete begins to fail due to uncontrolled dilation and subsequent yielding of the longitudinal reinforcement, as shown in Figure 5-49. After the core concrete began to fail, there was a subsequent significant decrease in the stiffness and load carrying capacity followed by a non-linear relationship until rupture of the FRP strengthening system, as shown in Figure 5-49.

![Figure 5-49 Load-Axial Deflection Behavior for Column.4ksi.1UD.030711](image_url)
The load-displacement behavior of the reinforced concrete columns cast on 03/07/11 with 4ksi target nominal strength concrete and strengthened with one layer and two layers of CFRP wrapping is shown in Figure 5-51 in comparison to the control specimen. These columns were cast with a 4ksi target compressive strength concrete on 03/07/11. The load-deflection behavior of the columns followed a linear relationship until the yielding of the transverse reinforcing ties. At this point, as shown in Figure 5-51, there was a decrease in the stiffness of the column due to the formation and propagation of large vertical cracks within the core concrete and the load-deflection behavior became highly non-linear. Once the core concrete began to fail, in general, there was a drop in load followed by failure of the column. The column strengthened with one and two layers of CFRP wrapping experienced an increase of the load carrying capacity due to the presence of the CFRP wraps which provided additional confinement and controlled the dilation of the concrete core. There was an increase in both maximum axial load carrying capacity and axial deflection of the columns.
strengthened with the CFRP wrapping compared to the control. Test results indicate that using two layers of CFRP wrapping increased the strength, the stiffness, and the axial deflection of the strengthened columns compared to one layer due to the increase of the CFRP transverse reinforcement ratio which lead to a larger increase of the confinement pressure on the core concrete of the column. The larger confinement pressure delayed the dilation of core concrete and therefore increased the axial load carrying capacity.

![Load-Axial Deflection Behavior for 4ksi Columns cast on 03/07/11](image)

**Figure 5-51 Load-Axial Deflection Behavior for 4ksi Columns cast on 03/07/11**

The load-deflection behavior of the columns cast on 03/07/11 with the 8ksi target nominal compressive strength concrete and strengthened with one layer and two layers of CFRP wrapping compared to the control column is shown Figure 5-52. The load-deflection behavior of the columns, in general, followed a linear relationship until yielding of the transverse reinforcement. At this stage, there was a decrease in the stiffness and the load-
deflection behavior became non-linear as shown in Figure 5-52. As the applied load induced a strain equal to the compressive strength of the concrete, vertical cracks were created followed by dilation of the column in the transverse direction. Typically, this would lead to yielding of the transverse reinforcement for typical column design which cannot control the propagation of vertical cracks and transverse dilation, leading to failure of the column. After this stage, there was a drop in load followed by failure of the column due to rapid uncontrolled dilation of the core concrete. Use of the CFRP wrapping controlled the vertical crack propagation and therefore increased the load carrying capacity and the axial deflection of the strengthened columns in comparison to the control column. Test results indicate that using two layers of CFRP wrapping increased the strength and the axial deflection of the strengthened columns by increasing the transverse reinforcement ratio of the columns. The increase in the transverse reinforcement ratio allowed an increase in the confining pressure on the core concrete, which delayed the dilation and crack propagation of the core concrete.
The behavior of the control column was characterized by spall off of the concrete cover followed by failure of the core concrete and buckling of the longitudinal bars, as shown in Figure 5-53. The typical observed failure mode for the strengthened columns was rupture of the FRP reinforcing followed by failure of the core concrete and buckling of the longitudinal bars, as shown in Figure 5-54. This failure mechanism is the dominant form of failure for reinforced concrete columns strengthened by CFRP wrapping. For a column under axial compression, the concrete begins to expand laterally, which is then restrained by the FRP wrapping. This mechanism induces tension in the hoop direction due to the dilation of the concrete. Failure will occur when the induced hoop tensile stress due to the applied load reaches the tensile strength of the FRP wrapping system. At this point the fibers will rupture, as shown in Figure 5-55.
Figure 5-53 Typical Failure for Reinforced Concrete Control Columns
Figure 5-54 Typical Failure of CFRP Strengthened Column
Figure 5-56 shows the load deflection behavior of all of the tested columns, strengthened with CFRP wraps cast on 01/19/11, using one and two layers. These results indicate that unlike the 4ksi columns, the effect of the CFRP wrapping was not as prominent for the 8ksi columns. These results may be attributed to the fact that high strength concrete does not have large rupture strain under axial compression in comparison to normal strength concrete at ultimate stress. Therefore dilation of core concrete becomes very limited to the extent not to activate the CFRP wrapping. As a result, the CFRP strengthening system will not be fully utilized prior to failure, as shown in Figure 5-56. As discussed, the use of two layers of CFRP increases the compressive strength and the axial deflection, in comparison to the use
of one layer of CFRP, for the 4ksi columns. The use of two layers of CFRP for the 8ksi columns increases the compressive strength and decreases the axial deflection in comparison to the use of one layer of CFRP. This behavior can be related to increasing the transverse reinforcement ratio, which increases the stiffness of the column and restrains the dilation of the core concrete prior to failure.

![Figure 5-56 Load-Deflection Behavior for all Columns cast on 03/07/11](image-url)

For each day of column testing, three 4x8 inch concrete cylinders were tested to determine concrete compressive strength at the day of testing. Summary of the measured average concrete strengths for three cylinders are given Table 5-9. The average compressive strength of the 4ksi and 8ksi target compressive strength concrete cast on 03/07/11 was 5.6ksi and 9.1ksi, respectively.
Table 5-18 Compressive Concrete Strength for Columns cast on 03/07/11

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Compressive Concrete Strength (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column.4ksi.none.030711</td>
<td>5.5</td>
</tr>
<tr>
<td>Column.4ksi.1UD.030711</td>
<td>5.5</td>
</tr>
<tr>
<td>Column.4ksi.2UD.030711</td>
<td>5.8</td>
</tr>
<tr>
<td>Average:</td>
<td>5.6</td>
</tr>
<tr>
<td>Column.8ksi.none.030711</td>
<td>9.0</td>
</tr>
<tr>
<td>Column.8ksi.1UD.030711</td>
<td>9.1</td>
</tr>
<tr>
<td>Column.8ksi.2UD.030711</td>
<td>9.1</td>
</tr>
<tr>
<td>Average:</td>
<td>9.1</td>
</tr>
</tbody>
</table>

The model discussed in the literature review for predicting the failure load for a reinforced concrete column strengthened with FRP material were compared to the measured results from testing the columns. The model used the material properties of the CFRP and steel reinforcement as discussed in chapter four as well as day of testing concrete strength, as given in Table 5-18. A summary of the predicted maximum load compared to the experimental results is given in Table 5-19 and Table 5-20 for the 4ksi and 8ksi axial compression strengthened columns, respectively.

Table 5-19 Experimental Results vs. Predictions for 4ksi Columns

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Measured Result (kips)</th>
<th>Ratio of Measured/Prediction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column.4ksi.none.030711</td>
<td>566</td>
<td>1.30</td>
</tr>
<tr>
<td>Column.4ksi.1UD.030711</td>
<td>652</td>
<td>1.33</td>
</tr>
<tr>
<td>Column.4ksi.2UD.030711</td>
<td>756</td>
<td>1.44</td>
</tr>
<tr>
<td>Average:</td>
<td></td>
<td>1.36</td>
</tr>
<tr>
<td>Standard Deviation:</td>
<td></td>
<td>0.08</td>
</tr>
<tr>
<td>Specimen ID</td>
<td>Measured Result (kips)</td>
<td>Ratio of Measured/Prediction</td>
</tr>
<tr>
<td>---------------------------</td>
<td>------------------------</td>
<td>------------------------------</td>
</tr>
<tr>
<td>Column.8ksi.none.030711</td>
<td>1001</td>
<td>1.50</td>
</tr>
<tr>
<td>Column.8ksi.1UD.030711</td>
<td>1046</td>
<td>1.45</td>
</tr>
<tr>
<td>Column.8ksi.2UD.030711</td>
<td>1051</td>
<td>1.39</td>
</tr>
<tr>
<td>Average:</td>
<td></td>
<td>1.45</td>
</tr>
<tr>
<td>Standard Deviation:</td>
<td></td>
<td>0.05</td>
</tr>
</tbody>
</table>

These results indicate that the model proposed in ACI 440.2R-08 was very conservative in predicting the maximum axial load compared to the experimental results. The results indicated that the contribution of the FRP system is not fully utilized in the model, leading to very conservative predictions for the failure load for the strengthened columns. The test results for the unstrengthened columns are also very conservative, indicating that the contribution of the transverse steel and concrete to resist axial load is not fully utilized in the model as well.
6. SUMMARY AND CONCLUSIONS

This chapter summarizes the research findings based on an experimental program conducted to examine the effectiveness of the use of a new CFRP material for strengthening of concrete structures. The experimental program consisted of the eight reinforced concrete slabs, eight reinforced concrete T-beams strengthened for flexure, six reinforced concrete T-beams strengthened to increase the shear capacity, and six reinforced concrete columns strengthened to increase the axial capacity. The effect of the CFRP strengthening system is examined for each type of application. The effect of the concrete strength, fiber reinforcement ratio, the use of bi-directional and uni-directional fibers, and the effect of the innovative anchorage system are discussed.

6.1. Flexural Strengthening of RC Slabs

A total of eight reinforced concrete slabs strengthened to increase the flexural capacity were tested to evaluate the effectiveness of the new carbon fiber reinforced polymer strengthening system. The different parameters that were examined for the reinforced concrete slabs were concrete strength, fiber reinforcement ratio, and type CFRP strengthening systems (uni-directional versus bi-directional). Test results indicate that there was a significant increase in ultimate load carrying capacity for the strengthened slabs compared to the control for the normal strength concrete slabs in comparison to the slabs cast using high strength concrete. Test results also indicate that there was a decrease in ductility of the strengthened slabs in comparison to the control for the normal strength concrete slabs more than the slabs cast with high strength concrete. Test results also indicate that increasing the fiber reinforcement ratio by adding a second layer of CFRP will increase the ultimate load carrying capacity when compared to the use of one layer of CFRP strengthening. Adding a second layer of CFRP had little effect on the ductility of the reinforced concrete slab when compared to the use of one layer of CFRP. The test results indicate also that the behavior of the reinforced concrete slab strengthened with bi-directional fiber was essentially identically to the slab strengthened
with uni-directional fiber. This result indicates that the addition of the transverse fiber does not enhance the performance or the strength of the reinforced concrete slabs.

6.2. Flexural Strengthening of RC T-Beams

A total of eight reinforced concrete T-beams strengthened to increase the flexural capacity were tested to evaluate the effectiveness of the new CFRP strengthening system. The different parameters that were examined for the reinforced concrete T-beams were concrete strength, fiber reinforcement ratio, and type CFRP strengthening systems (uni-directional versus bi-directional). Test results indicate that the concrete strength of the reinforced concrete T-beams had a negligible effect on the performance of the flexure T-beams strengthened with a CFRP system. All reinforced concrete T-beams strengthened with a CFRP strengthening system had an increase in ultimate load carrying capacity and decrease in ductility when compared to their respective control T-beams. Test results also indicate that increasing the fiber reinforcement ratio by adding a second layer of CFRP will increase the ultimate load carrying capacity when compared to the use of one layer of CFRP strengthening. Adding a second layer of CFRP had a negligible effect on the ductility of the reinforced concrete T-beam when compared to the one layer CFRP strengthening system. Test results also indicate that the behavior of the reinforced concrete T-beam strengthened with bi-directional fiber was essentially identically to the T-beam strengthened with uni-directional fiber. This result indicates that the addition of the transverse fiber does not enhance the performance or the strength of the reinforced concrete T-beams.

6.3. Shear Strengthening of RC T-beams

A total of six reinforced concrete T-beams strengthened to increase the shear capacity were tested to evaluate the effectiveness of the new CFRP strengthening system. The parameters investigated were the concrete strength and effect of using an innovative anchorage system for the U-wraps in comparison to the traditional U-wraps typically used for shear strengthening. Test results indicate that there was a larger increase in ultimate load carrying
capacity of the strengthened T-beams compared to the control for the normal strength concrete T-beams in comparison to the T-beams cast with high strength concrete. Test results also indicate that there was a relative increase in the ductility compared to the control for the normal strength concrete T-beams in comparison to the T-beams cast with high strength concrete. All reinforced concrete T-beams strengthened with a CFRP strengthening system had an increase in ultimate load carrying capacity and increase in ductility when compared to their respective control T-beams. The test results also indicate that using a CFRP anchorage system with the traditional CFRP U-wrap increases the ultimate load carrying capacity and ductility when compared to the use of the CFRP U-wrap system only.

6.4. Axial Compression Strengthening of RC Columns
A total of six reinforced concrete columns were wrapped to increase the axial load carrying capacity and evaluate the effectiveness of the new carbon fiber reinforced polymer strengthening system. The different parameters that were examined for the reinforced concrete columns were concrete strength and fiber reinforcement ratio. Test results indicate that there was a larger increase in ultimate load carrying capacity compared to the control for the normal strength concrete columns in comparison to the columns cast with high strength concrete. Test results indicate also that there was a greater increase in axial deformation in comparison to the control for the normal strength concrete columns compared to columns cast with high strength concrete. Test results also indicate that increasing the fiber reinforcement ratio, by adding a second layer of CFRP, increased the ultimate load carrying capacity in comparison to the use of one layer of CFRP strengthening. Test results were inconclusive on the effects of adding a second layer of CFRP on the axial deformation of the reinforced concrete columns when compared to using one layer of CFRP.

6.5. Future Work
Based on the work presented in this experimental study, the following topics are recommended for future research:
1. Evaluate the behavior of reinforced concrete members strengthened with the new carbon fiber reinforced polymer system under sustained loading.

2. Evaluate the behavior of reinforced concrete members strengthened with the new carbon fiber reinforced polymer system under cyclic loading.

3. Evaluate the behavior of reinforced concrete members strengthened with the new carbon fiber reinforced polymer system under loading that includes the effect of different environmental conditions.

4. Conduct additional tests using the new CFRP anchorage system to determine its effect on shear strengthening of reinforced concrete members.

5. Evaluate the effect of surface preparation, diamond-disk grinding versus sand blasting, on the behavior of reinforced concrete members strengthened with the new CFRP strengthening system.
REFERENCES


APPENDICES
APPENDIX A

This appendix provides detailed test results for each of the tested large specimens in the experimental program.
This reinforced slab was cast using 4ksi target nominal compressive strength concrete on January 19, 2011. The slab was not strengthened with any CFRP system and it was used as a control slab to study the effectiveness of the various flexural strengthening systems for the slabs cast with 4ksi target compressive concrete strength on the same day. The measured load-deflection behavior at mid-span for this slab is shown in Figure A-1.

The slab was loaded up to an equivalent pre-determined service load of 3.8 kips and then unloaded. The load-deflection behavior shown in Figure A-1, was linear up to the initiation of the first crack, which occurred at a load level of 2.53 kips. The behavior after the initiation of first crack was nonlinear with a reduction of the stiffness. After initial unloading, a residual deflection at midspan of 0.12 inches was measured. The slab was then...
loaded up to a pre-determined ultimate load of 6.9 kips and then unloaded. The load-deflection behavior was nonlinear until unloading as shown in Figure A-1. In the third loading stage; the slab was loaded until failure. The behavior of the load-deflection graph had a significant reduction of the stiffness during this portion of the loading due to yielding of the reinforcing steel at an applied load of 9.61 kips, as shown in Figure A-1. After yielding of the reinforcing steel, the load-deflection behavior was nonlinear until failure, as shown in Figure A-1.

