ABSTRACT

FARSHCHI TABRIZI, SALAR. Strengthening of Steel Structures with Carbon Fiber Reinforced Polymer (CFRP). (Under the direction of Dr. Sami Rizkalla).

There is a growing demand recently to strengthen steel structures and bridges due to increasing traffic loads and possible reduction of strength due to corrosion of the steel under the effect of environmental conditions. Carbon Fiber Reinforced Polymer (CFRP) materials are currently explored as a possible strengthening technique due to their high tensile strength, light weight, corrosion resistance and cost effectiveness. Conventional repair methods can be time consuming, costly and difficult to install. On the other hand, CFRP materials can be installed easily during traffic hours due to their light weight. Most of the reported research on CFRP strengthening is for concrete structures and bridges; however, relatively less research addresses strengthening of steel structures. This thesis presents an experimental program that was conducted to examine the behavior of steel-concrete composite beams strengthened with a new CFRP product under different types of loading. The focus of this research is exploring the effectiveness of small diameter CFRP strands on strengthening steel bridge girders. Design recommendations are presented on the use of this material to increase the strength and enhance the serviceability of steel bridges.
Strengthening of Steel Structures with Carbon Fiber Reinforced Polymer (CFRP)

by
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DEDICATION

I dedicate this thesis to my parents for their endless love and support.
BIOGRAPHY

Salar Farhchi Tabrizi was born and raised in Tabriz, Iran. He finished his high school at National Organization for Development of Special Talents (NODET), Tabriz, Iran. He started his Bachelor of Science program at Sharif University of Technology (SUT) at 2005 and he was ranked among the top 0.1% of five hundred thousand students who participated in university entrance exam. Upon completion of his Bachelors, he has been accepted to the Civil, Construction and Environmental Engineering graduate program at North Carolina State University under the direction of Dr. Sami Rizkalla. While enrolled in the graduate program, his research focused on strengthening of steel structures using CFRP materials. After graduation, he is planning to join a consulting engineering company as a structure design engineer.
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1. INTRODUCTION

1.1 BACKGROUND

Several researches have been reported recently on strengthening of steel structures using variety of methods. The goal of these researches was to explore the most efficient and effective strengthening system that can be used for steel structural members. Corrosion of steel structural members due to environmental conditions and increase in live loads are the main reasons for requirement of strengthening systems. Conventional strengthening systems such as using steel bars and plates as a reinforcement for steel members are common in current practice but these methods have several disadvantages including the heavy weight of steel plates which makes them hard to handle in addition to their low corrosion resistance. Recently, few researches explored the use of FRP materials for strengthening steel structures. FRP strengthening systems are light, easy to handle and have a relatively less tendency to corrode. Some of the researches have focused on the key parameters that could affect the behavior of FRP strengthening systems for steel structural members. At North Carolina State University, two different strengthening systems were developed to study the effect of externally bonded CFRP strands on the stiffness and the ultimate strength of the steel structural members. This thesis explores a new type of carbon fiber reinforced polymer material for strengthening steel structures. The new material deals with the same problem related to the use of FRP strengthening system for steel structures.
1.2 OBJECTIVES

The main objective of this research is to investigate the use of a new small diameter CFRP product for repair and strengthening of steel structures. The research investigated various parameters that are believed to influence the effectiveness of the proposed CFRP strengthening system. This goal was achieved by testing scaled typical steel-concrete composite bridge girders. The parameters examined in this research were type of CFRP strands, reinforcement ratio, strengthening configuration and type of loading. Seven Concrete-Steel composite beams were strengthened with CFRP strengthening systems to study the effect of these parameters. Two different types of CFRP strands, investigated through this experimental program, are high strength and intermediate modulus strands. Three types of loading used to examine the effect of the strengthening systems, are static, cyclic and fatigue loading. Specifically, the main objectives for this program are summarized as follows:

1. Evaluate the effectiveness of the strengthening system under different limit states and identify the failure modes.

2. Study the effectiveness of high tensile CFRP product versus the intermediate modulus CFRP.

3. Examine the behavior of the proposed system under cyclic and fatigue loading conditions.

4. Examine some of the detailed issues such as the possible shear-lag effect between the CFRP strands and steel beam.
1.3 SCOPE

The experimental program consisted of three phases. First phase of the experimental program included five scaled steel-concrete composite beams strengthened with two types of new CFRP product tested under static loading to failure. Based on the results acquired from the first phase, the best strengthening system was selected to be tested in the second and the third phases. Two beams were tested under cyclic and fatigue loading in the second and the third phases respectively to study the behavior under simulated traffic loading during the service life of the strengthened structure. The behavior of the strengthened beams was predicted using an analytical model. The moment-curvature curve for the section was modeled using layered sectional analysis. As a result, load-deflection behavior of the strengthened beam was obtained using the predicted moment-curvature curve. Results obtained from testing of strengthened beams were compared to the control specimen in order to determine the increase in ultimate load capacity and stiffness.

The second chapter will provide background information on the strengthening of steel structures using FRP strengthening systems. It will also provide a review of related researches that have been done so far in this field.

The third chapter in this thesis will describe the experimental program, the construction of test specimens, the test setup and the instrumentation. The material properties of concrete,
steel beam and CFRP material, used in this experimental program, will also be provided in the third chapter.

The fourth chapter will examine the experimental results and will discuss the findings made out of this experimental program. The effect of each parameter on the performance of strengthening systems will also be included in this chapter. It will also include the analytical model that was used to predict the behavior of the beam strengthened with the proposed strengthening system.

Finally, the fifth chapter will present the conclusions that were drawn from the research program. Future research recommendations will also be presented.
2. LITERATURE REVIEW

2.1 INTRODUCTION

A large number of steel structures including buildings and bridges are in need of retrofitting. Strengthening of steel structures and bridges using externally attached steel plates or post tensioning are examples of conventional methods for retrofitting steel structures. These methods are considered to be inefficient due to the heavy weight of steel which increases the dead load of the structure. High costs, labor intensity, service interruptions and the difficulty in accessing the top of the tension flange of the steel members add to the disadvantages of using conventional methods. Fiber Reinforced Polymer provides an excellent alternative material for strengthening of concrete structures and bridges due to its superior mechanical and physical properties such as its high strength to weight ratio and high corrosion resistance. Use of FRP to strengthen metallic structures was initiated by aerospace and automotive industry then it was also used in marine structures. Recently, several researches have begun to explore the use of FRP for strengthening steel structures and bridges. Until today, the use of FRP materials for retrofitting of steel bridges is not widely accepted due to the limited reported research and lack of field experience.