The measured load-strain relationships of the concrete at the top compression surface and bottom tension surface of the concrete as well at the level of the longitudinal tension steel for the control slab are shown in Figure A-2. The measured strains represent the average strain within the gauge length of the pi gauges. Test results indicate that the maximum measured concrete strain in compression was 0.009 and that the steel reinforcement had yielded at a strain of 0.0014 before failure.
The slab failed at a load of 11.6 kips due to crushing of the concrete in the region of maximum moment, which is the area between the two load points. The measured deflection at failure was 9.1 inches and the deflected shape of the control specimen prior to failure is shown in Figure A-3. At failure, there was significant propagation of a large number of flexure cracks, as shown in Figure A-4, and the failure was very ductile.
Figure A-3 Deflected Shape of the Control Slab at Failure

Figure A-4 Crack Pattern of Control Slab at Failure
This reinforced slab was cast using 4ksi target nominal compressive strength concrete on January 19, 2011. The slab was strengthened with one layer of uni-directional CFRP. The one layer of flexural strengthening consisted of two 12” wide CFRP sheets laid next to each other to cover the entire width of the bottom surface of the slab, as shown in Figure A-5. The measured load-deflection behavior at mid-span for this slab is shown in Figure A-6.
The slab was loaded up to an equivalent service load of 4.5 kips and then unloaded. The load-deflection behavior, shown in Figure A-6, was linear during this stage of loading. After initial unloading, a residual deflection at midspan of 0.03 inches was measured, compared to 0.12 inches for the control specimen. This behavior is due to the presence of the CFRP sheet, which is linear elastic and contributes to reduce the permanent deformation in comparison to the control slab. The slab was then loaded up to a pre-determined ultimate load of 14.0 kips and then unloaded. From the load-deflection behavior relationship in Figure A-6, first cracking occurs at 4.8 kips, compared to 2.53 kips for the control slab. Again, the increase of the cracking load capacity, in comparison to the control slab, is due to the presence of the CFRP sheet which delayed the initiation of the cracking load. The behavior after the initiation of first crack was nonlinear with a reduction of the stiffness, as shown in Figure A-6. In the third loading stage, the slab was loaded until failure. The load-deflection behavior
was nonlinear as shown in Figure A-6 during ultimate loading until the yielding of reinforcing steel, at an applied load of 16.0 kips compared to 9.61 kips for the control. At this point, there was a significant reduction of the stiffness and the load-deflection behavior followed a non-linear relationship up to failure.

The measured flexural load-strain relationships of the concrete at the top compression surface, surface of the CFRP strengthening system, and at the level of longitudinal tension steel are shown in Figure A-7. The measured strains indicate that the maximum measured concrete strain in compression was 0.0047. The results indicate clearly that all the steel reinforcement had yielded at a strain of 0.0037 before failure.
The slab failed at an applied load of 25.3 kips, compared to 11.6 kips measured for the control. The failure was due to debonding of the CFRP as a result of formation of intermediate cracks (IC). This debonding mechanism is the dominant form of debonding in flexural strengthened reinforced concrete structures. These cracks formed under an applied point load and propagated through the shear span in the direction of decreasing moment and induced high interfacial shear stress that caused FRP debonding. The shear span for four point-loading conditions is defined as the distance from the point load to the nearest support. Typically, this failure does not engage the aggregate in the concrete, but rather progresses through the thin mortar-rich layer comprising the surface of the structure, as shown in Figure A-8.

Figure A-8 Debonding Failure of CFRP for Slab.4ksi.1UD.011911
The deflection at failure was 4.0 inches compared to 9.1 inches for the control specimen. The deflected shape of this specimen at failure is shown in Figure A-9. At failure, there was significant propagation of a large number of cracks in the region of maximum moment, where failure occurred as shown in Figure A-10.

Figure A-9 Deflected Shape of Slab.4ksi.1UD.011911 at Failure
Test results show that the load carrying capacity was increased by 54% when compared to the control specimen. It was also seen from these results that the deflection at failure of the specimen decreased by 56% compared to the control specimen. The decrease in deflection can be associated with debonding of the CFRP strengthening system, which lead to the premature failure of the reinforced slab. The was no change of the failure mode compared to the control.
A-3  Slab.4ksi.2UD.011911

This reinforced slab was cast using 4ksi target nominal compressive strength concrete on January 19, 2011. The slab was strengthened with two layers of uni-directional CFRP. The two layers of flexural strengthening consisted of two 12” wide sheets laid next to each other to cover the entire width of the bottom surface of the slab, as shown in Figure A-11. The measured load-deflection behavior at mid-span for the specimen is shown in Figure A-12.

Figure A-11 Preparation for Application of Second Layer of CFRP
The slab was loaded up to an equivalent service load of 5.0 kips and then unloaded. The load-deflection behavior, as shown in Figure A-12, was linear up to the initiation of first crack, which occurred at a load level of 3.05 kips compared to 2.53 kips for the control. The increase of the cracking load capacity, in comparison to the control slab, is due to the presence of the CFRP sheets which delayed the initiation of the cracking load. The behavior after the initiation of first crack was nonlinear with a reduction of the stiffness. After initial unloading, the residual deflection at midspan of 0.09 inches was measured, compared to 0.12 inches for the control specimen. Again, this behavior is due to the presence of the CFRP sheets, which are linear elastic and contribute to reducing the permanent deformation in comparison to the control slab. The slab was then loaded up to a pre-determined ultimate load of 16.1 kips and then unloaded. The load-deflection behavior relationship is nonlinear and is shown in Figure A-12. In the third loading stage, the slab was loaded until failure.
The behavior of the load-deflection graph was nonlinear as shown in Figure A-12 during this loading stage until the yielding of reinforcing steel, at an applied load of 17.1 kips compared to 9.61 kips for the control. At this point, there was a significant reduction in stiffness and the behavior followed a non-linear load-deflection behavior up to failure.

The measured flexural load-strain relationships of the concrete at the top compression surface, tension surface of the CFRP strengthening system, and at the level of longitudinal tension steel are shown in Figure A-13. The measured strain indicates that the maximum measured concrete strain in compression was 0.0051. The results also indicate clearly that all the reinforcing steel had yielded before failure of the specimen.

![Figure A-13 Strain Measurements for Slab.4ksi.2UD.011911](image-url)
The slab failed at an ultimate load of 28.2 kips, compared to 11.6 kips measured for the control. The failure was due to IC debonding of the CFRP as a result of formation and propagation of intermediate cracks within the shear span of the slab. The failure of this specimen is shown in Figure A-14.

![Figure A-14 Debonding Failure of CFRP for Slab.4ksi.2UD.011911](image)

The deflection at failure was 4.1 inches compared to 9.1 inches for the control specimen. The deflected shape of this specimen at failure is shown in Figure A-15. At failure, there
was significant propagation of a large number of cracks in the maximum moment region, as shown in Figure A-16.

Figure A-15 Deflected Shape of Slab.4ksi.2UD.011911 at Failure
The results show that the load carrying capacity was increased by 59% when compared to the control specimen. It was also seen from these results that the deflection at failure of the specimen decreased by 55% compared to the control specimen. The decrease in deflection can be associated with the debonding of the CFRP strengthening system, which lead to the premature failure of the reinforced slab. There was no change of the failure mode compared to the control.
This reinforced slab was cast using 8ksi target nominal compressive strength concrete on January 19, 2011. The slab was not strengthened with any CFRP system and it was used as a control to study the effectiveness of the various flexural strengthening systems for the slabs cast with 8ksi target compressive concrete strength on the same day. The measured load-deflection behavior at mid-span for this slab is shown in Figure A-17.

The slab was loaded up to an equivalent pre-determined service load of 6.09 kips and then unloaded. The load-deflection behavior shown in Figure A-17, was linear up to the initiation of the first crack, which occurred at a load level of 3.05 kips. The behavior after the initiation of first crack was nonlinear with a reduction of the stiffness. After initial unloading, a residual deflection at midspan of 0.244 inches was measured. The slab was then
loaded up to a pre-determined ultimate load of 7.22 kips and then unloaded. The load-deflection behavior was nonlinear as shown in Figure A-17. In the third loading stage; the slab was loaded until failure. The behavior of the load-deflection graph had a significant reduction of the stiffness during this portion of the loading due to yielding of the reinforcing steel at an applied load of 9.28 kips, as shown in Figure A-17. After yielding of the reinforcing steel, the load-deflection behavior was nonlinear up to failure, as shown in Figure A-17.

The measured flexural load-strain relationships of the concrete at the top compression surface and bottom tension surface of the concrete as well as at the level of longitudinal tension steel reinforcing for the control slab are shown in Figure A-18. The measured concrete strains represent the average strain within the gauge length of the pi gauges. Test results indicate that the maximum measured concrete strain in compression was 0.010 and that the steel reinforcement had yielded at a strain of 0.0021 before failure.
The slab failed at a load of 11.81 kips due to concrete crushing in the region of maximum moment, which is the area between the two load points. The measured deflection at failure was 10.24 inches and the deflected shape of the control prior to failure is shown in Figure A-19. At failure, there was significant propagation of a large number of flexure cracks, as shown in Figure A-20, and the failure was very ductile.
Figure A-19 Deflected Shape of 8ksi Control at Failure

Figure A-20 Crack Pattern of 8ksi Control Slab at Failure
A-5  Slab.8ksi.1UD.011911

This reinforced slab was cast using 8ksi target nominal compressive strength concrete on January 19, 2011. The slab was strengthened with one layer of uni-directional CFRP. The one layer of flexural strengthening consisted of two 12” wide sheets laid next to each other to cover the entire width of the bottom surface of the slab, as shown in Figure A-21. The measured load-deflection behavior at mid-span for this specimen is shown in Figure A-22.
The slab was loaded up to an equivalent service load of 8.4 kips and then unloaded. The load-deflection behavior shown in Figure A-22, was linear during this stage up to the initiation of the first crack, which occurred at a load level of 3.40 kips compared to 3.05 kips for the control. The increase of the cracking load capacity, in comparison to the control slab, is due to the presence of the CFRP sheet which delayed the initiation of the cracking load. The behavior after the initiation of first crack was nonlinear with a reduction of the stiffness, as shown in Figure A-22. After initial unloading, a residual deflection at midspan of 0.20 inches was measured, compared to 0.244 for the control specimen. Again, this behavior is due to the presence of the CFRP sheet, which is linear elastic and contributes to reduce the permanent deformation in comparison to the control slab. The slab was then loaded up to a pre-determined ultimate load of 15.9 kips and then unloaded. The load-deflection behavior was nonlinear, as shown in Figure A-22, until yielding of reinforcing steel, at an applied load...
of 13.92 kips compared to 9.28 kips for the control. At this point, there was a significant reduction of the stiffness and the load-deflection behavior followed a nonlinear relationship. In the third loading stage, the slab was loaded until failure. The behavior of the load-deflection graph was nonlinear as shown in Figure A-22 up to failure of the slab.

The measured flexural load-strain relationships of the concrete at the top compression surface, tension surface of the CFRP strengthening system, and at the level of longitudinal tension steel are shown in Figure A-23. The measured strain indicates that the maximum measured concrete strain in compression was 0.0036. The results indicate clearly that all the steel reinforcement had yielded at a strain of 0.0033 before failure.

![Figure A-23 Strain Measurements for Slab](Slab.8ksi.1UD.011911)

The slab failed at an applied load of 19.2 kips, compared to 11.81 kips measured for the control. The failure was due to IC debonding of the CFRP as a result of formation of
intermediate cracks within the shear span of the slab that propagated throughout the slab. The failure of this specimen is shown in Figure A-24.

The deflection at failure was 4.0 inches, compared to 10.24 inches for the control specimen. The deflected shape of this specimen at failure is shown in Figure A-25. At failure, there was significant propagation of a large number of cracks in the region of maximum moment, where failure occurred as shown in Figure A-26.
Figure A-25 Deflected Shape for Slab.8ksi.1UD.011911 at Failure

Figure A-26 Crack Pattern for Slab.8ksi.1UD.011911 at Failure
Test results show that the load carrying capacity was increased by 38% when compared to the control specimen. It was also seen from these results that the deflection at failure of the specimen decreased by 66% compared to the control specimen. The decrease in deflection can be associated with the premature debonding of the CFRP strengthening system, which lead to premature failure of the reinforced slab. There was no change of the failure mode compared to the control.
This reinforced slab specimen was cast using 8ksi target nominal compressive strength concrete on January 19, 2011. The slab was strengthened with two layers of uni-directional CFRP. The two layers of flexural strengthening consisted of two 12” wide sheets laid next to each other to cover the entire width of the bottom surface of the slab, as shown in Figure A-27. The measured load-deflection behavior at mid-span for this slab is shown in Figure A-28.
The slab was loaded up to an equivalent service load of 9.9 kips and then unloaded. The load-deflection behavior shown in Figure A-28, was linear up to the initiation of the first crack, which occurred at 3.82 kips compared to 3.05 kips for the control. The increase of the cracking load capacity, in comparison to the control slab, is due to the presence of the CFRP sheet which delayed the initiation of the cracking load. The behavior after the initiation of first crack was nonlinear with a reduction of the stiffness. After initial unloading, the residual deflection at mid-span of 0.17 inches was measured, compared to 0.244 inches for the control specimen. The slab was then loaded up to the pre-determined ultimate load of 22.2 kips and then unloaded. The load-deflection behavior was nonlinear as shown in Figure A-28 until yielding of reinforcing steel, at a load of 18.17 kips compared to 9.28 kips for the control. Again, this behavior is due to the presence of the CFRP sheet, which is linear elastic and contributes to reduce the permanent deformation in comparison to the control slab. At
this point, there was a significant reduction of the stiffness and the load-deflection behavior followed a nonlinear relationship. In the third loading stage, the slab was loaded until failure. The behavior of the load-deflection graph was nonlinear as shown in Figure A-28 up to failure of the slab, as shown in Figure A-28.

The measured flexural load-strain relationships of the concrete at the top compression surface, tension surface of the CFRP strengthening system, and at the level of longitudinal tension steel are shown in Figure A-29. The measured strain indicates that the maximum measured concrete strain in compression was 0.0037. The results indicate clearly that all the steel reinforcement had yielded at a strain of 0.0032 before failure.

![Figure A-29 Strain Measurements for Slab.8ksi.2UD.011911](image)
The slab failed at an applied load of 28.0 kips, compared to 11.81 kips measured for the control. The failure was due to IC debonding of the CFRP as a result of formation of intermediate cracks under a load point that propagated within the shear span of the slab. The failure of this specimen is shown in Figure A-30.

![Figure A-30 Debonding Failure of CFRP for Slab.8ksi.2UD.011911](image)

The deflection at failure was 3.2 inches, compared to 10.24 inches for the control specimen. The deflected shape of this specimen at failure is shown in Figure A-31. At failure, there was significant propagation of a large number of cracks within the maximum moment region, where failure occurred as shown in Figure A-32.
Figure A-31 Deflected Shape for Slab.8ksi.2UD.011911 at Failure

Figure A-32 Crack Pattern for Slab.8ksi.2UD.011911 at Failure
Test results show that the load carrying capacity was increased by 58% when compared to the control specimen. It was also seen from these results that the deflection at failure of the specimen decreased by 69% compared to the control specimen. The decrease in deflection can be associated with the premature debonding of the CFRP strengthening system, which lead to the premature failure of the reinforced slab. There is no change of the failure mode compared to the control.
A-7  Slab.4ksi.1UD.051211

Two additional slabs were cast to determine the effects of a uni-directional CFRP strengthening system compared to a bi-directional CFRP strengthening system. These reinforced slab specimens were cast using 4ksi target nominal compressive strength concrete on May 12, 2011. The slab discussed in this section was strengthened with one layer of uni-directional CFRP. The one layer of flexural strengthening consisted of two 12” wide sheets laid next to each other to cover the entire width of the bottom surface of the slab, as shown in Figure A-33. The measured load-deflection behavior at mid-span for this slab is shown in Figure A-34.

![Figure A-33 Application of One Sheet of Uni-directional CFRP](image-url)
The slab was loaded up to an equivalent service load of 4.5 kips and then unloaded. The load-deflection behavior shown in Figure A-34, was linear up to initiation of the first crack, which occurred at a load level of 2.83 kips. The behavior after the initiation of first crack was nonlinear with a reduction of the stiffness. After initial unloading, the residual deflection at mid-span of 0.09 inches was measured. The slab was then loaded up to a pre-determined ultimate load of 14.0 kips and then unloaded. The load-deflection behavior was nonlinear as shown in Figure A-34 during ultimate loading until the yielding of reinforcing steel, at an applied load of 13.66 kips. At this point, there was a significant reduction of the stiffness and the load-deflection behavior followed a nonlinear relationship. In the third loading stage, the slab was loaded until failure. The load-deflection behavior was nonlinear as shown in Figure A-34 up to failure of the slab.
The measured flexural load-strain relationships of the concrete at the top surface, tension surface of the CFRP strengthening system, and at the level of longitudinal tension steel are shown in Figure A-35. The measured strain indicates that the maximum measured concrete strain in compression was 0.0057. The results indicate clearly that all the steel reinforcement had yielded at a strain of 0.0029 before failure.

The slab failed at an applied load of 22.81 kips due to IC debonding. The CFRP debonding was a result of formation of intermediate cracks within the shear span of the slab. The failure of this specimen is shown in Figure A-36.
The deflection at failure was 4.8 inches and the deflected shape of this specimen at failure is shown in Figure A-37. At failure, there was significant propagation of a large number of cracks within the maximum moment region, where failure occurred as shown in Figure A-38.
Figure A-37 Deflected Shape for Slab.4ksi.1UD.051211 at Failure

Figure A-38 Crack Pattern for Slab.4ksi.1UD.051211 at Failure
This reinforced slab specimen was cast using 4ksi target nominal compressive strength concrete on May 12, 2011. This slab was strengthened with one layer of bi-directional CFRP. This layer of flexural strengthening consisted of two 12” wide sheets laid next to each other to cover the entire width of the bottom surface of the slab, as shown in Figure A-39. The measured load-deflection behavior at mid-span for this specimen is shown in Figure A-40.
The slab was loaded up to an equivalent service load of 4.5 kips and then unloaded. The load-deflection behavior shown in Figure A-40, was linear up to initiation of the first crack, which occurred at a load level of 3.0 kips compared to 2.83 kips measured for the slab strengthened with the uni-directional fiber. These results indicate that the load-deflection behavior of the slabs strengthened with uni-directional and bi-directional fibers are virtually identical. The behavior after the initiation of first crack was nonlinear with a reduction of the stiffness, as shown in Figure A-40. After initial unloading, the residual deflection at mid-span of 0.12 inches was measured, compared to 0.09 inches for the slab strengthened with the uni-directional fiber. Again, this behavior indicates that the load-deflection behavior of the slabs strengthened with the uni-directional or bi-directional fibers are virtually identical. The slab was then loaded up to a pre-determined ultimate load of 14.0 kips and then unloaded. The load-deflection behavior was nonlinear as shown in Figure A-40. In the third loading
stage, the slab was loaded until failure. The load-deflection behavior was nonlinear as shown in Figure A-40 during ultimate loading until yielding of the reinforcing steel, at an applied load of 13.92 kips compared to 13.66 kips for the slab strengthened with the uni-directional fiber. At this point there was a significant reduction in the stiffness and the load-deflection behavior followed a nonlinear relationship up to failure of the slab.

The measured flexural load-strain relationships of the concrete at the top compression surface, tension surface of the CFRP strengthening system, and at the level of longitudinal tension steel are shown in Figure A-41. The measured strain indicates that the maximum measured concrete strain in compression was 0.0055. The results indicate clearly that all the steel reinforcement had yielded at a strain of 0.0026 before failure.
The slab failed at an applied load of 22.03 kips, compared to 22.81 kips measured for the slab strengthened with uni-directional fibers. These results again indicate that there is virtually no difference in behavior between the slabs strengthened with uni-directional or bidirectional fibers. The failure was due to IC debonding as a result of formation and propagation of intermediate cracks within the shear span of the slab. The failure of this specimen is shown in Figure A-42.

![Figure A-42 Debonding Failure of CFRP for Slab](image)

The deflection at failure was 4.9 inches compared to 4.8 inches for the slab strengthened with uni-directional fibers. The deflected shape of this specimen at failure is shown in Figure A-43. At failure, there was significant propagation of a large number of cracks within the region of maximum moment, where failure occurred as shown in Figure A-44.
Test results for the uni-directional and bi-directional CFRP reinforced slabs cast on 05/12/11 show that the failure loads for both of these specimens were identical and the deflection at failure was virtually the same as well. The results also show almost identical load-deflection
behavior, therefore it can be concluded that the presence of the transverse fiber is ineffective. The transverse fibers are ineffective because they do not aid in increasing the longitudinal reinforcement ratio of the specimen, which leads to no change in the load carrying capacity of the slab.
This reinforced flexure T-beam was cast using 4ksi target nominal compressive strength concrete on January 19, 2011. The T-beam was not strengthened with any CFRP system and it was used as a control beam to study the effectiveness of the various flexural strengthening systems for the beams cast with 4ksi target compressive concrete strength on the same day. The measured load-deflection behavior at mid-span for this specimen is shown in Figure A-45.

The T-beam was loaded up to an equivalent pre-determined service load of 9.4 kips and then unloaded. The load-deflection behavior shown in Figure A-45, was linear up to the initiation of the first crack, which occurred at a load level of 5.81 kips. The behavior after the initiation of first crack was nonlinear with a reduction of the stiffness. After initial
unloading, the residual deflection at mid-span of 0.01 inches was measured. The T-beam was then loaded up to a pre-determined ultimate load of 14.6 kips and then unloaded. The load-deflection behavior was nonlinear as shown in Figure A-45. At this load level, flexure cracks had propagated from the web into the flange of the beam. In the third loading stage; the T-beam was loaded until failure. The behavior of the load-deflection graph had a significant reduction in the stiffness during this portion of the loading due to the yielding of reinforcing steel at an applied load of 16.6 kips, as shown in Figure A-45. After yielding of the reinforcing steel, the load-deflection behavior was nonlinear up to failure as shown in Figure A-45.

The measured flexural load-strain relationships of the concrete at the top compression surface and bottom tension surface of the concrete as well at level of longitudinal tension steel for the control T-beam are shown in Figure A-46. The measured strains represent the average strain within the gauge length of the pi gauges. The measured strain indicates that the maximum measured concrete strain in compression was 0.0073. The results indicate clearly that all steel reinforcement had yielded at a strain of 0.0034 before failure.
The T-beam failed at a load of 26.5 kips due to concrete crushing in the region of maximum moment, which is the area between the two load points. The measured deflection at failure was 3.9 inches and the deflected shape of the control specimen prior to failure can is shown in Figure A-47. At failure, there was significant propagation of a large number of cracks in the web and flange, as shown in Figure A-48, and the failure was very ductile. Some of the flexure cracks in the web had significant widths within the maximum moment area at failure, ranging from 10-15 mm.
Figure A-47 Deflected Shape at Failure for 4ksi Control FlexBeam

Figure A-48 Crack Pattern at Failure for 4ksi Control FlexBeam
This reinforced flexure T-beam was cast using 4ksi target nominal compressive strength concrete on January 19, 2011. The T-beam was strengthened with one layer of uni-directional CFRP. The one layer of flexural strengthening consisted of a 12” wide CFRP sheet placed on the bottom surface of the web, as shown in Figure A-49. The measured load-deflection behavior at mid-span for this specimen is shown in Figure A-50.
The T-beam was loaded up to an equivalent service load of 13.1 kips and then unloaded. The load-deflection behavior shown in Figure A-50 was linear during this stage of loading. After initial unloading, the residual deflection at mid-span of 0.03 inches was measured, compared to 0.01 inches for the control specimen. The T-beam was then loaded up to the predetermined ultimate load of 28.6 kips and then unloaded. From the load-deflection behavior shown in Figure A-50 first cracking occurred at an applied load of 15.5 kips, compared to 5.81 kips for the control flexure beam. The increase of the cracking load capacity, in comparison to the control slab, is due to the presence of the CFRP sheet which delayed the initiation of the cracking load. The behavior after the initiation of first crack was nonlinear with a reduction of the stiffness, as shown in Figure A-50. The behavior of the load-deflection graph was nonlinear until yielding of the reinforcing steel, at an applied load of 25.1 kips compared to 16.6 kips for the control. At this point there was a large reduction in
the stiffness and the load-deflection behavior followed a nonlinear relationship up to unloading. In the third loading stage, the T-beam was loaded until failure. The load-deflection behavior shown in Figure A-50 was nonlinear up to failure of the beam.

The measured flexural load-strain relationships of the concrete at the top compression surface, tension surface of the CFRP strengthening system, and at the level of longitudinal tension steel are shown in Figure A-51. The measured strain indicates that the maximum measured concrete strain in compression was 0.0019. The results indicate clearly that all the steel reinforcement had yielded at a strain of 0.0036 before failure.

![Figure A-51 Strain Measurements for FlexBeam.4ksi.1UD.011911](image)

The T-beam failed at an applied load of 41.8 kips, compared to 26.5 kips measured for the control. The failure was due to debonding of the CFRP as a result of formation and
propagation of intermediate cracks (IC) within the shear span of the beam. The failure of this specimen is shown in Figure A-52.

![Figure A-52 Debonding Failure of CFRP for FlexBeam.4ksi.1UD.011911](image)

The deflection at failure was 1.2 inches compared to 3.9 inches for the control. This behavior can be attributed to the CFRP strengthening system being linear elastic and reducing the permanent deformation in comparison to the control specimen. The deflected shape of this specimen at failure is shown in Figure A-53. At failure, there was significant propagation of a large number of cracks in the region of maximum moment, where failure occurred as shown in Figure A-54. There were no significant crack widths present at failure, as was seen in the control specimen. However, there was formation of a number of flexure-shear cracks, as shown in Figure A-54, which could have induced the intermediate crack debonding.
Test results show that the load carrying capacity was increased by 37% when compared to the control specimen. It was also seen from these results that the deflection at failure of the specimen decreased by 69% compared to the control specimen. The decrease in deflection can be associated with the premature debonding of the CFRP strengthening system, which
lead to the premature failure of the reinforced T-beam. There was no change in the failure mode compared to the control.
A-11  FlexureBeam.4ksi.2UD.011911

This reinforced flexure T-beam was cast using 4ksi target compressive strength concrete on January 19, 2011. The T-beam was strengthened with two layers of uni-directional CFRP. The two layers of flexural strengthening consisted of two 12” wide sheets placed on the bottom surface of the web, as shown in Figure A-55. The measured load-deflection behavior at mid-span for this specimen is shown in Figure A-56.

![Figure A-55 Application of Second Layer of CFRP to FlexBeam](image)
The flexure T-beam was loaded up to an equivalent service load of 16.8 kips and then unloaded. The load-deflection behavior as shown in Figure A-56, was linear until the initiation of the first crack at 14.3 kips, compared to 5.81 kips for the control slab. The increase of the cracking load capacity, in comparison to the control, is due to the presence of the CFRP sheets which delayed the initiation of the cracking load. The behavior after the initiation of first crack was nonlinear with a reduction of the stiffness, as shown in Figure A-56. After initial unloading, the residual deflection at midspan of 0.05 inches was measured, compared to 0.01 inches for the control specimen. The T-beam was loaded up to a predetermined ultimate load of 36.1 kips and then unloaded. The load-deflection behavior was nonlinear up to yielding of the reinforcing steel, at an applied load of 29.4 kips compared to 16.6 kips for the control. At this point there was a significant reduction in the stiffness and the load-deflection behavior followed a nonlinear relationship. In the third loading stage, the
T-beam was loaded until failure. The load-deflection behavior was nonlinear, as shown in Figure A-56, up to failure of the beam.

The measured flexural load-strain relationships of the concrete at the top compression surface, tension surface of the CFRP strengthening system, and at the level of longitudinal tension steel are shown in Figure A-57. The measured strain indicates that the maximum measured concrete strain in compression was 0.0016. The results indicate clearly that all the steel reinforcement had yielded at a strain of 0.003 before failure.

The T-beam failed at an applied load of 44.2 kips, compared to 26.5 kips measured for the control. The failure was due to debonding of the CFRP as a result of formation and
propagation of intermediate cracks (IC) within the shear span of the beam. The failure of this specimen is shown in Figure A-58.

The deflection at failure was 0.95 inches compared to 3.9 inches for the control specimen. This behavior can be attributed to the CFRP strengthening system being linear elastic and reducing the permanent deformation in comparison to the control specimen. The deflected shape of this specimen at failure is shown in Figure A-59. At failure, there was significant propagation of a large number of cracks in the region of maximum moment, where failure occurred as shown in Figure A-60. There were no significant crack widths present at failure, as was seen in the control specimen. However, there was formation of a number of flexure-shear cracks, which could have induced the intermediate crack debonding, as shown in Figure A-60.
Test results show that the load carrying capacity was increased by 40% when compared to the control specimen. It was also seen from these results that the deflection at failure of the specimen decreased by 76% compared to the control specimen. The decrease in deflection can be associated with the premature debonding of the CFRP strengthening system, which
lead to the premature failure of the reinforced T-beam. There was no change in the failure mode compared to the control.
This reinforced T-beam was cast using 8ksi target nominal concrete strength concrete on January 19, 2011. The T-beam was not strengthened with any CFRP system and it was used as a control beam to study the effectiveness of the various flexure strengthening systems for the beams cast with 8ksi target compressive concrete strength on the same day. The measured load-deflection behavior at mid-span for this specimen is shown in Figure A-61.

![Figure A-61 Load-Deflection Behavior for 8ksi Control FlexBeam](image)

The T-beam was loaded up to an equivalent pre-determined service load of 9.5 kips and then unloaded. The load-deflection behavior shown in Figure A-61 was linear during this stage of loading. After initial unloading, the residual deflection at midspan was 0.02 inches. The T-beam was then loaded up to a pre-determined ultimate load of 15.9 kips and then unloaded. The load-deflection behavior shown in Figure A-61, was linear up to the initiation of the first
crack, which occurred at a load level of 12.0 kips. The behavior after the initiation of first
crack was nonlinear with a reduction of the stiffness. At this load step, flexure cracks had
propagated from the web into the flange. In the third loading stage, the T-beam was loaded
until failure. The behavior of the load-deflection graph had a significant reduction in the
stiffness during this portion of the loading due to yielding of the reinforcing steel at an
applied load of 17.5 kips, as shown in Figure A-61. The load-deflection behavior was
nonlinear after yielding of the reinforcing still until failure of the beam.

The measured flexure load-strain relationships of the concrete at the top compression surface
and bottom tension surface of the concrete as well as at the level of longitudinal tension steel
for the control T-beam are shown in Figure A-62. The measured strain indicates that the
maximum measured concrete strain in compression was 0.0072. The results clearly indicate
all the steel reinforcement had yielded at a strain of 0.0035 before failure.
The T-beam failed at an applied load of 28.6 kips due to concrete crushing in the region of maximum moment, which is the area between the two load points. The measured deflection at failure was 5.3 inches and the deflected shape of the control specimen prior to failure is shown in Figure A-63. At failure, there was significant propagation of a large number of cracks in the web and flange, as shown in Figure A-64. Some of the flexure cracks in the web had significant widths within the maximum moment area at failure, ranging from 10-15 mm, and the failure was very ductile.
Figure A-63 Deflected Shape of 8ksi Control FlexBeam at Failure

Figure A-64 Crack Pattern of 8ksi Control FlexBeam at Failure
A-13 FlexureBeam.8ksi.1UD.011911
This reinforced flexure T-beam was cast using 8ksi target nominal compressive strength concrete on January 19, 2011. The T-beam was strengthened with one layer of uni-directional CFRP. The one layer of flexural strengthening consisted of a 12” wide sheet placed on the bottom of the web, as shown in Figure A-65. The measured load-deflection behavior at mid-span for this specimen is shown in Figure A-66.

Figure A-65 Application of One Layer of CFRP to 8ksi FlexBeam
The T-beam was loaded up to an equivalent service load of 13.3 kips and then unloaded. From the load-deflection behavior in Figure A-66, first cracking occurred at 12.7 kips, compared to 12.0 kips for the control flexure beam. The increase of the cracking load capacity, in comparison to the control T-beam, is due to the presence of the CFRP sheet which delayed the initiation of the cracking load. The behavior after the initiation of first crack was nonlinear with a reduction of the stiffness, as shown in Figure A-66. After initial unloading, the residual deflection at mid-span of 0.02 inches was measured, compared to 0.02 inches for the control. The T-beam was then loaded up to a pre-determined ultimate load of 28.7 kips and then unloaded. The load-deflection behavior as shown in Figure A-66 was linear until the yielding of the reinforcing steel, at an applied load of 23.4 kips compared to 17.5 kips measured for the control. At this point there was a significant reduction in the stiffness and the load-deflection behavior followed a nonlinear relationship. In the third
loading stage, the T-beam was loaded until failure. The load-deflection behavior, as shown in Figure A-66, was nonlinear up to failure of the beam.