Some of the researchers focused on using relatively low modulus FRP to retrofit steel beams (Tavakkolizadeh & Saadatmanesh, 2003). These researches showed that significant increase in ultimate capacity can be provided using CFRP with a modulus of elasticity equal to two-thirds of steel. However, their findings reported that adding more layers of carbon fiber can
lead to ineffective utilization of the CFRP materials. Some other researchers focused on using externally bonded high elastic modulus carbon fibers to strengthen steel beams (Dawood, 2005). Research findings indicate that the use of high modulus fibers significantly increased both the ultimate capacity and the stiffness of the steel beams (Schnerch, 2005). While the research in the field of strengthening steel structures and bridge girders is gaining more acceptance, some researchers directed their focus also to strengthening of other steel members such as columns (Shaat et al, 2006). J.G. Teng (2012) and Schnerch (2005) recorded the details of all the documented research in comprehensive state of art reviews in this field.

This chapter is a summary of selected major research findings related to the research reported in this thesis.

2.2 SURFACE PREPARATION

One of the major issues that control the effectiveness of strengthening for steel structures using FRP materials is proper preparation of the substrate surface to prevent premature delamination of the FRP materials. Debonding of the FRP material is highly dependent on proper surface preparation and quality of the adhesive to match the FRP material. Debonding could also be caused by stress concentrations due to lack of steps in multilayer strengthening systems. Proper surface preparation is discussed in this section as one of the main factors influencing the bond behavior. Premature failure due to debonding can be avoided by proper
treating the surface and selecting the appropriate strengthening materials (Linghoff, 2009; Schnerch, 2005).

Steel substrate needs to be treated prior to strengthening in order to ensure a good bond between FRP material and steel. The purpose of surface treatment is to expose the steel and remove weak surfaces such as paint, rust chemicals and mill scale. The two effective steps of surface treatment are reported as the mechanical abrasion and using solvent wiping (Schnerch, 2005). Abrasion can be achieved by wire brushing, abrasion paper, sand blasting and grit blasting. One of the most efficient abrasion methods that has been recommended to avoid adhesion failure is the use of grit blasting (Teng, 2012, Tavakkolizadeh and Saadatmanesh, 2003). It has been proved that even the size of grit can affect the bond behavior. Dawood (2005) included in his research that solvent wiping should be done immediately after abrasion. In order to avoid the recontamination of the steel surfaces, solvent chemical should be used in excess to drip off from the steel surface (Schnerch, 2005). Strengthening process should be conducted immediately after solvent wiping to minimize the recontamination and rusting in the still surface (Dawood, 2005).

Fibers should be cut to the desired length and prepared prior to strengthening (Schnerch, 2006). It has been indicated that tapering the ends of FRP plates can considerably improve the performance of the bond. Peel ply is preferred for FRP strips but in case of absence of
peel ply, the bonding surface of the strips should be abraded using sandpaper and cleaned with a solvent (Schnerch, 2006).

2.3 **DOUBLE STRAP JOINTS**

In order to have a better understanding of the bond behavior, a great deal of research related to strengthening steel structures has been devoted to double strap joints. Number of CFRP layers, adhesive properties, using steps between each layer, rate of loading, tapering and temperature are some of the parameters that have been studied by researchers in double strap joint tests.

Adhesive properties are major factors that can significantly affect debonding. Debonding is not desirable due to low utilization of strengthening materials prior to failure. Research has shown that using adhesives with higher strain capacity results in a better bond performance versus adhesives with a similar or even higher tensile strength (J.G. Teng, 2012). Bond performance is also influenced by adhesive thickness. It’s recommended that adhesive thickness should be minimized to be no more than 0.08” (2 mm) (Xia, 2005; Schnerch, 2005). Based on Yu’s (2011) parametric study, bond strength can be increased using thinner layer of adhesive. Desired adhesive thickness can be obtained by using non-metallic bond line spacer beads inside the adhesive mix and spreading the mixture using shaped trowels (Schnerch, 2005). Based on the results from an experimental program done by Chao Wu
(2012), it was found that ductile adhesives have a higher ability to distribute the force across a larger bond area resulting in higher joint strength and improvement in failure mode.

It’s crucial to use steps for multi-layer strengthening systems to ensure a good bond. It was found that the risk of debonding can be reduced even if the step lamination was applied only to the first layer (Hidekuma, 2012). While normal tapering can be helpful in the strengthening systems with softer laminates and stiffer adhesives, reverse tapering is more effective in stress reduction leading to an increased joint strength (Haghani, 2010). Fibers should be clamped to steel surface during the curing time to ensure a good spread of adhesive (Schnerch, 2005). Reducing the laminate thickness and increasing the bond length can improve the quality of the bond (Yu, 2011).

Temperature is another factor that could influence the behavior of the bond between CFRP and steel. It’s found that higher temperatures could cause larger air pockets in the resin matrix, resulting in reduction of strength (Liu, 2013). It was also proved that increasing the temperature results in an increase in the development length of CFRP materials (Liu, 2013).

2.4 RETROFIT OF STEEL BEAMS WITH CFRP MATERIALS

Significant number of research has been reported recently on strengthening steel beams using CFRP materials. Various parameters and different initial conditions have been taken into consideration by several researchers. In this section, the research on strengthening of steel
beams using CFRP materials is classified into three different categories. Primarily some researches focused on strengthening steel beams that were deteriorated naturally under real service life conditions, which is more common in field applications, while some other researchers investigated strengthening of steel members that were damaged artificially by cutting a portion of the flange. Research has also been conducted on strengthening of undamaged plain steel beams to study the effectiveness of CFRP strengthening systems. Additionally, some researchers investigated the behavior of steel-concrete composite beams strengthened with FRP materials which represents typical bridge girder application.

2.4.1. Rehabilitation of deteriorated steel beams

Recently, many bridges are in-need for rehabilitation due to their continuous loss of strength due to corrosion and many other factors such as physical impact, creep due to the effect of fatigue loading and other factors that can lead to loss of cross section. The main purpose of strengthening of these bridges is to increase their ultimate capacity and in some cases to restore their stiffness. There are available reports on field applications of repairing naturally deteriorated steel bridges such as Takiguchi Bridge in Japan and the Acton railroad bridge, UK. A bridge located on Interstate 95 was strengthened using low elastic modulus CFRP material with an elastic modulus approximately one-half of the steel’s modulus. A series of tests that were done by Miller (2001) verified the soundness of the strengthening system. Four full scale naturally corroded beams taken from a bridge in Schuylkill County, PA, have been strengthened. It was shown that attaching one layer of a low modulus CFRP to the
tension flange has increased the elastic stiffness by 10% to 37% and the ultimate capacity by 17% to 25%.

Different approaches were used by other researchers to study the effectiveness of the CFRP strengthening systems for steel structures. In some studies, some parts of steel beam were cut and notched intentionally in order to simulate the corrosion damage at steel members. Normal modulus CFRP has been used to strengthen a series of beams, cut to simulate the corrosion (Liu, 2001). Results showed an increase in the stiffness and the ultimate capacity of the damaged beam. The reported failure mode for this type of beams was the peeling of the material due to stress concentration at the edge of the notch. The experimental program conducted by Sen (2001) included a damage that was simulated by loading unreinforced steel beams beyond the yield strength before the strengthening. Based on the results from this research program, the use of bolting mechanism for thicker CFRP plates for a proper transfer of the shear stresses is recommended. Higher modulus CFRP is more utilized compared to normal modulus CFRP material (Amr Shaat, 2008). In an experimental program conducted by Kim (2011), damage was simulated by creating three different notches at mid-span of the beams to study the effect of the CFRP strengthening system. The study concluded that the failure mode is independent of the level of the initial damage created at the mid-span and also the use of CFRP strengthening system delays the crack formation of the repaired beams. K. Galal (2012) has recommended a mechanical anchorage system to prevent premature failure.
due to peeling of the CFRP material and to have a ductile behavior by controlling the failure to be through the yielding of the linked members.

2.4.2. Retrofit of plain steel beams

Ten small-scale beams were tested under three and four point loadings by Jung Deng (2005) to study the effect of various parameters on behavior of beams strengthened with CFRP strengthening systems. The beams were 4 feet long mild steel 127x67 UB13 with 30000 ksi elastic modulus. CFRP plates used for strengthening were 1/2 in and 1/4 in thick with an elastic modulus of 31000 ksi. The research program showed that the ultimate strength of the beams increases with the length of the CFRP plate whereas; it tends to reduce by using thicker CFRP plate. D. Linghoof (2010) conducted a research program on performance of steel beams strengthened with CFRP laminates. The research program included testing of five steel beams under four point bending load. Parameters that used in this study were CFRP material and type of epoxy for different strengthening configurations. The elastic moduli of the three CFRP materials used for strengthening were 24 ksi, 29 ksi and 48 ksi respectively. Test results indicate that up to 20 percent increase was achieved using these types of materials for strengthening. Results from this program showed that using a CFRP with equivalent stiffness of the steel produces the most desirable behavior in the strengthened beam and also an increase in the stiffness can be achieved by using CFRP with higher elastic modulus but it can result in early failure and brittle behavior in the strengthened section.
2.4.3. Retrofit of composite steel-concrete beams

In a research program conducted by A.H. Saidy (2005), three steel-concrete composite were strengthened using two types of normal modulus CFRP. Two of the mentioned beams were strengthened only at their tension flange while one beam was strengthened at its flange and the web. All of the beams were W 8 x 15 steel beams in composite action with 3” x 33” concrete slabs. The beams were simply supported with a span of 10 feet. Test results indicate that a 45 percent increase in the ultimate flexural capacity can be achieved using these materials for strengthening. Stiffness increase was also higher in the post yield range leading to 25 percent reduction in the deflection for the strengthened beams compared to the control beams at failure.

The state of art review on strengthening steel-concrete composite beams by N. Ragab (2007) reported that CFRP strengthening systems for steel structures are effective on strengthening and enhancing the serviceability due to increase of the stiffness. Non-prestressed CFRP can’t carry dead load whereas prestressed CFRP carries both portions of dead and live load and improves the serviceability. Using prestressed CFRP results in more utilization of fibers and increase of the yield load compared to non-prestressed CFRP.

D. Scnherch (2008) conducted an experimental program to examine the behavior of full-scale steel-concrete composite beams strengthened with high modulus CFRP material. Effects of pre-stressing of CFRP material, CFRP modulus and different splice length of
CFRP strips were studied in this program. Two types of CFRP with elastic modules of 33 ksi and 66 ksi were attached to full scale steel-concrete composite beams and tested under four point bending configuration. The Beams consisted of a W 12 x 30 steel beam and 4 in x 33 in concrete slab. Test results indicated an increase in the stiffness of 10 to 34 percent and an increase up to 46 percent in the ultimate moment capacity achieved using high modulus CFRP for strengthening. The research also indicated that use of pre-stressed CFRP strips provides a significant stiffness increase while maintaining the ductility of the original section.

2.5 FATIGUE BEHAVIOR OF CFRP-STRENGTHENED STEEL BEAMS

The remaining service life of steel structures is one of the most important issues for the existing steel structures and bridges. The remaining service life of these structures is strongly affected by the extent of fatigue damage. Fatigue damage in steel structures is mostly induced by presence of fatigue cracks and their gradual expansion. Two methods that are mostly used for rehabilitation are the conventional methods and CFRP strengthening systems. Although the use of conventional methods such as bolting and welding steel plates improves the fatigue performance of steel structures, it has its own disadvantages such as corrosion of steel plates, increase of dead loads, labor intensity of application and recreation of fatigue cracks at bolting spots. On the other hand, CFRP strengthening systems proved to be highly corrosion resistant and easy to apply in addition to their enhancement of fatigue performance of steel structures. Research in fatigue performance of steel structures
strengthened with CFRP mainly falls into two groups; research on fatigue behavior of steel plates strengthened with CFRP materials in tension and research on fatigue performance of steel beams strengthened with CFRP materials in bending.

2.5.1. Fatigue performance of single & double strap joints

In an experimental study performed by A. Monfared (2008), 15 notched steel plates were tested under fatigue loading to investigate the effectiveness of high modulus CFRP in fatigue performance of steel structures. 4 out of 15 specimens were unreinforced specimens that were tested with different stress ranges as reference specimens. The parameters included in this program were strengthening type, surface preparation and stress range. The fatigue life of the specimens reinforced on one side increased by 79% to 119% whereas applying CFRP on both sides of the specimens did not further improve the fatigue life in comparison to specimens strengthened on one side. It was also concluded that surface preparation and properties of high modulus CFRP system are the two critical factors that could improve the fatigue performance of steel structures.

Hongbu Liu (2009) conducted an experimental program to study the fatigue crack growth behavior of CFRP strengthened steel structures. A total of 21 specimens including 11 double strap joints and 10 single strap joints strengthened with two type of CFRP were tested under fatigue loading. The purpose of these series of tests was to study the effectiveness of CFRP strengthening systems on preventing fatigue crack propagation and consequently extending
the remaining service life of the strengthened joint. Based on the results, it was found that CFRP strengthening system successfully reduced the crack growth rate and extended the service life of steel members. Double strap joints strengthened with normal modulus CFRP had 2.2 to 2.7 time longer service life compared to unstrengthened specimen, whereas double strap joints strengthened with high modulus CFRP had 4.7 to 7.9 times longer service life than unstrengthened specimen. It was found that high modulus CFRP is more effective in enhancing the fatigue performance of the steel plates. Test results also showed that optimizing the parameters such as patch thickness, patch length and patch configuration can extend the fatigue life of both double and single strap joints up to 20 percent.