The measured load-strain relationships of the concrete at the top compression surface, tension surface of the CFRP strengthening system, and at the level of longitudinal tension steel are shown in Figure A-67. These findings indicate that the maximum measured concrete strain in compression was 0.0019. The results clearly indicate that all steel reinforcement had yielded at a strain of 0.0017 before failure.

The T-beam failed at an applied load of 38.5 kips, compared to 28.6 measured kips for the control. The failure was due to debonding of the CFRP as a result of formation and
propagation of intermediate cracks (IC) within the shear span of the beam. The failure of this specimen is shown in Figure A-68.

The deflection at failure was 1.2 inches, compared to 5.3 inches for the control. This behavior can be attributed to the CFRP strengthening system being linear elastic and reducing the permanent deformation in comparison to the control specimen. The deflected shape of this specimen at failure is shown in Figure A-69. At failure, there was significant propagation of a large number of cracks in the region of maximum moment, where failure occurred as shown in Figure A-70. There were no significant crack widths present at failure, as was seen in the control specimen. However, there was formation of a number of flexure-shear cracks, which could have induced the intermediate crack debonding, as shown in Figure A-70.
Figure A-69 Deflected Shape of FlexBeam.8ksi.1UD.011911 at Failure

Figure A-70 Crack Pattern of FlexBeam.8ksi.1UD.011911 at Failure
Test results show that the load carrying capacity was increased by 26% when compared to the control specimen. It was also seen from these results that the deflection at failure of the specimen decreased by 78% compared to the control specimen. The decrease in deflection can be associated with the premature debonding of the CFRP strengthening system, which lead to the premature failure of the reinforced T-beam. There is no change in the failure mode compared to the control.
A-14  FlexureBeam.8ksi.2UD.011911

This reinforced flexure T-beam was cast using 8ksi target nominal compressive strength concrete on January 19, 2011. The T-beam was strengthened with two layers of uni-directional CFRP. The two layers of flexural strengthening consisted of two 12” wide sheets placed on the bottom surface of the web, as shown in Figure A-71. The measured load-deflection behavior at mid-span for this specimen is shown in Figure A-72.

Figure A-71 Application of Second Layer of CFRP to 8ksi FlexBeam
The T-beam was loaded up to an equivalent service load of 17.0 kips and then unloaded. The load-deflection behavior shown in Figure A-72 was linear until the initiation of the first crack at 13.3 kips, compared to 12.0 kips for the control specimen. The increase of the cracking load capacity, in comparison to the control, is due to the presence of the CFRP sheets which delayed the initiation of the cracking load. The behavior after the initiation of first crack was nonlinear with a reduction of the stiffness, as shown in Figure A-72. After initial unloading, the residual deflection at mid-span was 0.04 inches, compared to 0.02 inches for the control specimen. The T-beam was then loaded up to a pre-determined ultimate load of 43.9 kips and then unloaded. The load-deflection behavior relationship shown in Figure A-72, was nonlinear up to yielding of the reinforcing steel, at an applied load of 29.7 kips compared to 17.5 kips for the control. At this point there was a significant reduction in the stiffness and the load-deflection behavior followed a nonlinear relationship up to unloading. In the third
loading stage, the T-beam was loaded until failure. The load-deflection behavior relationship shown in Figure A-72 is non-linear up to failure of the beam.

The measured flexural load-strain relationships of the concrete at the top compression surface, tension surface of the CFRP strengthening system, and at the level of longitudinal tension steel are shown in Figure A-73. The measured strain indicates that the maximum measured concrete strain in compression was 0.0017. The results indicate clearly that all the steel reinforcement had yielded at a strain of 0.0041 before failure.

![Figure A-73 Strain Measurements for Slab.8ksi.2UD.011911](image)

The T-beam failed at an applied load of 53.2 kips, compared to 28.6 kips measured for the control. This was due to debonding of the CFRP as a result of formation and propagation of
intermediate cracks (IC) within the shear span of the beam. The failure of this specimen is shown in Figure A-74.

Figure A-74 Debonding Failure of CFRP for FlexBeam.8ksi.2UD.011911

The deflection at failure was 1.1 inches, compared to 5.3 inches for the control. This behavior can be attributed to the CFRP strengthening system being linear elastic and reducing the permanent deformation in comparison to the control specimen. The deflected shape of this specimen at failure is shown in Figure A-75. At failure, there was significant propagation of a large number of cracks in the region of maximum moment, where failure occurred as shown in Figure A-76. There were no significant crack widths present at failure, as was seen in the control specimen. However, there was formation of a number of flexure-shear cracks, which could have induced the intermediate crack debonding, as shown in Figure A-76.
Figure A-75 Deflected Shape of FlexBeam.8ksi.2UD.011911 at Failure

Figure A-76 Crack Pattern of FlexBeam.8ksi.2UD.011911 at Failure
Test results show that the load carrying capacity was increased by 46% when compared to the control specimen. It was also seen from these results that the deflection at failure of the specimen decreased by 79% compared to the control specimen. The decrease in deflection can be associated with the premature debonding the CFRP strengthening system, which lead to the premature failure of the reinforced T-beam. There is no change in the mode of failure compared to the control.
Two additional T-beams were cast to determine the effects of a uni-directional CFRP strengthening system compared to a bi-directional CFRP strengthening system. These reinforced T-beams specimens were cast using 4ksi target nominal compressive strength concrete on May 12, 2011. The T-beam discussed in this section was strengthened with one layer of uni-directional CFRP. This one layer of flexural strengthening consisted of a 12” wide sheet placed on the bottom surface of the web, as shown in Figure A-77. The measured load-deflection behavior at mid-span for this specimen is shown in Figure A-78.
The T-beam was loaded up to an equivalent service load of 13.1 kips and then unloaded. The load-deflection behavior shown in Figure A-78 was linear during this stage of loading. After initial unloading, a residual deflection at mid-span of 0.01 inches was measured. The T-beam was then loaded to the ultimate load of 28.6 kips and then unloaded. The load-deflection behavior relationship shown in Figure A-78 was linear until the initiation of the first crack at 16.8 kips. The behavior after the initiation of first crack was nonlinear with a reduction of the stiffness, as shown in Figure A-78. The behavior of the load-deflection graph was nonlinear until yielding of the reinforcing steel, at an applied load of 24.3 kips. At this point there was a significant reduction in stiffness and the behavior followed a nonlinear load-deflection relationship. In the third loading stage, the T-beam was loaded until failure. The load-deflection behavior was nonlinear as shown in Figure A-78 until failure of the beam.
The measured flexural load-strain relationships of the concrete at the top compression surface, tension surface of the CFRP strengthening system, and at the level of longitudinal tension steel are shown in Figure A-79. These findings indicate that the maximum measured concrete strain in compression was 0.0025. The results clearly indicate that all the steel reinforcement had yielded at a strain of 0.0027 before failure.

The T-beam failed at an applied load of 46.1 kips. The failure was due to debonding of the CFRP as a result of formation and propagation of intermediate cracks (IC) within the shear span of the beam. The failure of this specimen is shown in Figure A-80.
The deflection at failure was 1.4 inches and the deflected shape of this specimen at failure is shown in Figure A-53. At failure, there was significant propagation of a large number of flexure and flexure-shear cracks in the region of maximum moment, where failure occurred as shown in Figure A-54.
Figure A-81 Deflected Shape of FlexBeam.4ksi.1UD.051211 at Failure

Figure A-82 Crack Pattern of FlexBeam.4ksi.1UD.051211 at Failure
A-16  **FlexureBeam.4ksi.1BD.051211**

The other T-beam cast using 4ksi target nominal compressive strength concrete on May 12, 2011 for evaluation of the different strengthening systems was strengthened with one layer of bi-directional CFRP. This one layer of flexural strengthening consisted of a 12” wide bi-directional sheet placed on the bottom surface of the web, as shown in Figure A-83. The measured load-deflection behavior at mid-span for this specimen is shown in Figure A-84.

![Figure A-83 Application of One Layer of Bi-directional CFRP on FlexBeam](image-url)
The T-beam was loaded up to an equivalent service load of 13.1 kips and then unloaded. The load-deflection behavior shown in Figure A-84 was linear during this stage of loading. After initial unloading, the residual deflection at mid-span of 0.02 inches was measured compared to 0.01 inches for the T-beam strengthened with the uni-directional fiber. These results indicate there is virtually no difference in behavior when comparing uni-directional to the bi-directional fiber. The T-beam was then loaded to a pre-determined ultimate load of 28.6 kips and then unloaded. The load-deflection behavior shown in Figure A-84 was linear until the initiation of the first crack at 18.2 kips, compared to 16.8 kips for the T-beam strengthened with the uni-directional fibers. Again, this behavior indicates that there is virtually no difference in behavior when comparing uni-directional to the bi-directional fiber. The behavior after the initiation of first crack was nonlinear with a reduction of the stiffness, as shown in Figure A-84. The behavior of the load-deflection graph followed this nonlinear
relationship until yielding of the reinforcing steel, at an applied load of 25.7 kips compared to 24.3 kips for the T-beam strengthened with uni-directional fiber. At this point there was a significant reduction in the stiffness and the behavior followed a nonlinear load-deflection behavior. In the third loading stage, the T-beam was loaded until failure. The load-deflection behavior was nonlinear as shown in Figure A-84 until failure of the beam.

The measured flexural load-strain relationships of the concrete at the top compression surface, tension surface of the CFRP strengthening system, and at the level of longitudinal tension steel are shown in Figure A-79. The measure strain indicates that the maximum measured concrete strain in compression was 0.0024. The results clearly indicate that all the steel reinforcement had yielded at a strain of 0.0029 before failure.

![Figure A-85 Strain Measurements for FlexBeam.4ksi.1BD.051211](image)

The T-beam failed at an applied load of 46.1 kips, compared to 46.1 kips for the T-beam strengthened with the uni-directional fiber. The failure was due to debonding of the CFRP as
a result of intermediate crack (IC) debonding of the CFRP within the shear span of the beam. The failure of this specimen is shown in Figure A-86.

![Figure A-86 Debonding Failure of CFRP for FlexBeam.4ksi.1BD.051211](image)

The deflection at failure was 1.5 inches compared to 1.4 inches for the T-beam strengthened with uni-directional fibers. The deflected shape of this specimen at failure is shown in Figure A-87. At failure, there was significant propagation of a large number of flexure and flexure-shear cracks in the region of maximum moment, where failure occurred as shown in Figure A-88.
Figure A-87 Deflected Shape of FlexBeam.4ksi.1BD.051211 at Failure

Figure A-88 Crack Pattern of FlexBeam.4ksi.1BD.051211 at Failure
Test results show that the behavior of the T-beams strengthened with either the uni-directional or bi-directional was virtually identical. The failure loads for both of these specimens were identical and the deflection at failure was virtually the same as well. The transvers fibers are ineffective because they do not aid in increasing the longitudinal reinforcement ratio of the specimen, which leads to no change in the load carrying capacity of the T-beam. Therefore, it can be concluded therefore that there is no difference in behavior when strengthening with uni-directional or bi-directional fibers.
This reinforced T-beam was cast using 4ksi target nominal compressive strength concrete on January 19, 2011. The T-beam was not strengthened with any CFRP system; and it was used as the control T-beam to study the effectiveness of the various shear strengthening systems for the beams cast with 4ksi target compressive concrete strength, in the same day. The measured load-deflection at mid-span for this T-beam is shown in Figure A-89.

Figure A-89 Load-Deflection Behavior for 4ksi Control Shear Beam

The T-beam was loaded up to a pre-determined ultimate load of 60.8 kips and then unloaded. The load-deflection behavior, shown in Figure A-89, is linear up to the initiation of the first flexural crack, which occurred at an applied load level of 25.9 kips. The behavior after the initiation of first flexural crack was nonlinear with a reduction of the stiffness. In second loading stage, the T-beam was loaded until failure. The load-deflection behavior was
nonlinear up to failure as shown in Figure A-89. The behavior of the load-deflection graph had a reduction in the stiffness during failure loading due to the yielding of the transverse reinforcing steel stirrups at an applied load of 80.9 kips, as shown in Figure A-89. The measured flexure load-strain relationships of the concrete at the top compression surface and bottom tension surface of the concrete as well as at the level of longitudinal tension steel for the control T-beam are shown in Figure A-90. These findings indicate that the maximum measured concrete strain in compression was 0.0013. As indicated in Figure A-90, not all the longitudinal steel reinforcement had yielded before failure, as evident in the linear behavior of the load-strain relationship. This figure also indicates that the yielding strain had been reached prior to failure, suggesting that the extreme fiber of the first layer of steel may have begun to yield prior to failure of the beam.

![Figure A-90 Measured Flexure Strains for 4ksi Control Shear Beam](image)

The measured axial and transverse load-strain relationships for both ends of the T-beam are shown in Figure A-91 and Figure A-92. These findings are separated according to their
placement on the shear beam, either to the right or left of the load points, as shown in Figure 3-23. These figures also indicate whether the strain instrumentation was put on the front or back side of the beam, as per Figure 3-23. These results indicate that shear cracks formed at all four locations of shear strain instrumentation. The results also clearly indicate that the transverse steel reinforcement near the supports had yielded at a load of 80.9 kips for the left side of the beam and 93.5 kips for the right side of the beam, as shown in Figure A-91 and Figure A-92, respectively.

Figure A-91 Measured Left-End Axial and Transverse Strains for 4ksi Control Shear Beam
The T-beam failed at a load of 133.9 kips due to the propagation of a critical shear crack from a load point to the support, as shown in Figure A-93. At failure, there were few flexure cracks present and the failure was very brittle.
The deflection at failure of 0.52 inches was measured and the overall view of the control specimen at failure is shown in Figure A-94. At failure, there was propagation of a few well-distributed flexure cracks in the web, as shown in Figure A-95.
Figure A-94 Failure of 4ksi Control Shear Beam

Figure A-95 Distribution of Flexure Cracks of 4ksi Control Shear Beam at Failure
A-18  ShearBeam.4ksi.WrapsUD.011911

This reinforced T-beam was cast using 4ksi target nominal compressive strength concrete on January 19, 2011. The T-beam was strengthened with uni-directional CFRP U-Wraps. The strengthening system consisted of seven 12” wide U-wrap sheets spaced 15” center to center wrapped around the bottom surface of the web up to the bottom of the flange, as shown in Figure A-96. The measured load-deflection at mid-span for this T-beam is shown in Figure A-97.
The T-beam was loaded up to a pre-determined ultimate load of 74.4 kips and then unloaded. The load-deflection behavior shown in Figure A-97, was linear up to the initiation of the first flexural crack, which occurred at an applied load of 21.5 kips, compared to 25.9 kips for the control specimen. The behavior after the initiation of first flexure crack was nonlinear with a reduction of the stiffness. In the second loading stage, the T-beam was loaded until failure. The load-deflection behavior was nonlinear as shown in Figure A-97 until yielding of the transverse reinforcing steel at an applied load of 91.9 kips, compared to 80.9 kips for the control specimen, as evident from the measured strain of the transverse steel in Figure A-99. The increase of the yield load, in comparison to the control T-beam, is due to the presence of the CFRP U-Wraps, which delayed the initiation of the yielding load. At this point, there was a reduction of the stiffness and the load-deflection behavior followed a nonlinear relationship up to failure of the specimen. The measured flexure load-strain relationships of
The measured strain indicates that the maximum measured concrete strain in compression was 0.0019. The results clearly show that not all the longitudinal steel reinforcement had yielded before failure, as evident in the linear behavior of the load-strain relationship. This figure also indicates that the yielding strain had been reached prior to failure, suggesting that the extreme fiber of the first layer of steel may have begun to yield prior to failure of the beam.

The measured axial and transverse load-strain relationships for the CFRP U-wraps next to the supports are shown in Figure A-99 and Figure A-100. These findings are separated according to their placement on the shear beam, either to the right or left of the load points,
as shown in Figure 3-23. These figures also indicate whether the strain instrumentation was put on the front side or the back side of the beam, as shown in Figure 3-23. These results indicate that shear cracks formed at all four locations of shear strain instrumentation. The results also show that the critical shear crack formed on the right side of the T-beam, indicated by the higher axial strain readings on the right side of the T-beam. The results also indicate that the transverse reinforcing steel was yielded near the supports prior to failure as shown by the large tension strains measured.

![Graph showing measured left axial and transverse strains for shear beam.](Figure A-99 Measured Left Axial and Transverse Strains for ShearBeam.4ksi.WrapsUD.011911)
The T-beam failed at an applied load of 181.5 kips, compared to 133.9 kips for the control shear beam. The failure was due to debonding of the CFRP U-Wrap as a result of the propagation of a critical shear crack from a load point to the support, as shown in Figure A-101.
The deflection failure was 0.71 inches, compared to 0.52 inches for the control specimen. The overall view of the beam at failure is shown in Figure A-102.
Test results show that the load carrying capacity was increased by 26% when compared to the control beam. It was also seen from these results that the deflection at failure of the specimen increased by 38% compared to the control beam. The increase in deflection can be associated with the higher loads achieved for this specimen compared to the control. The higher loads lead to an increase in the flexural deflection, which is the predominant form of deflection in the specimens. There is also no change of the failure mode compared to the control beam.
ShearBeam.4ksi.AnchoredWrapsUD.011911

This reinforced T-beam was cast with 4ksi target compressive strength concrete on January 19, 2011. The T-beam was strengthened with Anchored uni-directional CFRP U-Wraps. This strengthening system consisted of 3 CFRP anchors placed into drilled holes at the top of the web at the location of each wrap, as shown in Figure A-103. Uni-directional CFRP U-wraps were then placed over these anchors spaced 15” center to center wrapped around the bottom surface of the web up to the bottom of the flange, as shown in Figure A-104. The measured load-deflection at mid-span for this T-beam is shown in Figure A-105.

Figure A-103 CFRP Anchors for ShearBeam.4ksi.AnchoredWrapsUD.011911
Figure A-104 Application of CFRP U-Wraps with Shear Anchors
The T-beam was loaded up to a pre-determined ultimate load of 81.7 kips and then unloaded. The load-deflection behavior shown in Figure A-105, was linear up to the initiation of the first flexure crack, which occurred at an applied load level of 23.2 kips, compared to 25.9 kips for the control specimen. The behavior after the initiation of first crack was nonlinear with a reduction of the stiffness. In the second loading stage, the T-beam was loaded until failure. The load-deflection behavior was nonlinear as shown in Figure A-105 until yielding of the transverse reinforcing steel. The behavior of the load-deflection graph had a reduction in the stiffness at yielding of the transverse reinforcing steel at an applied load of 90.2 kips, compared to 80.9 kips for the control specimen. The increase of the yielding capacity in comparison to the control T-beam is due to the presence of the CFRP U-Wraps which delayed the initiation of the yielding load. At this point, there was a reduction of the stiffness and the load-deflection behavior followed a nonlinear relationship up to yielding of the
longitudinal reinforcement. At this point, there was a substantial decrease in the stiffness of the beam, as shown in Figure A-105, and the load-deflection behavior followed a nonlinear behavior until failure of the specimen due to the formation of a critical shear crack. The measured flexure load-strain relationships of the top compressive surface of concrete, bottom tension surface of the CFRP U-wraps strengthening system, and at the level of longitudinal tension steel are shown in Figure A-106. These findings indicate that the maximum measured concrete strain in compression was 0.0033. The results also indicate that the longitudinal steel reinforcement had yielded before failure, as evident by the non-linear behavior of the load-strain relationship prior to failure, as shown in Figure A-106.

![Graph of measured flexure strains](image)

**Figure A-106 Measured Flexure Strains for ShearBeams.4ksi.AnchoredWrapsUD.011911**

The measured axial and transverse load-strain relationships for the CFRP U-Wraps next to the supports are shown in Figure A-107 and Figure A-108. These findings are separated
according to their placement on the shear beam, either to the right or left of the load points, as shown in Figure 3-23. These figures also indicate whether the strain instrumentation was put on the front side or the back side of the beam, as shown in Figure 3-23. These results indicate that shear cracks formed at all four locations of shear strain instrumentation. As shown in Figure A-107 and Figure A-108, the transverse steel reinforcement near the supports had yielded before failure as evident at the large tension strains that were measured.
The T-beam failed at an applied load of 204.2 kips, compared to 133.9 kips measured for the control beam and 181.5 kips for the strengthened beam without the anchorage. The failure was due to rupture of the CFRP anchors and the corresponding debonding of the CFRP U-Wraps due to the propagation of a critical shear crack from a load point to the support, as shown in Figure A-109.
The deflection at failure of 0.97 inches was measured, compared to 0.52 inches for the control specimen and 0.71 inches for the strengthened beam without anchorage. The overall view of the beam at failure is shown in Figure A-110.
Test results show that the load carrying capacity was increased by 34% when compared to the control specimen and 26% for the beam without anchorage. It was also seen from these results that the deflection at failure of the specimen increased by 88% compared to the control specimen. The increase in deflection can be associated with the higher loads achieved for this specimen compared to the control. The higher loads lead to an increase in the flexural deflection, which is the predominant form of deflection in the specimens.
This reinforced T-beam was cast using 8ksi target nominal compressive strength concrete on January 19, 2011. The T-beam was not strengthened with any CFRP system and was used as a control to study the effectiveness of the various shear strengthening systems for the T-beams cast with 8ksi target compressive strength concrete on the same day. The loading sequence for this specimen is shown in Figure A-111.

The T-beam was loaded up to a pre-determined ultimate load of 65.2 kips and then unloaded. The load-deflection behavior, shown in Figure A-111, is linear up to the initiation of the first flexure crack, which occurred at an applied load level of 20.2 kips. The behavior after the initiation of first flexure crack was nonlinear with a significant reduction of the stiffness. In the second loading stage, the T-beam was loaded until failure. The load-deflection behavior
was nonlinear as shown in Figure A-111 during this loading stage until yielding of the transverse reinforcing steel. At this point, the behavior of the load-deflection graph had a reduction in the stiffness due to the yielding of the transverse reinforcing steel at a load of 82.2 kips, as shown in Figure A-111. The load-deflection behavior followed a nonlinear behavior until failure. The measured flexure load-strain relationships of the concrete at the top compression surface and bottom tension surface of the concrete as well at the level of longitudinal tension steel for the control shear T-beam are shown in Figure A-112. These findings indicate that the maximum measured concrete strain in compression was 0.0013. Test results indicate that not all the longitudinal steel reinforcement had yielded before failure, as evident in the linear behavior of the load-strain relationship, as shown in Figure A-112. This figure also indicates that the yielding strain had been reached prior to failure, suggesting that the extreme fiber of the first layer of steel may have begun to yield prior to failure of the beam.
The measured axial and transverse load-strain relationships for the concrete surface near the supports are shown in Figure A-113 and Figure A-114. These findings are separated according to their placement on the shear beam, either to the right or left of the load points, as shown in Figure 3-23. These figures also indicate whether the strain instrumentation was put on the front side or the back side of the beam, as shown in Figure 3-23. These results indicate that shear cracks formed at all four locations of shear strain instrumentation. From Figure A-113 and Figure A-114, it is shown that the transverse steel reinforcement had yielded near the supports prior to failure which is evident by the large tension strains that were measured at these locations.
Figure A-113 Measured Left-End Axial and Transverse Strains for 8ksi Control Shear Beam

Figure A-114 Measured Right-End Axial and Transverse Strains for 8ksi Control Shear Beam
The T-beam failed at a load of 178.5 kips due to the propagation of a critical shear crack from a load point to the support, as shown in Figure A-115.

Figure A-115 Critical Shear Crack for 8ksi Control Shear Beam

The deflection at failure of 0.66 inches was measured and the overall view of the beam at failure is shown in Figure A-116. At failure, there was propagation of a few well-distributed flexure cracks in the web, as shown in Figure A-117, and the failure was very brittle in nature.
Figure A-116 Failure of 8ksi Control Shear Beam

Figure A-117 Distribution of Flexure Cracks of 8ksi Control Shear Beam at Failure
A-21 ShearBeam.8ksi_WRAPsUD.011911

This reinforced T-beam was cast using 8ksi target nominal compressive strength concrete on January 19, 2011. The T-beam was strengthened with uni-directional CFRP U-Wraps. The strengthening system consisted of seven 12” wide U-wrap sheets spaced 15” center to center wrapped around the bottom surface web of the beam up to the bottom of the flange, as shown in Figure A-118. The measured load-deflection at mid-span for this specimen is shown in Figure A-119.

Figure A-118 Uni-directional CFRP U-Wrap Shear Reinforcing
The T-beam was loaded up to a pre-determined ultimate load of 85.1 kips and then unloaded. The load-deflection behavior shown in Figure A-119, was linear during this stage of loading up to the initiation of the first flexure crack, which occurred at an applied load level of 24.8 kips, compared to 20.2 kips for the control specimen. The increase of the cracking load capacity, in comparison to the control T-beam, is due to the presence of the CFRP U-wraps which delayed the initiation of the cracking load. The behavior after the initiation of first crack was nonlinear with a reduction of the stiffness. In the second loading stage, the T-beam was loaded until failure. The load-deflection behavior was nonlinear as shown in Figure A-119 until yielding of the transverse reinforcing steel. At this point, the behavior of the load-deflection graph had a reduction in the stiffness due to the yielding of the transverse reinforcing steel at a load of 95.5 kips, compared to 82.2 kips for the control specimen, as shown in Figure A-119. The increase of the yielding capacity in comparison to the control
T-beam is due to the presence of the CFRP U-Wraps which delayed the initiation of the yielding load. The load-deflection behavior followed a nonlinear relationship until the longitudinal steel reinforcement yielded, at a load of 178.8 kips. At this point, there was a significant reduction in the stiffness and the load-deflection behavior followed a nonlinear behavior until T-beam failed due to the debonding of the CFRP U-wrap from the formation of a critical shear crack. The measured flexure load-strain relationships of the concrete at the top compression surface, bottom surface of the CFRP U-wrap strengthening system, and at the level of longitudinal tension steel are shown in Figure A-120. The measured strain indicates that the maximum measured concrete strain in compression was 0.002. The results also indicate that the longitudinal steel reinforcement had yielded at a strain of 0.0026 before failure.

![Figure A-120 Measured Flexure Strains for ShearBeams.8ksi.WrapsUD.011911](image)
The measured axial and transverse load-strain relationships for the CFRP U-wraps located next to the supports are shown in Figure A-121 and Figure A-122. These findings are separated according to their placement on the shear beam, either to the right or left of the load points, as shown in Figure 3-23. These figures also indicate whether the strain instrumentation was put on the front side or the back side of the beam, as shown in Figure 3-23. These results indicate that shear cracks formed at all four locations of shear strain instrumentation and that there were greater shear strains on the side that experienced the critical shear crack. From Figure A-121 and Figure A-122, it is shown that transverse steel reinforcement near the supports had yielded prior to failure of the beam.

![Graph](image-url)

**Figure A-121 Measured Left-End Axial and Transverse Strains for ShearBeam.8ksi.WrapsUD.011911**
The T-beam failed at an applied load of 188.2 kips, compared to 178.5 kips for the control shear specimen. The failure was due to debonding of the CFRP U-wrap as a result of the propagation of a critical shear crack from a load point to the support, as shown in Figure A-123.
The deflection at failure was 0.76 inches, compared to 0.66 inches for the control beam. The overall view of the beam at failure is shown in Figure A-124.
Test results show that the load carrying capacity was increased by 5% when compared to the control beam. It was also seen from these results that the deflection at failure of the specimen increased by 16% compared to the control beam. The increase in deflection can be associated with the higher loads achieved for this specimen compared to the control. The higher loads lead to an increase in the flexural deflection, which is the predominant form of deflection in the specimens. There was also no change of the failure mode compared to the control beam.
A-22  ShearBeam.8ksi.AnchoredWrapsUD.011911
This reinforced T-beam specimen was cast using 8ksi target nominal compressive strength concrete on January 19, 2011. The T-beam was strengthened with anchored uni-directional CFRP U-Wraps. This strengthening system consisted of 3 CFRP anchors placed into drilled holes at the top of the web at the location of each wrap, as shown in Figure A-125. Uni-directional CFRP U-wraps were placed over these anchors which were spaced 15” center to center wrapped around the bottom surface of the web up to the bottom of the flange, as shown in Figure A-126. The measured load-deflection at mid-span for this specimen is shown in Figure A-127.
Figure A-126 Uni-directional CFRP U-Wraps with Anchorage
The T-beam was loaded up to a pre-determined ultimate load of 90.4 kips and then unloaded. The load-deflection behavior shown in Figure A-127, was linear during this loading stage up to the initiation of the first flexure crack, which occurred at an applied load level of 21.5 kips, compared to 20.2 kips for the control specimen. The increase of the cracking load capacity, in comparison to the control T-beam, is due to the presence of the CFRP U-wraps which delayed the initiation of the cracking load. The behavior after the initiation of first flexure crack was nonlinear with a reduction of the stiffness. In the second loading stage, the T-beam was loaded until failure. The load-deflection behavior was nonlinear as shown in Figure A-127 until yielding of the transverse reinforcing steel. At this point, the behavior of the load-deflection graph had a reduction in the stiffness due to the yielding of the transverse reinforcing steel at an applied load of 101.5 kips, compared to 82.2 kips for the control specimen, as shown in Figure A-127. The increase of the yielding capacity in comparison to
the control T-beam is due to the presence of the CFRP U-Wraps which delayed the initiation of the yielding load. The load-deflection behavior followed a nonlinear relationship until yielding of the longitudinal reinforcing steel, at an applied load of 176.9 kips, compared to 155.5 kips for the control beam. There was a reduction in the stiffness at this point, as shown in Figure A-127, and the load-deflection behavior followed a nonlinear relationship until the T-beam failed due to the formation of a critical shear crack. The measured flexure load-strain relationships for the top concrete compression surface, bottom surface of the CFRP U-wraps, and at the level of longitudinal tension steel are shown in Figure A-120. These findings indicate that the maximum measured concrete strain in compression was 0.0022. The results indicate that all the longitudinal steel reinforcement had yielded at a strain of 0.0025 before failure.

![Figure A-128 Measured Flexure Strains for ShearBeams.8ksi.AnchoredWrapsUD.011911](image-url)
The measured axial and transverse load-strain relationships for the CFRP U-wraps next to the support are shown in Figure A-129 and Figure A-130. These findings are separated according to their placement on the shear beam, either to the right or left of the load points, as shown in Figure 3-23. These figures also indicate whether the strain instrumentation was put on the front side or the back side of the beam, as shown in Figure 3-23. These results indicate that shear cracks formed at all four locations of shear strain instrumentation and that there were greater axial strains on the side of the T-beam where the critical crack had propagated. As shown in Figure A-129 and Figure A-130, the transverse steel near the supports had yielded prior to failure, evident through the high tension strains that were measured at these locations.

Figure A-129 Measured Left-End Shear Strains for ShearBeam.8ksi.AnchoredWrapsUD.011911
The T-beam failed at an applied load of 195.2 kips, compared to 188.2 kips and 178.5 kips for the beam strengthened with U-wraps only and the control beam, respectively. The failure was due to rupture of the CFRP anchors and corresponding debonding of the CFRP U-wraps as a result of the propagation of a critical shear crack from a load point to the support, as shown in Figure A-131.
The deflection at failure of 0.76 inches was measured, compared to 0.66 inches for the beam strengthened with U-wraps only and the control beam, respectively. The overall view of the beam at failure is shown in Figure A-132.
Test results show that the load carrying capacity was increased by 9% when compared to the control specimen. It was also seen from these results that the deflection at failure of the specimen increased by 16% compared to the control specimen. The increase in deflection can be associated with the higher loads achieved for this specimen compared to the control. The higher loads lead to an increase in the flexural deflection, which is the predominant form of deflection in the specimens.
This reinforced column was cast using 4ksi target nominal compressive strength concrete on March 7, 2011. The column was not strengthened with any CFRP system and was used as a control specimen to study the effectiveness of the various axial compressive strengthening systems for the columns cast with 4ksi target concrete strength on the same day. The measured load-axial deflection behavior for this specimen is shown in Figure A-133.