In another research program conducted by Ye Huawen (2010), the effect of other parameters on crack growth such as stress range, CFRP stiffness, and pre-stressing levels, was investigated. 14 double edge notched specimens were tested under fatigue cyclic loading. Research findings indicated that the use of prestressed CFRP could prolong the fatigue life significantly because of the cyclic stress and stress ratio reduction in comparison to non-pre-stressed CFRP. The pre-stressing level and the applied stress range are the two most critical parameters that were found to influence the fatigue behavior. Results showed that the CFRP with the highest pre-stressing level had the best fatigue performance and increased the fatigue life by approximately four times.
2.5.2. Fatigue performance of steel beams strengthened with CFRP

Gang Wu (2012) studied the fatigue behavior of artificially notched beams that were strengthened using high modulus CFRP, high strength CFRP, steel-wire basalt FRP and welded steel plate. In addition to types of strengthening materials; strengthening configuration, number of CFRP layers and interface treatment were the other parameters investigated in this research program. Seven of eight artificially damaged beams were strengthened using these four types of material and the remaining damaged steel beam was used as reference beam. Results showed that high modulus CFRP has the best strengthening effect on fatigue performance of the beams. The fatigue life of the beams strengthened with FRP materials was 3 to 5 times the life of the unstrengthened beam whereas the fatigue life of the steel beam strengthened with a welded steel plate was only 1.74 times that of the unstrengthened beam. Use of FRP materials resulted in reduction of crack growing rate, delay in crack initiation of the steel beam and improvement of the failure mode. Reduction of residual stresses and reduction of stiffness decay compared to welded steel plate were the other advantages of using FRP for strengthening.

In a comparative study conducted by Hui Jiao (2012) on fatigue behavior of steel beams, 14 beams were strengthened with pultruded CFRP plates and wet layup CFRP sheets and 7 beams were strengthened by welding. All of the beams had an initial cut to simulate damage. Two types of CFRP and two types of epoxies were used for strengthening purposes. The fatigue life of the beams strengthened with CFRP was significantly longer than that of the
beams strengthened with welding. It was found that one layer of CFRP plate extended the fatigue life 7 times while using 4 layers of CFRP sheets extended the fatigue life 3 times compared to the specimens that were only strengthened with welding.

A research on fatigue behavior of notched beams reinforced with bonded CFRP was conducted by E. Ghafoori (2012). This study included the fatigue behavior of notched steel beams strengthened with non-prestressed and prestressed CFRP plates. Five beams were tested including one control specimen, one beam strengthened with non-prestressed CFRP and three beams strengthened with prestressed CFRP plates. Experimental results showed that the fatigue life of beams strengthened with prestressed CFRP was five times more than fatigue life of the beams strengthened with non-prestressed CFRP. In addition, it was indicated that pre-stressing significantly reduces the residual deflection during fatigue crack growth process. Furthermore, it was shown that the fatigue crack growth rate increases as the stiffness of the CFRP to steel bond decreases.

2.6 FLEXURAL ANALYSIS OF STEEL BEAMS STRENGTHENED WITH FRP

Recently, several researches have reported detailed analysis to predict the full range flexural behavior of steel beams strengthened with FRP materials. Some researchers have focused on evaluation of flexural behavior using moment-curvature analysis and assuming a linear behavior of materials whereas some others have made their predictions using the non-linearity of structural materials. Shear-lag effect at FRP-steel interface has been taken into
consideration in some researches while some researches have been based on the assumption that the strain profile remains linear. Additionally, some numerical studies including finite element analysis has been conducted by some researchers to evaluate the flexural behavior of FRP-strengthened steel structures.

Massimiliano Bocciarelli (2009) introduced a simple approach in his study towards the evaluation of flexural response of the CFRP-strengthened steel structures. His assumptions included linear-elastic behavior of CFRP, perfectly elastic behavior for steel, no slippage between CFRP and steel, negligible CFRP bending stiffness and uniformity of CFRP axial stress along its thickness. Based on his study, the solution is valid only at a certain distance from the reinforcement ends. His approach comprised simply supported beams loaded with distributed and concentrated load. It was concluded that bearing capacity of a reinforced beam must be evaluated taking into account the failure mechanisms. Finally, he concluded that the capacity is highly affected by CFRP reinforcement ratio, reinforcement strength, and CFRP elastic modulus, interface strength, adhesive properties and loading conditions.

The analytical approach presented by Carlo Pellegrino (2009) evaluated the flexural response of FRP-strengthened steel and steel concrete composite structures using non-linear material characteristics based on cross sectional equilibrium and strain compatibility. His approach is an extension of the classic equilibrium and strain compatibility for the post-elastic range
assuming a non-linear behavior of the materials. The approach is applicable to any geometric configuration of FRP strengthening.

Recently, Finite element modeling has been adopted successfully by Mohammad Z. Kabir (2010) to predict the load carrying capacity of steel-concrete composite girders. The results from the model were compared to published test results to examine the validity of the model. Finally, a detailed parametric study was performed on the effects of geometry and material characteristics on flexural performance of composite section. Linear strain variation over depth of cross section, no slippage between FRP and steel and elastic linearity of CFRP were the main assumptions for this finite element study.

Ehab Ellobody (2011) developed nonlinear 3-D finite element models to investigate the flexural behavior and load carrying capacity of CFRP strengthened girders. The nonlinear material properties were incorporated in his finite element modeling. The finite element model has been validated using published experimental results. Using this model load-vertical displacement behavior and failure modes were predicted. A parametric study was conducted on flexural performance of composite girders strengthened with CFRP. The study showed that the increase in concrete strength results in a considerable increase in initial stiffness, whereas increase in structural steel strength offers a significant increase in the post-yielding stiffness.
3. **EXPERIMENTAL PROGRAM**

This chapter describes an experimental program conducted at the Constructed Facilities Laboratory at North Carolina State University to investigate the effectiveness of carbon fiber reinforced polymer materials for strengthening steel structures. This chapter includes the material properties used in the system, fabrication of test specimens, test setup, instrumentation and the standards used to determine the properties of the materials. Additionally, the process of strengthening, application of adhesive and CFRP materials are also presented in this chapter. This chapter describes the three different phases of testing including testing under static, cyclic and fatigue loading.

3.1 **MATERIAL PROPERTIES**

3.1.1. **Concrete Properties**

All of the slabs were casted using normal Portland cement concrete for this experimental program. The concrete was provided by a local ready mix concrete company. Since the experimental program had three different phases of testing, two batches were used at two different dates. The target normal strength for concrete was 6500 psi with a 4” slump and 3/8” maximum aggregate size. Several 4” x 8” cylinders were made from both batches in order to measure the compressive strength of concrete at the time of testing the beams. The cylinders were placed next to the specimens to be subjected to the same curing conditions. For each slab three cylinders were made and tested according to ASTM C39/C39M. The elastic modulus of concrete cylinders was determined according to ASTM C469/C469. A
A typical test setup for measuring the elastic modulus of concrete is presented in Figure 3-1.