![Figure A-133 Load-Axial Deflection Behavior for 4ksi Control Column](image)

The column was loaded up to an equivalent pre-determined ultimate load of 209.4 kips and then loaded to 335 kips, at which stage it was unloaded. The load-axial deflection behavior shown in Figure A-133 was linear for this stage of loading while under passive confinement from the transverse reinforcement. Due to the passive nature of the transverse reinforcement, the confinement pressure increased linearly until the transverse reinforcement reached its yield strength. The transverse reinforcement provides passive confinement since confining
due to the transverse reinforcement is activated only when the concrete cone dilates. The confining pressure provided by the transverse reinforcement increased linearly with the increase of the transverse strain causing an increase in axial load capacity until the yielding of the transverse reinforcement. In the second loading stage; the column was loaded until failure. There was a decrease in the stiffness of the load-deflection behavior due to the formation of large vertical cracks in the core concrete due to yielding of the transverse ties and therefore could not control the opening of the vertical cracks. Figure A-133, shows an increase in load until yielding of the transverse steel, at an applied load of 565 kips, followed by a subsequent failure of the core concrete. Yielding of the transverse steel was confirmed by the measured load-strain relationships shown in Figure A-134 of the axial compressive and transverse tension strains of the concrete for the control column. The measured concrete strains represent the average strain within the gauge length of the pi gauges. The column failed at a load 566 kips due to crushing of the core concrete coinciding with buckling of the four longitudinal rebar, as shown in Figure A-135. The measured axial deflection at the maximum load was 0.13 inches, corresponding to an axial strain of 0.003, and there was significant spalling of the cover concrete, as shown in Figure A-135. Test results indicate that the maximum measured concrete strain in compression was 0.019 and that the maximum measured concrete strain in tension was 0.007, indicating that both the longitudinal and transverse reinforcement had yielded prior to failure as show in Figure A-134.
Figure A-134 Measured Strain for 4ksi Control Column
Figure A-135 Failure of 4ksi Control Column
This reinforced column was cast using 4ksi target nominal compressive strength concrete on March 7, 2011. The column was strengthened with one layer of uni-directional CFRP. The one layer of axial compression strengthening consisted of three 12” wide CFRP sheets placed to cover the entire effective height of the column, as shown in Figure A-136. The measured load-axial deflection behavior for this specimen is shown in Figure A-137.
The column was loaded up to an equivalent pre-determined ultimate load of 232 kips, and was unloaded. The measured load-axial deflection behavior shown in Figure A-137 was linear when the applied load induced stresses equivalent to the ultimate strength of the concrete. Initiation of the vertical cracks create a dilation of the concrete core and activated the steel ties, causing a linear increase of the axial load until yielding of the ties and the confinement pressure became constant. At this stage, the CFRP became more active and increased the axial load with a lower stiffness as shown in Figure A-137 due to the lower elastic modulus of the CFRP in comparison to the steel ties. This is evident by the non-linear behavior of the column beyond yielding of the transverse reinforcement. The load-deflection behavior followed a linear relationship until the yielding of the transverse reinforcement, at a load of 551 kips, compared to 565 kips for the control column. Yielding of the transverse reinforcement was confirmed when examining the measured load-strain relationships of the axial compressive and transverse tensile strains of the column at mid-height are shown in

Figure A-137 Load-Axial Deflection Behavior for Column.4ksi.1UD.030711
Figure A-138. The load-deflection behavior indicates that the presence of the FRP strengthening system is not fully activated until the core concrete begins to expand rapidly after yielding of the transverse reinforcement. The load-deflection behavior became nonlinear and there was a reduction of the stiffness once the yielding of transverse steel occurred, as shown in Figure A-137. The decrease in the stiffness after yielding of the transverse reinforcement is due to the elastic modulus of the CFRP is less than that of the transverse steel and the load-deflection behavior followed a non-linear behavior, as shown in Figure A-137. The additional load carrying capacity of the column after this stage is a function of the FRP only. The behavior of the load-deflection graph had a significant reduction of the stiffness near failure due to a rapid dilation of the core concrete and the inability of the transverse reinforcement to increase the confinement pressure on the core concrete, as shown in Figure A-137. After reaching the maximum applied axial compressive load of 655 kips, compared to 566 kips for the control column, the CFRP was unable to provide adequate confinement to control the deformation of the core concrete, which lead to the rapid dilation of the core concrete correlating with an increase in the transverse strain in the concrete. The column failed due to rupture of the CFRP confining wraps in the hoop direction which lead to crushing of the core concrete and subsequent buckling of the longitudinal bars, as shown in Figure A-139. The measured axial deflection at the maximum load was 0.18 inches, corresponding to an axial strain of 0.0041 in/in, compared to 0.003 in/in for the control. Test results indicate that the maximum measured strain in compression was 0.018 and that the maximum measured strain in tension was 0.023, indicating that both the longitudinal and transverse reinforcing steel had yielded before failure as shown in Figure A-138.
Figure A-138 Measured Strain for Column.4ksi.1UD.030711
Test results show that the load carrying capacity was increased by 13% when compared to the control specimen. It was also seen from these results that the axial strain at maximum load increased by 38% compared to the control specimen. The increase in axial strain and maximum load can be associated with the delay of the deterioration of the core concrete due to the additional confinement provided by the CFRP wraps.
This reinforced column was cast using 4ksi target nominal compressive strength concrete on March 7, 2011. The column was strengthened with two layers of uni-directional CFRP. This strengthening system consisted of two layers of three 12” wide CFRP sheets placed to cover the entire effective height of the column, as shown in Figure A-140. The measured response for this specimen is shown in Figure A-141.

Figure A-140 Application of Second Layer of CFRP Wraps
The column was loaded up to an equivalent pre-determined ultimate load of 254 kips, and was unloaded. The measured load-axial deflection behavior shown in Figure A-141 was linear when the applied load induced stresses equivalent to the ultimate strength of the concrete. Initiation of the vertical cracks create a dilation of the concrete core and activated the steel ties, causing a linear increase of the axial load until yielding of the ties and the confinement pressure became constant. At this stage, the CFRP became more active and increased the axial load with a lower stiffness as shown in Figure A-141 due to the lower elastic modulus of the CFRP in comparison the steel ties. This is evident by the non-linear behavior of the column beyond yielding of the transverse reinforcement. The load-deflection behavior followed a linear relationship until the yielding of the transverse reinforcement, at a load of 686 kips, compared to 565 kips for the control column. Yielding of the transverse steel was confirmed when examining the measured load-strain relationships of the axial compressive and transverse tensile strains of the column at mid-height are shown in Figure A-141.
The delay in the yielding of the transverse reinforcement can be associated with presence of the CFRP strengthening system, which increase the transverse reinforcement ratio and controls the outward expansion of the core concrete, causing a delay of the yielding of the transverse reinforcement. At this point, the transverse rebar provided active constant confinement of the core concrete because due to yielding of the transverse reinforcement. The decrease in the stiffness after yielding of the transverse reinforcement is due to the elastic modulus of the CFRP, is less than that of the transverse steel and the load-deflection behavior followed a non-linear behavior, as shown in Figure A-141. The additional load carrying capacity of the column after this stage is a function of the FRP only. The behavior of the load-deflection graph had a significant reduction of the stiffness near failure due to a rapid dilation of the core concrete and the inability of the transverse reinforcement to increase the confinement pressure on the core concrete, as shown in Figure A-141. After reaching the first maximum applied axial compressive load of 723 kips, the core concrete began to degrade quickly, which was controlled by the confinement pressure of the CFRP. Because the column was well confined with two layers of CFRP wrapping, the column did not fail at this maximum load but rather experienced a second maximum load. At this applied load of 756 kips, compared with 566 kips for the control column, there was rapid dilation of the core concrete and an increase lateral strain, as shown in Figure A-142, in the concrete. The column failed at this load due to rupture of the CFRP confining wraps in the hoop direction which lead to crushing of the core concrete and subsequent buckling of the longitudinal bars, as shown in Figure A-143. The measured axial deflection at the maximum load was 0.58 inches, corresponding to an axial strain of 0.0132 in/in compared to 0.003 in/in for the control. Test results indicate that the maximum measured strain in compression was 0.022 and that the maximum measured strain in tension was 0.042, as shown in Figure A-142.
Figure A-142 Measured Strain for Column.4ksi.2UD.030711
Test results show that the load carrying capacity was increased by 25% when compared to the control specimen. It was also seen from these results that the axial strain at maximum load increased by 346% compared to the control specimen. The increase in axial strain and maximum load can be associated with the delay of the deterioration of the core concrete due to the additional confinement provided by the CFRP wraps.
This reinforced column was cast using 8ksi target nominal compressive strength concrete on March 7, 2011. The column was not strengthened with any CFRP system and was used as the control column to study the effectiveness of the various axial compressive strengthening systems for the columns cast with 8ksi target concrete strength on the same day. The measured load-axial deflection for this specimen is shown in Figure A-144.

![Graph showing load-axial deflection response for 8ksi control column with labels for yielding, concrete crush, predicted failure load, and ultimate load.](image)

**Figure A-144 Load-Axial Deflection Response for 8ksi Control Column**

The column was loaded up to an equivalent pre-determined ultimate load of 308 kips at which stage it was unloaded. The load-axial deflection behavior shown in Figure A-144 was linear for this stage of loading while under passive confinement from the transverse reinforcement. Due to the passive nature of the transverse reinforcement, the confinement pressure increased linearly until the transverse reinforcement reached its yield strength. The transverse reinforcement provides passive confinement since the confining of the transverse
reinforcement is activated only when the concrete cone dilates. The confining pressure provided by the transverse reinforcement increased linearly with the increase of the transverse strain causing an increase in the axial load capacity until yielding of the transverse reinforcement. In the second loading stage; the column was loaded until failure. There was a reduction of the stiffness of the load-deflection behavior due to yielding of the transverse reinforcement at a load of 861 kips, as shown in Figure A-144. Yielding of the transverse steel was confirmed by examining the measured load-strain relationships of the axial compressive and transverse tension strains of the concrete for the control column that are shown in Figure A-145. The measured concrete strains represent the average strain within the gauge length of the pi gauge. The behavior of the load-deflection graph was nonlinear during this portion of the loading due to the propagation of vertical cracks and dilation of the core as shown in Figure A-144. The column failed at a load 1001 kips due to crushing of the core concrete coinciding with buckling of the four longitudinal rebar reinforcement. The measured axial deflection at the maximum load was 0.14 inches, corresponding to an axial strain of 0.0032, and there was significant spalling of the cover concrete, as shown in Figure A-146. Test results indicate that the maximum measured concrete strain in compression was 0.0033 and that the maximum measured concrete strain in tension was 0.0025, indicating that the transverse and longitudinal steel had yielded as shown in Figure A-145.
Figure A-145: Measured Strain for 8ksi Control Column
Figure A-146 Failure for 8ksi Control Column
This reinforced column was cast using 8ksi target compressive strength concrete on March 7, 2011. The column was strengthened with one layer of uni-directional CFRP. The one layer of axial compression strengthening consisted of three 12” wide CFRP sheets placed to cover the entire effective height of the column, as shown in Figure A-147. The measured load-axial deflection behavior for this specimen is shown in Figure A-148.
The column was loaded up to an equivalent pre-determined ultimate load of 403 kips, and was unloaded. The measured load-axial deflection behavior shown in Figure A-148 was linear when the applied load induced stresses equivalent to the ultimate strength of the concrete. Initiation of the vertical cracks create a dilation of the concrete core and activated the steel ties, causing a linear increase of the axial load until yielding of the ties and the confinement pressure became constant. At this stage, the CFRP became more active and increased the axial load with a lower stiffness as shown in Figure A-148 due to the lower elastic modulus of the CFRP in comparison the steel ties. This is evident by the non-linear behavior of the column beyond yielding of the transverse reinforcement. The load-deflection behavior followed a linear relationship until the yielding of the transverse reinforcement, at a load of 1001 kips, compared to 861 kips for the control column. The delay in the yielding of the transverse reinforcement can be associated with presence of the CFRP strengthening system, which acts to controls the outward expansion of the core concrete, causing a delay in
yielding of the transverse reinforcement. Yielding of the transverse reinforcement was confirmed by the measured load-strain relationship of the axial compressive and transverse tensile strains of the column at mid-height as shown in Figure A-149. The load-deflection behavior indicates that the presence of the FRP strengthening system is not fully activated until the core concrete begins to expand rapidly yielding of the transverse reinforcement. The load-deflection behavior became nonlinear and there was a reduction of the stiffness once the yielding of transverse steel occurred, as shown in Figure A-148. There was a decrease in the stiffness after yielding of the transverse reinforcement because the elastic modulus of the CFRP, which is now fully activated, is less than that of the transverse steel and the load-deflection behavior followed a non-linear behavior, as shown in Figure A-148. The additional load carrying capacity of the column after this stage is a function of the FRP only. The behavior of the load-deflection graph had a significant reduction of the stiffness near failure due to a rapid dilation of the core concrete and the inability of the transverse reinforcement to increase the confinement pressure on the core concrete, as shown in Figure A-148. After reaching the maximum applied axial compressive load of 1046 kips, compared to 1001 kips for the control column, the CFRP was unable to provide adequate confinement to control the deformation of the core concrete, which lead to the rapid dilation of the core concrete correlating with an increase in the lateral strain in the concrete. The column failed due to rupture of the CFRP confining wraps in the hoop direction which lead to crushing of the core concrete and subsequent buckling of the longitudinal bars, as shown in Figure A-150. The measured deflection at the maximum load was 0.20 inches, which corresponds to an axial strain of 0.0045 in/in, compared to 0.0032 in/in for the control. Test results also indicate that the maximum measured strain in compression was 0.0032 and that the maximum measured strain in tension was 0.013, indicating that the longitudinal and transverse steel yielded as shown in Figure A-149.
Figure A-149 Measured Strain for Column.8ksi.1UD.030711
Test results show that the load carrying capacity was increased by 4% when compared to the control specimen. It was also seen from these results that the axial strain at the maximum load increased by 43% compared to the control specimen. The increase in axial strain and maximum load can be associated with the delay of the deterioration of the core concrete due to the additional confinement provided by the CFRP wraps.
This reinforced column was cast using 8ksi target nominal compressive strength concrete on March 7, 2011. The column was strengthened with two layers of uni-directional CFRP. This axial compression strengthening system consisted of two layers of three 12” wide CFRP sheets placed to cover the entire effective height of the column, as shown in Figure A-151. The measured load-axial deflection behavior for this specimen is shown in Figure A-152.
The column was loaded up to an equivalent pre-determined ultimate load of 425 kips, and was unloaded. The measured load-axial deflection behavior shown in Figure A-152 was linear when the applied load induced stresses equivalent to the ultimate strength of the concrete. Initiation of the vertical cracks create a dilation of the concrete core and activated the steel ties, causing a linear increase of the axial load until yielding of the ties and the confinement pressure became constant. At this stage, the CFRP became more active and increased the axial load with a lower stiffness as shown in Figure A-152 due to the lower elastic modulus of the CFRP in comparison the steel ties. This is evident by the non-linear behavior of the column beyond yielding of the transverse reinforcement. The load-deflection behavior followed a linear relationship until the formation and propagation of large vertical cracks through the core concrete, as shown in Figure A-152, at which point the load-deflection behavior followed a non-linear behavior until yielding of the transverse reinforcement, at a load of 1151 kips, compared to 861 kips for the control column. The
delay in the yielding of the transverse reinforcement can be associated with presence of the CFRP strengthening system, which acts to control the outward expansion of the core concrete, causing a delay in yielding of the transverse reinforcement. Yielding of the transverse reinforcement was verified through the measured load-strain relationships of the axial compressive and transverse tensile strains of the column at mid-height are shown in Figure A-153. The load-deflection behavior became nonlinear and there was a reduction of the stiffness once the yielding of transverse steel occurred, as shown in Figure A-152. The behavior of the load-deflection graph had a significant reduction of the stiffness near failure due to a rapid dilation of the core concrete and the inability of the transverse reinforcement to increase the confinement pressure on the core concrete, as shown in Figure A-152. After reaching the maximum applied axial compressive load of 1151 kips, compared to 1001 kips for the control column, the CFRP was unable to provide adequate confinement to control the deformation of the core concrete, which lead to the rapid dilation of the core concrete correlating with an increase in the lateral strain in the concrete. The column failed due to rupture of the CFRP confining wraps in the hoop direction which lead to crushing of the core concrete and subsequent buckling of the longitudinal bars, as shown in Figure A-154. The measured deflection at the maximum load was 0.19 inches, which corresponds to an axial strain of 0.0043 in/in, compared to 0.0032 in/in for the control. Test results also indicate that the maximum measured strain in compression was 0.0044 and that the maximum measured strain in tension was 0.0084, indicating the longitudinal and transverse reinforcement had yielded as shown in Figure A-153.
Figure A-153 Measured Strain for Column.8ksi.2UD.030711
Test results show that the load carrying capacity was increased by 5% when compared to the control specimen. It was also seen from these results that the axial strain at the maximum load increased by 36% compared to the control specimen. The increase in axial deflection and maximum load can be associated with the delay of the deterioration of the core concrete due to the additional confinement provided by the CFRP wraps.
APPENDIX B

This appendix provides detailed test results for the material test of this experimental program.
**B-1 Steel Reinforcement Test Data**

The results for the material tests on the steel reinforcement are given in Table B-1. The yield load, yield stress, max load, and max stress are reported. The average of these values was used in the analysis of the reinforced concrete structures. The specimens having the letter A in their specimen ID are from the batch of reinforcing steel that was used in the concrete specimens cast on 05/12/11. All other specimens were from the steel used in the concrete specimens cast on 01/19/11.

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<th>Max Load (kips)</th>
<th>Max Stress (ksi)</th>
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*Table B-1 Yield and max load and stress*
The results for the rupture strain for each material test on the steel reinforcement is given in Table B-2. The average of these results was used in the analysis of the reinforced concrete specimens. The specimens having the letter A in their specimen ID are from the batch of reinforcing steel that was used in the concrete specimens cast on 05/12/11. All other specimens were from the steel used in the concrete specimens cast on 01/19/11.

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Table B-2 Rupture Strain

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B-2 Concrete Properties Test Data

The results for the concrete material properties tests are given in Table B-3, Table B-4, and Table B-5. These results are separated into date of cast, target concrete strength, and from when the concrete samples were taken during the casting of the reinforced concrete specimens.

### Table B-3 Concrete strength for beams and slabs cast on 01/19/11

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<th>Specimen</th>
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<th>Max Load (lbs)</th>
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### Table B-4 Concrete strength for columns cast on 01/19/11

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<td>4ksi- #2</td>
<td>3.9673</td>
<td>12.36</td>
<td>49,338</td>
<td>3,990</td>
</tr>
<tr>
<td>4ksi- #3</td>
<td>3.9953</td>
<td>12.54</td>
<td>52,505</td>
<td>4,185</td>
</tr>
<tr>
<td>8ksi- #1</td>
<td>3.9478</td>
<td>12.24</td>
<td>99,214</td>
<td>8,105</td>
</tr>
<tr>
<td>8ksi- #2</td>
<td>4.0000</td>
<td>12.57</td>
<td>99,808</td>
<td>7,940</td>
</tr>
<tr>
<td>8ksi- #3</td>
<td>4.0008</td>
<td>12.57</td>
<td>98,931</td>
<td>7,870</td>
</tr>
</tbody>
</table>
### Table B-5 Concrete strength for beams and slabs cast on 05/12/11

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Average Diameter (in)</th>
<th>Area (in^2)</th>
<th>Max Load (lbs)</th>
<th>Max Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4ksi- #1A</td>
<td>4.0335</td>
<td>12.78</td>
<td>63,786</td>
<td>4,991</td>
</tr>
<tr>
<td>4ksi- #2A</td>
<td>4.0310</td>
<td>12.76</td>
<td>64,097</td>
<td>5,022</td>
</tr>
<tr>
<td>4ksi- #3A</td>
<td>4.0303</td>
<td>12.76</td>
<td>62,684</td>
<td>4,914</td>
</tr>
</tbody>
</table>
B-3 CFRP Material Properties

The dimensions of the specimens used in the material tests on the carbon fiber reinforced polymer sheets used in strengthening the large concrete specimens are given in Table B-6. The width, thickness, and nominal thickness are reported. The average of the widths and the nominal thickness was used in the analysis of the concrete structures strengthened with either the uni-directional or bi-directional CFRP system.

<table>
<thead>
<tr>
<th>Material</th>
<th>Specimen</th>
<th>Width (inches)</th>
<th>Thickness (inches)</th>
<th>Nominal Thickness (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uni-directional</td>
<td>UD-A</td>
<td>0.930</td>
<td>0.045</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>UD-B</td>
<td>0.969</td>
<td>0.042</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>UD-C</td>
<td>0.953</td>
<td>0.046</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>UD-D</td>
<td>0.984</td>
<td>0.040</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>UD-E</td>
<td>1.014</td>
<td>0.049</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>UD-F</td>
<td>0.970</td>
<td>0.036</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>UD-G</td>
<td>1.002</td>
<td>0.042</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>UD-H</td>
<td>0.960</td>
<td>0.038</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>UD-I</td>
<td>0.987</td>
<td>0.046</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>UD-J</td>
<td>0.978</td>
<td>0.041</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>UD-K</td>
<td>0.967</td>
<td>0.037</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>UD-L</td>
<td>0.985</td>
<td>0.045</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>UD-M</td>
<td>0.956</td>
<td>0.039</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>UD-N</td>
<td>0.976</td>
<td>0.046</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>UD-O</td>
<td>0.944</td>
<td>0.050</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>UD-P</td>
<td>1.014</td>
<td>0.042</td>
<td>0.02</td>
</tr>
</tbody>
</table>

Table B-6 Continued
The results of the material tests on the carbon fiber reinforced polymer sheets used in strengthening the large concrete specimens are given in Table B-7. The peak load, stress at ultimate, and elastic modulus is reported. The average of these values was used in the analysis of the concrete structures strengthened with either the uni-directional or bidirectional CFRP system.

**Table B-7 Material Test Results for CFRP Sheets**

<table>
<thead>
<tr>
<th>Material</th>
<th>Specimen</th>
<th>Peak Load (kips)</th>
<th>Stress at Ultimate (ksi)</th>
<th>Elastic Modulus (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uni-directional</td>
<td>UD-A</td>
<td>4.04</td>
<td>217</td>
<td>12,387</td>
</tr>
<tr>
<td></td>
<td>UD-B</td>
<td>4.41</td>
<td>228</td>
<td>14,137</td>
</tr>
<tr>
<td></td>
<td>UD-C</td>
<td>4.52</td>
<td>237</td>
<td>13,112</td>
</tr>
<tr>
<td></td>
<td>UD-D</td>
<td>4.43</td>
<td>225</td>
<td>15,009</td>
</tr>
<tr>
<td></td>
<td>UD-E</td>
<td>4.80</td>
<td>237</td>
<td>14,737</td>
</tr>
<tr>
<td></td>
<td>UD-F</td>
<td>3.78</td>
<td>195</td>
<td>14,923</td>
</tr>
<tr>
<td>Bi-directional</td>
<td>BD-A</td>
<td>0.958</td>
<td>0.052</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>BD-B</td>
<td>0.948</td>
<td>0.051</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>BD-C</td>
<td>0.969</td>
<td>0.050</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>BD-D</td>
<td>0.954</td>
<td>0.052</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>BD-E</td>
<td>0.973</td>
<td>0.049</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>UD-G</td>
<td>4.60</td>
<td>230</td>
<td>13,597</td>
</tr>
<tr>
<td>--------</td>
<td>------</td>
<td>------</td>
<td>-----</td>
<td>--------</td>
</tr>
<tr>
<td>UD-H</td>
<td>4.60</td>
<td>240</td>
<td></td>
<td>13,477</td>
</tr>
<tr>
<td>UD-I</td>
<td>4.33</td>
<td>220</td>
<td></td>
<td>13,145</td>
</tr>
<tr>
<td>UD-J</td>
<td>4.83</td>
<td>247</td>
<td></td>
<td>13,639</td>
</tr>
<tr>
<td>UD-K</td>
<td>4.61</td>
<td>239</td>
<td></td>
<td>14,627</td>
</tr>
<tr>
<td>UD-L</td>
<td>4.65</td>
<td>236</td>
<td></td>
<td>13,094</td>
</tr>
<tr>
<td>UD-M</td>
<td>4.18</td>
<td>219</td>
<td></td>
<td>14,707</td>
</tr>
<tr>
<td>UD-N</td>
<td>4.73</td>
<td>243</td>
<td></td>
<td>13,520</td>
</tr>
<tr>
<td>UD-O</td>
<td>4.55</td>
<td>241</td>
<td></td>
<td>13,785</td>
</tr>
<tr>
<td>UD-P</td>
<td>4.58</td>
<td>226</td>
<td></td>
<td>12,559</td>
</tr>
<tr>
<td>UD-Q</td>
<td>4.32</td>
<td>217</td>
<td></td>
<td>14,303</td>
</tr>
<tr>
<td>UD-R</td>
<td>4.18</td>
<td>216</td>
<td></td>
<td>15,341</td>
</tr>
<tr>
<td>UD-S</td>
<td>3.73</td>
<td>193</td>
<td></td>
<td>13,347</td>
</tr>
<tr>
<td>UD-T</td>
<td>4.35</td>
<td>219</td>
<td></td>
<td>11,757</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>Bi-directional</th>
</tr>
</thead>
<tbody>
<tr>
<td>BD-A</td>
<td>3.94</td>
</tr>
<tr>
<td>BD-B</td>
<td>4.17</td>
</tr>
<tr>
<td>BD-C</td>
<td>3.91</td>
</tr>
<tr>
<td>BD-D</td>
<td>4.07</td>
</tr>
<tr>
<td>BD-E</td>
<td>3.95</td>
</tr>
</tbody>
</table>
APPENDIX C

This appendix provides detailed sample calculations for the different IC debonding models.
All sample calculations will be conducted on Slab.8ksi.1UD.011911, whose section properties are listed below.

**Slab Properties**

- **b** = 24 in            Diameter of Tension Steel = 0.5 in
- **h** = 6 in             Quantity of Tension Reinforcement Bars (n₀) = 5 bars
- **d** = 5 in             **σ**ₚ = 65,770 psi
- **f'c** = 7600 psi       **Es** = 29,000,000 psi

**CFRP Properties**

- **E_f** = 13,760,000 psi
- **ε_fu** = 0.01333
- **σ_fu** = 226,000 psi
- **t_f** = 0.02 in
- **d_f** = 6.01 in

**ACI 440.2R-08 IC Debonding Sample Calculation**
Determine Debonding Strain of FRP Strengthening System

- All units for the calculation of the debonding strain are SI (only)

\[
\varepsilon_{fd} = 0.41 \times \frac{f'_c}{nE_f t_f} \leq 0.9\varepsilon_{fu}
\]

\[
\varepsilon_{fd} = 0.41 \times \sqrt{\frac{52.40 \text{ Mpa}}{1 \times 94872 \text{ Mpa} \times 0.508 \text{ mm}}} \leq 0.9 \times 0.1333
\]

\[
\varepsilon_{fd} = 0.0135 \leq 0.01197
\]

Therefore the debonding strain of the FRP is \( \varepsilon_{fd} = 0.01197 \)

Assume a value for \( c \) = depth of the concrete compression zone

Will assume a value of \( c = 1.0 \) inch

Determine Strain in the reinforcing steel and at the top compression fiber of concrete
Determine Stress in FRP, tension steel, and the concrete

\[ \varepsilon_s = \frac{\varepsilon_{fd}}{d_f - c} \times (d - c) \]

\[ \varepsilon_s = \frac{0.01197}{6.01 \text{ in} - 1.0 \text{ in}} \times (5 \text{ in} - 1.0 \text{ in}) \]

\[ \varepsilon_s = 0.00956 \]

\[ \varepsilon_c = \frac{\varepsilon_{fd}}{d_f - c} \times (c) \]

\[ \varepsilon_c = \frac{0.01197}{6.01 \text{ in} - 1.0 \text{ in}} \times (1.0 \text{ in}) \]

\[ \varepsilon_c = 0.00239 \]

\[ \sigma_f = E_f \times \varepsilon_f \]

\[ \sigma_f = 13760000 \text{ psi} \times 0.01197 \]

\[ \sigma_f = 164707 \text{ psi} \]

Since the strain in the reinforcing steel is greater than the yield strain,

\[ \varepsilon_y = 0.0023 \]

the stress in the steel can be calculated using the following equation

\[ \sigma_s = \sigma_y \]

\[ \sigma_s = 65770 \text{ psi} \]

An approximation of the Hognestad Parabola will be used in determining the equivalent compression stress-block

\[ \sigma_c = \alpha \times f'_c \]

where

\[ \alpha = \frac{\varepsilon_c f'_c - \left(\frac{1}{3}\right) \times (\varepsilon_c f'_c)^2}{\beta} \]
where
\[
\beta = \left(4 - \frac{\epsilon'_c}{\epsilon_c'}\right)/(6 - \left(\frac{2\epsilon'_c}{\epsilon_c'}\right))
\]

where

\[
\epsilon'_c = 0.002
\]

therefore from Eq. (8a)

\[
\beta = \left(4 - \frac{0.00239}{0.002}\right)/\left[6 - \left(\frac{2 \times 0.00239}{0.002}\right)\right]
\]

\[
\beta = 0.777
\]

therefore from Eq. (7a)

\[
\alpha = \frac{0.00239 - \left(\frac{1}{3}\right) \times \left(\frac{0.00239}{0.002}\right)^2}{0.777}
\]

\[
\alpha = 0.925
\]

therefore from Eq. (6a)

\[
\sigma_c = 0.925 \times 7600\ psi
\]

\[
\sigma_c = 7030\ psi
\]

→ Determine Forces developed in the FRP, tension steel, and concrete

\[
F_f = \sigma_f \times A_f
\]

\[
F_f = 164707\ psi \times (n \times t_f \times b)
\]

\[
F_f = 164707\ psi \times (1 \times 0.02\ in \times 24\ in)
\]

\[
F_f = 79059\ lbs
\]

\[
F_s = \sigma_s \times A_s
\]

\[
F_s = 65770\ psi \times (n_s \times \left(\pi \times \frac{\text{diameter}^2}{4}\right))
\]

\[
F_s = 65770\ psi \times (5 \times \frac{\pi \times (0.5\ in)^2}{4})
\]

\[
F_s = 64454\ lbs
\]
An approximation of the Hognestad Parabola will be used in determining the equivalent compression force in the concrete

\[ F_c = \sigma_c \cdot A_c \]
\[ F_c = \sigma_c \cdot (\beta c \cdot b) \]
\[ F_c = 7030 \text{ psi} \cdot (0.777 \cdot 1.0 \text{ in} \cdot 24 \text{ in}) \]
\[ F_c = 131095 \text{ lbs} \]

→ Check for Force Equilibrium

\[ T = C \]
\[ T_f + T_s = C_c \]
\[ 79059 \text{ lb} + 64454 \text{ lb} = 131095 \text{ lb} \]
\[ 143514 \text{ lb} \neq 131095 \text{ lb} \]

Because equilibrium was not met with this assumed value for \( c \), a new value of \( c \) should be selected and the process iterated until a force equilibrium is met

→ Through iteration

\[ c = 1.07 \text{ in} \]

\[ T_f + T_s = C_c \]
\[ 79059 \text{ lb} + 64454 \text{ lb} = 143643 \text{ lb} \]
\[ 143514 \text{ lb} \approx 143643 \text{ lb} \]

Equilibrium condition is satisfied

→ Calculate moment capacity of the strengthened slab

\[ M_u = M_c + M_f + M_s \]

where

Eq. (13a)
where

\[ M_s = F_s \times (d_s - c) \]

\[ M_s = 64454 \text{ lb} \times (5 \text{ in} - 1.07 \text{ in}) \]

\[ M_s = 253304 \text{ in} - \text{lb} \]

\[ M_f = F_f \times (d_f - c) \]

\[ M_f = 79059 \text{ lb} \times (6.01 \text{ in} - 1.07 \text{ in}) \]

\[ M_f = 390551 \text{ in} - \text{lb} \]