The averaged results from compressive tests and elastic modulus tests made on cylinders are provided in Table 3-1.

Table 3-1: Concrete Properties

<table>
<thead>
<tr>
<th>Cast ID</th>
<th>Compressive Strength (f'c)</th>
<th>Elastic Modulus (E)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6480 psi</td>
<td>4610 ksi</td>
</tr>
<tr>
<td>2</td>
<td>5730 psi</td>
<td>4370 ksi</td>
</tr>
</tbody>
</table>
3.1.2. Steel Properties

Steel beams used in this test were provided by a local steel supply company. In order to evaluate the properties of steel, a piece was cut from the web portion of an extra undamaged steel beam. Four steel coupons were made out of this piece according to ASTM A370 then the four coupons were tested using an MTS machine. Figure 3-2 represents the steel coupon tensile test setup. Strain was measured using an extensometer and a strain gauge in order to determine the stress-strain diagram of steel. The stress-strain diagram is plotted using an average of four coupon tests as shown in Figure 3-3.

Figure 3-2: Steel Coupon Tensile Test Setup
3.1.3. CFRP Properties

Two set of tests were done in order to evaluate the properties of CFRP materials. First set of tests was designed to evaluate the elastic modulus, ultimate strain and ultimate stress whereas second set of tests was used to determine the bond length of the new CFRP material.

3.1.3.1. CFRP Coupon Tests

The CFRP material used for strengthening purposes were provided by Nippon Steel and Sumikin Materials Company, Japan. Two types of CFRP used for strengthening are High Tensile (HT) CFRP and Intermediate Modulus (IM) CFRP. The CFRP materials were
produced as strand sheets as shown in Figure 3-4. The commercial name of the material is FORCA strands. Several tensile coupons were made from the strand sheets and were tested to evaluate the properties of CFRP materials as shown in Figure 3-5. Cross sectional area of strands was measured using a digital micrometer. Table 3-2 presents the tensile properties of CFRP materials.

Table 3-2: CFRP Properties

<table>
<thead>
<tr>
<th>CFRP Type</th>
<th>Strand Area</th>
<th>Ultimate Strain</th>
<th>Load Per Strand</th>
<th>Elastic Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>High Tension</td>
<td>0.0016 in²</td>
<td>0.017 in/in</td>
<td>531 lbs</td>
<td>19522 ksi</td>
</tr>
<tr>
<td>Intermediate Modulus</td>
<td>0.00148 in²</td>
<td>0.0098 in/in</td>
<td>476 lbs</td>
<td>32820 ksi</td>
</tr>
</tbody>
</table>

Figure 3-4: CFRP Strand Sheet

Figure 3-5: CFRP Coupon Test Setup
3.1.3.2. Double Strap Joint Tests

In order to evaluate the development length of the high tensile CFRP material, six double strap joints strengthened with one layer of the high tensile CFRP strands on each side were tested in tension using an MTS machine. The \( \frac{1}{2} \)" x 1 \( \frac{1}{2} \)" steel plates were initially connected to each other using one layer of duct tape. The strengthening process began by bonding one layer of the high tensile CFRP strands to one side of each specimen. The specimens were then cured for 24 hours and then they were flipped over and strengthened on the other side. The specimens were cured for a period of 7 days before any testing was conducted on them. A cross section view of a double strap joint is shown in Figure 3-6. The length of the CFRP strands used varied from 3” to 13” and the length of all of the specimens allowed a 6” clearance at both ends for gripping as shown in Figure 3-7. Table 3-3 includes the strengthening details, the failure modes and the test results for all of the specimens. Test results indicated that the specimens with a shorter strengthened length had a different failure mode on each side. It should be noted that for specimens 2, 3 and 4 debonding occurred for one side at first therefore the total load was carried by one side only which failed due to rupture of the CFRP strands. It should be noted that the bottom side of each specimen was cured under the weight of the specimen itself while the top surface was cured without any pressure. This may have also affected the behavior of the double strap joints. Results suggest that clamping can significantly enhance the performance of the bond. The bottom and the top surfaces of specimen 3 after failure are shown in Figure 3-8 & Figure 3-9. The CFRP strands of the top surface were debonded followed by rupture of the CFRP strands at the bottom
surface as shown in Figure 3-8 & Figure 3-9. The ultimate load versus the strengthened length relationships of the double strap joints are shown in Figure 3-10. Test results suggest that the development length of the high tensile CFRP strands is approximately of 11”. Due to limited number of the test specimens, twelve double strap joints will be tested to provide statistical data for the development length tests.

Figure 3-6: Cross Section View of a Double Strap Joint

Figure 3-7: Elevation View of a Double Strap Joint
Table 3-3: *Strengthening Details and Test Results of the Double Strap Joints*

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Failure Mode</th>
<th>L_d</th>
<th>Ultimate Load (Kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Side 1</td>
<td>Side 2</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Debonding</td>
<td>Debonding</td>
<td>3.1”</td>
</tr>
<tr>
<td>2</td>
<td>Rupture</td>
<td>Debonding</td>
<td>5.1”</td>
</tr>
<tr>
<td>3</td>
<td>Rupture</td>
<td>Debonding</td>
<td>7.1”</td>
</tr>
<tr>
<td>4</td>
<td>Rupture</td>
<td>Debonding</td>
<td>9.1”</td>
</tr>
<tr>
<td>5</td>
<td>Rupture</td>
<td>Rupture</td>
<td>11.1”</td>
</tr>
<tr>
<td>6</td>
<td>Rupture</td>
<td>Rupture</td>
<td>13.1”</td>
</tr>
</tbody>
</table>

*Figure 3-8: Side 1 of Specimen 3, Rupture of the CFRP strands*

*Figure 3-9: Side 2 of Specimen 3, Debonding of the CFRP Strands*
3.2 TEST SPECIMENS

A total of eight specimens were tested under three types of loading in this research program. All of the steel-concrete beams had the same properties except for the fatigue test specimen where additionally concrete blocks were added at the both ends of the specimen to provide stability under the fatigue loading. All of specimens consisted of an 11 feet long W 8 x 13 steel beam which was in a composite action with a 2.6” x 21” concrete slab. Figure 3-11 presents the typical cross section of the specimen. The composite action was achieved by 1/2” x 2 ½” Nelson Studs connectors on top of the steel beam flange. The studs were welded
at 4” spacing center to center along the beam and at 2” clearance from both ends of the beams as show in Figure 3-12. Stiffeners ¼” thick were welded at mid-span and supports to prevent possible local buckling.

Figure 3-11: Cross Sectional View of a Test Specimen

Figure 3-12: Elevation View of a Test Specimen
Welded wired mesh was used to reinforce the concrete slab. Fatigue specimen had the same properties and dimensions except with the additional blocks added at the ends to provide stability under fatigue loading. Four 1/2” x 2 ½” studs and two C shaped steel bars were welded on both sides of the web at two ends of the beam to fix the blocks to the beam. Dimensions of the blocks are shown in Figure 3-13.