Therefore from Eq. (13a)

\[ M_u = 92534 + 253304 + 390551 \text{ in} - \text{lb} \]

\[ M_u = 736389 \text{ in} - \text{lb} \]

\[ M_u = 61.4 \text{ ft} - k \]

\[ \Rightarrow \text{ Calculate the ultimate load carrying capacity of the strengthened slab} \]

\[ P_u = 2 \left( \frac{M_u}{\left(\frac{1}{2}\right) \times (\text{clear span - distance between load points})} \right) \]

\[ P_u = 2 \times \left( \frac{61.4 \text{ k-ft}}{\left(\frac{1}{2}\right) \times (12 \text{ ft} - 2 \text{ ft})} \right) \]

\[ P_u = 2 \times \frac{61.4 \text{ k-ft}}{5 \text{ ft}} \]

\[ P_u = 24.56 \text{ k-ft} \]
Said & Wu IC Debonding Sample Calculation

→ Determine Debonding Strain of FRP Strengthening System

➤ All units for the calculation of the debonding strain are SI (only)

\[ \varepsilon_{deb} = 0.23\left(\frac{f'_c}{E_f}t_f\right)^{0.2}/(E_f t_f)^{0.35} \]

\[ \varepsilon_{deb} = 0.23 \times (52.40 \text{ MPa})^{0.2}/(94872 \text{ MPa} \times 0.508 \text{ mm})^{0.35} \]

\[ \varepsilon_{deb} = 0.01166 \]

Therefore the debonding strain of the FRP is \( \varepsilon_{deb} = \varepsilon_{fd} = 0.01166 \)

→ Assume a value for \( c = \) depth of the concrete compression zone

Will assume a value of \( c = 1.0 \) inch

→ Determine Strain in the reinforcing steel and at the top compression fiber of concrete
from Eq. (2a)

\[ \varepsilon_s = \frac{\varepsilon_{f_d}}{d_f - c} \times (d - c) \]

\[ \varepsilon_s = \frac{0.01166}{6.01 \text{ in} - 1.0 \text{ in}} \times (5 \text{ in} - 1.0 \text{ in}) \]

\[ \varepsilon_s = 0.00931 \]

from Eq. (3a)

\[ \varepsilon_c = \frac{\varepsilon_{f_d}}{d_f - c} \times (c) \]

\[ \varepsilon_c = \frac{0.01166}{6.01 \text{ in} - 1.0 \text{ in}} \times (1.0 \text{ in}) \]

\[ \varepsilon_c = 0.00233 \]

→ Determine Stress in FRP, tension steel, and the concrete

from Eq. (4a)

\[ \sigma_f = E_f \times \varepsilon_f \]

\[ \sigma_f = 13760000 \text{ psi} \times 0.01166 \]

\[ \sigma_f = 160442 \text{ psi} \]

Since the strain in the reinforcing steel is greater than the yield strain,

\[ \varepsilon_y = 0.0023 \]

the stress in the steel can be calculated using the following equation

from Eq. (5a)

\[ \sigma_s = \sigma_y \]

\[ \sigma_s = 65770 \text{ psi} \]

An approximation of the Hognestad Parabola will be used in determining the equivalent compression stress-block from Eq. (6a)

\[ \sigma_c = \alpha \times f'_c \]
where from Eq. (7a)

\[ \alpha = \frac{\varepsilon'_{c} - \left(\frac{1}{3}\right) \times (\varepsilon'_{c})^2}{\beta} \]

where from Eq. (8a)

\[ \beta = \frac{4 - \varepsilon_{c}}{\varepsilon'_{c}} / \left(6 - \left(\frac{2\varepsilon_{c}}{\varepsilon'_{c}}\right)\right) \]

where

\[ \varepsilon'_{c} = 0.002 \]

therefore from Eq. (8a)

\[ \beta = \left(4 - \frac{0.00233}{0.002}\right) / \left[6 - \left(\frac{2 \times 0.00233}{0.002}\right)\right] \]

\[ \beta = 0.772 \]

therefore from Eq. (7a)

\[ \alpha = \frac{0.00233 - \left(\frac{1}{3}\right) \times (0.00233)^2}{0.772} \]

\[ \alpha = 0.923 \]

therefore from Eq. (6a)

\[ \sigma_{c} = 0.923 \times 7600 \text{psi} \]

\[ \sigma_{c} = 7015 \text{ psi} \]

→ Determine Forces developed in the FRP, tension steel, and concrete from Eq. (9a, 10a, 11a)

\[ F_{f} = \sigma_{f} \times A_{f} \]
An approximation of the Hognestad Parabola will be used in determining the equivalent compression force in the concrete.

\[ F_f = 160442 \, \text{psi} \times (n \times t_f \times b) \]

\[ F_f = 160442 \, \text{psi} \times (1 \times 0.02 \, \text{in} \times 24 \, \text{in}) \]

\[ F_f = 77012 \, \text{lbs} \]

\[ F_s = \sigma_s \times A_s \]

\[ F_s = 65770 \, \text{psi} \times (n_s \times \left(\frac{\pi \times \text{diameter}^2}{4}\right)) \]

\[ F_s = 65770 \, \text{psi} \times (5 \times \frac{\pi \times (0.5 \, \text{in})^2}{4}) \]

\[ F_s = 64454 \, \text{lbs} \]

An approximation of the Hognestad Parabola will be used in determining the equivalent compression force in the concrete.

\[ F_c = \sigma_c \times A_c \]

\[ F_c = \sigma_c \times (\beta c \times b) \]

\[ F_c = 7015 \, \text{psi} \times (0.772 \times 1.0 \, \text{in} \times 24 \, \text{in}) \]

\[ F_c = 129974 \, \text{lbs} \]

\[ \Rightarrow \text{Check for Force Equilibrium from Eq. (12a)} \]

\[ T = C \]

\[ T_f + T_s = C_c \]

\[ 77012 \, \text{lb} + 64454 \, \text{lb} = 129974 \, \text{lb} \]

\[ 141466 \, \text{lb} \neq 129974 \, \text{lb} \]

Because equilibrium was not met with this assumed value for c, a new value of c should be selected and the process iterated until a force equilibrium is met.

\[ \Rightarrow \text{Through iteration} \]
\( c = 1.06 \text{ in} \)

\[
T_f + T_s = C_c
\]

\[ 77012 \text{ lb} + 64454 \text{ lb} = 137775 \text{ lb} \]

\[ 141466 \text{ lb} \approx 137768 \text{ lb} \]

Equilibrium condition is satisfied

→ Calculate moment capacity of the strengthened slab from Eq. (13a)

\[
M_u = M_c + M_f + M_s
\]

where from Eq. (14a)

\[
M_c = F_c \ast (c - \frac{\beta c}{2})
\]

\[
M_c = 137768 \text{ lb} \ast (1.06 \text{ in} - \frac{0.7725 \ast 1.06 \text{ in}}{2})
\]

\[ M_c = 89628 \text{ in} - \text{lb} \]

where from Eq. (15a)

\[
M_s = F_s \ast (d_s - c)
\]

\[
M_s = 64454 \text{ lb} \ast (5 \text{ in} - 1.06 \text{ in})
\]

\[ M_s = 253949 \text{ in} - \text{lb} \]

where from Eq. (16a)

\[
M_f = F_f \ast (d_f - c)
\]

\[
M_f = 77012 \text{ lb} \ast (6.01 \text{ in} - 1.06 \text{ in})
\]

\[ M_f = 381209 \text{ in} - \text{lb} \]

therefore from Eq. (13a)

\[
M_u = 89628 + 253949 + 381209 \text{ in} - \text{lb}
\]

\[ M_u = 724786 \text{ in} - \text{lb} \]

\[ M_u = 60.40 \text{ ft} - k \]

→ Calculate the ultimate load carrying capacity of the strengthened slab from Eq. (17a)
\[ P_u = 2 \left( \frac{M_u}{\left( \frac{1}{2} \right) \ast (\text{clear span} - \text{distance between load points})} \right) \]

\[ P_u = 2 \left( \frac{60.40 \, k \, - \, ft}{\left( \frac{1}{2} \right) \ast (12 \, ft \, - \, 2 \, ft)} \right) \]

\[ P_u = 24.16 \, k \, - \, ft \]

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IC Debonding Sample Calculation

→ Determine Debonding Strain of FRP Strengthening System
   ✓ All units for the calculation of the debonding strain are SI (only)

<table>
<thead>
<tr>
<th>Eq. (19a)</th>
<th>( \sigma_f \leq \sqrt{2G_f E_f/t_f} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eq. (20a)</td>
<td>( G_f = 0.644 f_c^{0.19} )</td>
</tr>
<tr>
<td></td>
<td>( G_f = 0.644 \times (52.40 \text{ Mpa})^{0.19} )</td>
</tr>
<tr>
<td></td>
<td>( G_f = 1.366 \text{ N/mm} )</td>
</tr>
<tr>
<td></td>
<td>therefore from Eq. (19a)</td>
</tr>
<tr>
<td></td>
<td>( \sigma_f \leq \sqrt{\frac{2 \times 1.366 \times 94872 \text{ Mpa}}{0.508 \text{ mm}}} )</td>
</tr>
<tr>
<td></td>
<td>( \sigma_f \leq 714.4 \text{ Mpa} )</td>
</tr>
<tr>
<td></td>
<td>therefore</td>
</tr>
<tr>
<td></td>
<td>( \varepsilon_{deb} = \frac{E_f}{\sigma_f} )</td>
</tr>
<tr>
<td></td>
<td>( \varepsilon_{deb} = \frac{94872 \text{ Mpa}}{714.4 \text{ Mpa}} )</td>
</tr>
<tr>
<td></td>
<td>( \varepsilon_{deb} = 0.0075 )</td>
</tr>
<tr>
<td></td>
<td>Therefore the debonding strain of the FRP is ( \varepsilon_{deb} = \varepsilon_{fd} = 0.0075 )</td>
</tr>
</tbody>
</table>

→ Assume a value for \( c = \) depth of the concrete compression zone
Will assume a value of $c = 1.0$ inch

→ Determine Strain in the reinforcing steel and at the top compression fiber of concrete

from Eq. (2a)

$$\varepsilon_s = \frac{\varepsilon_{fd}}{d_f - c} \ast (d - c)$$

$$\varepsilon_s = \frac{0.0075}{6.01 \text{ in} - 1.0 \text{ in}} \ast (5 \text{ in} - 1.0 \text{ in})$$

$$\varepsilon_s = 0.00599$$

from Eq. (3a)

$$\varepsilon_c = \frac{\varepsilon_{fd}}{d_f - c} \ast (c)$$

$$\varepsilon_c = \frac{0.0075}{6.01 \text{ in} - 1.0 \text{ in}} \ast (1.0 \text{ in})$$

$$\varepsilon_c = 0.00150$$

→ Determine Stress in FRP, tension steel, and the concrete

from Eq. (4a)
Since the strain in the reinforcing steel is greater than the yield strain,

\[ \varepsilon_y = 0.0023 \]

the stress in the steel can be calculated using Eq. (5a)

\[ \sigma_s = \sigma_y \]
\[ \sigma_s = 65770 \text{ psi} \]

An approximation of the Hognestad Parabola will be used in determining the equivalent compression stress-block from Eq. (6a)

\[ \sigma_c = \alpha * f'_c \]

where Eq. (7a)

\[ \alpha = \frac{\frac{\varepsilon_c}{\varepsilon'_c} - \left( \frac{1}{3} \right) * \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)^2}{\beta} \]

where from Eq. (8a)

\[ \beta = \left( 4 - \frac{\varepsilon_c}{\varepsilon'_c} \right) / \left( 6 - \left( \frac{2 \varepsilon_c}{\varepsilon'_c} \right) \right) \]

where

\[ \varepsilon'_c = 0.002 \]

therefore from Eq. (8a)

\[ \beta = \left( 4 - \frac{0.0015}{0.002} \right) / \left[ 6 - \left( \frac{2 \times 0.0015}{0.002} \right) \right] \]
\[ \beta = 0.722 \]

therefore from Eq. (7a)

\[ \alpha = \frac{0.0015 / 0.002 - \left( \frac{1}{3} \right) * \left( \frac{0.0015}{0.002} \right)^2}{0.722} \]
\[ \alpha = 0.779 \]

therefore from Eq. (6a)

\[ \sigma_c = 0.779 * 7600 \text{ psi} \]
\[ \sigma_c = 5921 \text{ psi} \]
→ Determine Forces developed in the FRP, tension steel, and concrete from Eq. (9a, 10a, 11a)

\[
F_f = \sigma_f * A_f
\]

\[
F_f = 103200 \text{ psi} * (n * t_f * b)
\]

\[
F_f = 103200 \text{ psi} * (1 * 0.02 \text{ in} * 24 \text{ in})
\]

\[
F_f = 49536 \text{ lbs}
\]

\[
F_s = \sigma_s * A_s
\]

\[
F_s = 65770 \text{ psi} * (n_s * \left( \frac{\pi * \text{diameter}^2}{4} \right))
\]

\[
F_s = 65770 \text{ psi} * (5 * \frac{\pi * (0.5 \text{ in})^2}{4})
\]

\[
F_s = 64454 \text{ lbs}
\]

An approximation of the Hognestad Parabola will be used in determining the equivalent compression force in the concrete

\[
F_c = \sigma_c * A_c
\]

\[
F_c = \sigma_c * (\beta c * b)
\]

\[
F_c = 5921 \text{ psi} * (0.722 * 1.0 \text{ in} * 24 \text{ in})
\]

\[
F_c = 102599 \text{ lbs}
\]

→ Check for Force Equilibrium from Eq. (12a)

\[
T = C
\]

\[
T_f + T_s = C_c
\]

\[
49536 \text{ lb} + 64454 \text{ lb} = 102599 \text{ lb}
\]

\[
113990 \text{ lb} \neq 102599 \text{ lb}
\]

Because equilibrium was not met with this assumed value for c, a new
value of c should be selected and the process iterated until a force equilibrium is met

→ Through iteration

\[ c = 1.07 \text{ in} \]

\[ T_f + T_s = C_c \]

\[ 49536 \text{ lb} + 64454 \text{ lb} = 115404 \text{ lb} \]

\[ 113990 \text{ lb} \approx 115404 \text{ lb} \]

Equilibrium condition is satisfied

→ Calculate moment capacity of the strengthened slab from Eq. (13a)

\[ M_u = M_c + M_f + M_s \]

Where from Eq. (14a)

\[ M_c = F_c \ast \left( c - \frac{\beta c}{2} \right) \]

\[ M_c = 115404 \text{ lb} \ast \left( 1.07 \text{ in} - \frac{0.5913 \ast 1.07 \text{ in}}{2} \right) \]

\[ M_c = 86975 \text{ in} - \text{lb} \]

Where from Eq. (15a)

\[ M_s = F_s \ast (d_s - c) \]

\[ M_s = 64454 \text{ lb} \ast (5 \text{ in} - 1.07 \text{ in}) \]

\[ M_s = 253304 \text{ in} - \text{lb} \]

where from Eq. (16a)

\[ M_f = F_f \ast (d_f - c) \]

\[ M_f = 49536 \text{ lb} \ast (6.01 \text{ in} - 1.07 \text{ in}) \]

\[ M_f = 244708 \text{ in} - \text{lb} \]

Therefore from Eq. (13a)

\[ M_u = 86975 + 253304 + 244708 \text{ in} - \text{lb} \]
Calculate the ultimate load carrying capacity of the strengthened slab from Eq. (17a)

\[
P_u = 2\left(\frac{M_u}{\left(\frac{1}{2}\right) \times (\text{clear span} \ - \ \text{distance between load points})}\right)
\]

\[
P_u = 2 \times \left(\frac{48.75 \text{ k} - \text{ft}}{\left(\frac{1}{2}\right) \times (12 \text{ ft} - 2 \text{ ft})}\right)
\]

\[
P_u = 2 \times \frac{48.75 \text{ k} - \text{ft}}{5 \text{ ft}}
\]

\[
P_u = 19.50 \text{ k} - \text{ft}
\]