![Figure 3-13: Fatigue Specimen Cross Sectional View](image)

3.2.1. Fabrication of Test Specimens

The steel beams were cut to the appropriate length. Shear connectors were welded on top of the steel flange using Nelson stud welder equipment as shown in Figure 3-14 & Figure 3-15. In order to ensure that studs were welded properly, several studs were welded on to an extra piece of beam and then were struck smartly until they bent 45° as shown in Figure 3-16.
The construction was continued by welding the stiffeners at mid span and supports to avoid possible local buckling of the steel beam. Two partial depth stiffeners were used at the location of the applied loads at mid span while full depth stiffeners were used at the support locations as illustrated in Figure 3-17.
Wooden Formworks were used to cast the slabs in an upside down position. After placing welded wire mesh, the beams were placed in the formworks as shown in Figure 3-18.
In order to hold the steel beams at the appropriate height from the top of the slab, the beams were supported by small concrete blocks as shown in Figure 3-19. Concrete blocks were left inside the formworks during the cast. Curing compound was applied to the surface of the concrete to ensure a proper curing process. Formworks were removed after 14 days. Figure 3-20 shows the blocks casted at the ends of the fatigue test specimen for more stability.
Figure 3-19: Concrete Block for Holding the Beams

Figure 3-20: Fatigue Test Specimen Supports
3.2.2. **Strengthening Process**

The outer surface of the tension flange for all of the steel beams were sand blasted using a sand blasting equipment as shown in Figure 3-21 & Figure 3-22.

![Figure 3-21: Sand Blasting](image)

![Figure 3-22: Sand Blasting Close Shot](image)

The blasted surfaces were cleaned using compressed air flow followed by acetone to clean the surfaces. Acetone was used in large amounts to drip off from the surfaces to avoid any recontamination of the blasted surfaces.

CFRP strand sheets were cut to the appropriate width and length and the beams were placed on top of concrete blocks to apply overhead strengthening as shown in Figure 3-23. The
overhead strengthening was used to simulate the field applications and to apply the strengthening system while the dead load is acting on the specimens through spanning of the beams. Width of the CFRP strand sheets was same as width of the tension flange which is 4”. Length of the sheets varied for each layer added to the strengthening. First layer of strand sheets was 9 ½’ long with 3” distance from the supports to avoid pinching of the strands. After the first layer, for each layer added, CFRP strand sheet were shortened by 6” from both sides to avoid stress concentrations as shown in Figure 3-24.

Figure 3-23: Overhead Strengthening
After the application of the CFRP strand sheets, wooden boards and C-clamps were used to ensure a good spread of adhesive and a good bond between CFRP strand sheets and steel flange as shown in Figure 3-25. Test specimens were cured for seven days before testing.
Details of the strengthened beams are provided in Table 3-4 including type of the CFRP material, number of CFRP layers used for strengthening of each specimen and the type of loading used for testing each specimen.

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Number of Layers</th>
<th>CFRP Type</th>
<th>Loading Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>S0-C</td>
<td>-</td>
<td>-</td>
<td>Static</td>
</tr>
<tr>
<td>S3-HT</td>
<td>3</td>
<td>High Tensile</td>
<td>Static</td>
</tr>
<tr>
<td>S4-HT</td>
<td>4</td>
<td>High Tensile</td>
<td>Static</td>
</tr>
<tr>
<td>S3-IM</td>
<td>3</td>
<td>Intermediate Modulus</td>
<td>Static</td>
</tr>
<tr>
<td>S4-IM</td>
<td>4</td>
<td>Intermediate Modulus</td>
<td>Static</td>
</tr>
<tr>
<td>S4-HT-IM</td>
<td>3,4</td>
<td>HT &amp; IM</td>
<td>Static</td>
</tr>
<tr>
<td>S4-HT-C</td>
<td>4</td>
<td>High Tensile</td>
<td>Cyclic</td>
</tr>
<tr>
<td>S4-HT-F</td>
<td>4</td>
<td>High Tensile</td>
<td>Fatigue</td>
</tr>
</tbody>
</table>

3.3 INSTRUMENTATION

For all of the specimens tested under static loading except specimen S4-HT-IM, 12 measurements were measured and recorded using three different types of instruments including 4 strain gauges, 6 pi gauges and 2 string potentiometers as indicated in Figure 3-26. Specimen S4-HT-IM was instrumented similarly with two additional strain gauges at bottom flange as shown in Figure 3-27. For fatigue test specimen, 18 measurements were recorded
including 7 strain gauges, 7 pi gauges and 4 string potentiometers as shown in Figure 3-28.

All of test specimens were instrumented at mid span, within the maximum moment region.

![Diagram](image1)

*Figure 3-26: Instrumentation for Overloading Test Specimens*

![Diagram](image2)

*Figure 3-27: Instrumentation for Specimen S4-HT-IM*
3.4 TEST SETUP

The test setup used for seven overloading specimens including the control specimen is shown in Figure 3-30. All of the test specimens consisted of an 11’ long steel-concrete composite beam that were tested under four point bending configuration with a clear span of 10’. All of
the beams were simply supported and were loaded using two point loads at the mid span which were two feet apart. Two 4” x 4” HSS sections were used to apply the loads. HSS bars were supported by two steel dywidag bars which were supported by the strong floor of the lab. Two 120 Kips jacks supported by C-channels were used to apply the load as shown in Figure 3-30. One load cell was placed under one of the jacks to measure the applied load.

![Figure 3-30: Test Setup Schematic for Overloading Specimens](image)

All of the specimens were braced laterally at supports and quarter spans in order to prevent lateral buckling as show in Figure 3-31.
The fatigue test specimen was loaded using a 200 kips actuator. Two 4”x4” HSS bars were attached to the end of the actuator with 2’ distance from each other to act as point loads as shown in Figure 3-32. The supports and the frame used for fatigue testing were stressed to the floor to prevent any possible movements. The specimen was only braced laterally at quarter spans due the presence of concrete blocks that significantly improved the stability of the beam as shown in Figure 3-33.
Figure 3-32: Fatigue Specimen Test Setup

Figure 3-33: Lateral Bracings for Fatigue Test Specimen
3.5 **LOADING PROTOCOL**

All of the specimens were tested under displacement-controlled condition. The loading rate for all of the overloading specimens were 0.05” per minute before yielding and 0.1” per minute after yielding of the steel at bottom flange of the beam. The cyclic loading included ten levels of loading. Each level of loading included three steps of loading and the load was increased by 10 percent at each level. The loading pattern for the cyclic testing is provided at Figure 3-34.

![Figure 3-34: Cyclic Loading](image)
The fatigue test specimen was loaded with a frequency of 1.55 Hz using a 200 kip actuator. The minimum fatigue load was selected to be 10 percent of the ultimate load whereas the maximum fatigue load was selected to be the maximum of the yield load of the unstrengthened beam, $F_{yu}$, or 80 percent of the yield load of the strengthened beam, $F_{ys}$.

\[
F_{max} = \text{Max} \left( F_{yu}, 0.8 \times F_{ys} \right)
\]  

(1)  

Loading pattern for fatigue testing is indicated in Figure 3-35. The specimen was loaded up to two million cycles in fatigue testing then it was loaded statically to failure using same test setup.

![Figure 3-35: Fatigue Loading](image)
4. ANALYSIS, TEST RESULTS AND DISCUSSION

This chapter describes the analysis used to determine the effectiveness of the CFRP materials for strengthening steel structures based on the results of the experimental program described in chapter 3. The assumptions used to predict the behavior, comparison of the behavior of different strengthening systems and criteria used to evaluate the effectiveness of strengthening systems are also presented in this chapter. Detailed discussion of the test results and the process for selecting the best strengthening system for cyclic and fatigue tests are also included in this chapter.

4.1 ANALYSIS

Primarily, moment-curvature analyses were adopted in the analysis of each specimen in order to predict the behavior of the specimens strengthened using CFRP materials. In order to perform moment-curvature analyses, the following assumptions were made:

1. Plain sections remain plane after deformation therefore the strain profile of the section is linear for all loading stages
2. The CFRP material is perfectly elastic material with a linear stress-strain relationship up to failure
3. The steel is elastic-plastic material
4. The CFRP tensile force is assumed to be applied at bottom surface of the flange of the cross section
The stress-strain relation for concrete was modeled using the Hognestad model:

\[ f_c = f'_c \left[ \frac{2\varepsilon_c}{\varepsilon_0} - \left( \frac{\varepsilon_c}{\varepsilon_0} \right)^2 \right] \]  

(2)

Where \( f_c \) is the stress corresponding to strain \( \varepsilon_c \) and \( \varepsilon_0 \) is the strain at the maximum compressive strength of concrete as shown in Figure 4-1.

\[ \text{Figure 4-1: Hognestad Stress-Strain Model for Concrete} \]
The stress-strain relation used for steel is:

\[
 f_s = \begin{cases} 
 \varepsilon_s \varepsilon_s & \varepsilon_s < \varepsilon_y \\
 f_y & \varepsilon_s \geq \varepsilon_y 
\end{cases}
\]  

(3)

Where \( f_s \) is the stress in the steel during the loading, \( \varepsilon_s \) is the corresponding strain and \( \varepsilon_y \) are the yield strain and the yield stress of the steel as show in Figure 4-2.

\[\text{Figure 4-2: Elastic-Plastic Stress-Strain Relationship for Steel}\]
The analysis started by assuming a strain distribution along the cross-section as shown in Figure 4-3 where $\varepsilon_c$ is the compressive strain in the concrete. The steel-concrete composite section was divided into layers consist of small sections as shown in Figure 4-4. The neutral axis depth was determined by satisfying the equilibrium of the tension and the compression forces along the section under consideration. After the iterative process for finding the neutral axis depth, the total moment was determined by summing the moments about the neutral axis for each increment of the strain, $\varepsilon_c$.

![Figure 4-3: Strain Profile for Steel-Concrete Composite Beam](image)

![Figure 4-4: Layered Sectional Analysis](image)
The compression force $C_{Total}$ was determined as follows:

$$C_{Total} = \sum_{1}^{n}(f_c \times A_{cn}) \quad (4)$$

Where $f_c$ is the average compressive stress of the concrete at a given concrete area $A_{cn}$ for each layer along the depth of the concrete with a thickness of $h_{cn}$.

$$f_c = f'_c \left[ \frac{2\varepsilon_{cn}}{\varepsilon_{co}} - \left( \frac{\varepsilon_{cn}}{\varepsilon_{co}} \right)^2 \right] \quad (5)$$

$$A_{cn} = h_{cn} \times width \ of \ the \ Slab \quad (6)$$

The tension force $T_{Total}$ was determined as follows:

$$T_{Total} = \left( \sum_{1}^{n}(f_{sn} \times A_{ns}) \right) + \varepsilon_{cfrp} \times A_{CFRP} \times E_{CFRP} \quad (7)$$

$$f_{sn} = \begin{cases} 
\varepsilon_{sn}E_s \quad \varepsilon_{sn} < \varepsilon_y \\
f_{ys} \quad \varepsilon_{sn} \geq \varepsilon_y 
\end{cases} \quad (8)$$

$$A_{sn} = h_{sn} \times width \ of \ the \ web \ or \ flange \quad (9)$$

For each assumed $\varepsilon_c$, the neutral axis depth $c$ was determined by satisfying the equilibrium:
Based on the location of the neutral axis depth $C$, the curvature was determined as follows:

$\varphi = \frac{\varepsilon_c}{c}$  \hspace{1cm} (11)

The corresponding moment for each strain increment was calculated by summing all of the forces about the neutral axis as shown in Figure 4-5.

$$M_{Total} = \sum_{1}^{n}(C_{cn} \times d_{cn}) + \sum_{1}^{n}(T_{sn} \times d_{sn}) + T_{CFRP} \times (h - c)$$  \hspace{1cm} (12)

A typical moment-curvature relationship for a section was plotted based on the described procedure and used to determine the load-deflection relationship of the beam as shown in
Figure 4-6. The predicted load-deflection relationship for specimen S4-HT shown in dotted line in Figure 4-6 is compared to the measured values shown in solid line.

![Figure 4-6: Predicted Load-Deflection Diagram versus Measured Load-Deflection Diagram](image)

4.2 **TEST RESULTS OF PHASE I**

A total of six specimens were tested under static loading to failure in this phase of the experimental program. One of these six specimens was an unstrengthened beam used as a control specimen. Details of the test specimens tested in this phase are provided in Table 4-1.
The second character of the specimen mark after the letter “S” represents the number of the strengthening layers. The characters after the dash identify the type of the CFRP strengthening material.

Table 4-1: *Phase I Test Specimens Details*

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Number of Layers</th>
<th>CFRP Type</th>
<th>Loading Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>S0-C</td>
<td>-</td>
<td>-</td>
<td>Static</td>
</tr>
<tr>
<td>S3-HT</td>
<td>3</td>
<td>High Tensile</td>
<td>Static</td>
</tr>
<tr>
<td>S4-HT</td>
<td>4</td>
<td>High Tensile</td>
<td>Static</td>
</tr>
<tr>
<td>S3-IM</td>
<td>3</td>
<td>Intermediate Modulus</td>
<td>Static</td>
</tr>
<tr>
<td>S4-IM</td>
<td>4</td>
<td>Intermediate Modulus</td>
<td>Static</td>
</tr>
<tr>
<td>S4-HT-IM</td>
<td>3,4</td>
<td>HT &amp; IM</td>
<td>Static</td>
</tr>
</tbody>
</table>

4.2.1. **Test Results of Specimen S0-C**

The control specimen was loaded using four point bending configuration to failure using hydraulic jacks as described in chapter 3. The measured load-deflection behavior of the control specimen is illustrated in Figure 4-7. The dead load of the steel-concrete composite beam and the weight of the test setup were taken into consideration in load-deflection diagram. The failure was due to crushing of the concrete slab in compression as shown in Figure 4-8.
The measured ultimate load at failure was 48 kips including the dead load and the corresponding deflection was 2.3”. The load-deflection diagram indicates a significant reduction of the stiffness at 30 kips due to yielding of the steel flange in tension. The yielding load was determined when the strain in the steel reached the yielding strain. The flexural stiffness of the composite beam was determined using the following equation within the elastic range:

$$\Delta_{max} = \frac{F}{2a} \left(3l^2 - 4a^2\right) \frac{1}{24EI}$$  \hspace{1cm} (13)
Where $\Delta$ is the mid-span deflection, $L$ is the beam span and $a$ is the distance from support to the nearest point load. The elastic stiffness of the control specimen was calculated as 66.7 kips/in. Test results of the control specimen are summarized in Table 4-2.

![Figure 4-8: Crushing of the Concrete Slab of the Control Specimen](image)

**Table 4-2: Test Results Summary of the Control Specimen**

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Measured Ultimate Load</th>
<th>Deflection at Failure</th>
<th>Flexural Stiffness</th>
</tr>
</thead>
<tbody>
<tr>
<td>S0-C</td>
<td>48 kips</td>
<td>2.3”</td>
<td>66.7 kips/in</td>
</tr>
</tbody>
</table>
4.2.2. Test Results of Specimen S3-HT

Test specimen S3-HT was strengthened with three layers of high tensile CFRP material which has a higher ultimate stress and strain capacity in comparison to the intermediate modulus CFRP strands. The beam loaded to failure under static loading. Failure, which was ductile in nature, occurred due to crushing of the concrete at the top compression zone of the concrete slab as shown in Figure 4-9. The predicted load-deflection behavior versus the measured load-deflection relationship of specimen S3-HT is shown in Figure 4-10. The predicted behavior matches the measured values very well. The results suggest that the analytical approach can be successfully used to predict the behavior of the beams strengthened with high tensile CFRP strengthening systems. The measured load at failure was 70 kips and the corresponding deflection was 2.2”. The results reflect that an increase of 49 percent in the ultimate flexural capacity is achieved using three layers of high tensile CFRP system in comparison to the control specimen. The displacement ductility of specimen S3-HT was calculated as 4.43 in/in. The deflection in the mid-span was measured using an average of the readings of two string potentiometers at mid-span. The ultimate deflection and the displacement ductility of the strengthened specimen was approximately the same as the control specimen. The advantage of using the three layers of high tensile CFRP strengthening system was achieving a 49 percent increase in the ultimate flexural capacity while maintaining the ultimate deflection and the displacement ductility same as the control specimen. The pre-yielding flexural stiffness, determined based on the slope of the linear portion of the load-deflection curve, was 75.45 kips/in. The pre-yielding and the post-
yielding stiffness were increased by 13.2 and 96.1 percent respectively compared to the control specimen using the three layers of high tensile strengthening system. This strengthening system did not have a significant effect on the pre-yielding flexural stiffness whereas it improved the post-yielding stiffness significantly. Results show that the three layers of the high tensile strengthening system can provide an increase in the service loads while it can provide a margin of safety by significantly increasing the ultimate flexural capacity. The measured strains at the top surface of the steel bottom flange and at the bottom surface of the CFRP strands are shown in Figure 4-11. The load-strain behavior of the steel reflects a yield load of 37.5 kips. The yield load was increased by 25 percent compared to the control specimen. The strains of the steel at bottom flange and the CFRP strands are compatible through all loading stages and the small difference in the strains can be attributed to the strain measurement locations which were different by the thickness of the flange. The results indicate that there is no shear lag between the high tensile CFRP strands and the steel bottom flange. This is mainly due the elastic modulus of the high tensile CFRP strands which is relatively low in comparison to the elastic modulus of the steel. The strain of the high tensile CFRP strands at failure was 0.0135 in/in which shows a 79.5 % utilization of the high tensile CFRP strands. The utilization of the CFRP strands was determined using the ratio of the strain of the CFRP strands at failure to their ultimate strain capacity measured from the tensile coupon tests. The load-strain diagram of the top surface of the concrete slab is shown in Figure 4-12. The strain of the top surface of the concrete slab at failure was 0.006 in/in. The strain of the concrete was measured using an average of two pi gauges readings. The
load-strain diagram of the steel top flange is shown Figure 4-13. The change from the compressive strain to the tensile strain in the steel top flange is due to shifting of the neutral axis towards the concrete slab. It was observed that the shifting of the neutral axis started at the yield load of beam. After yielding of the steel at the bottom flange which resulted in a stiffness loss at the bottom flange, the neutral axis moved up in order to satisfy the force equilibrium in the section.

Figure 4-9: Crushing of the Concrete Slab of Specimen S3-HT
Figure 4-10: Predicted versus Measured Load-Deflection Diagram of Specimen S3-HT

Figure 4-11: Load-Strain Diagram of Steel Bottom Flange and CFRP Strands of Specimen S3-HT
Figure 4-12: Load-Strain Diagram of the Top Surface of the Concrete Slab of S3-HT

Figure 4-13: Load-Strain Diagram for the Steel Beam Top Flange of Specimen S3-HT
4.2.3. Test Results of Specimen S4-HT

Test specimen S4-HT was strengthened with four layers of high tensile CFRP strand sheets and loaded to failure under static loading. Failure, which was ductile in nature, occurred due to crushing of the concrete at the top compression zone of the concrete slab as shown in Figure 4-14. The predicted load-deflection behavior versus the measured load-deflection relationship of specimen S4-HT is shown in Figure 4-15. The predicted load-deflection behavior matches the measured values very well. Test results suggest that the analytical model can be successfully used to predict the behavior of the specimens strengthened with high tensile CFRP materials. The measured load at failure was 82 kips including the dead load and the weight of the test setup and the corresponding deflection was 2.07". An increase of 70.8 percent in the ultimate flexural capacity was achieved using the four layers of high tensile CFRP strengthening system. The ultimate flexural capacity increase of specimen S4-HT is 11.8 percent higher than specimen S3-HT. Adding another layer of the high tensile CFRP strand sheets increased the capacity by 11.8 percent. The displacement ductility of specimen S4-HT was calculated as 4.06 in/in. The ultimate deflection and the displacement ductility of specimen S4-HT was lower than specimen S3-HT. Test results show that as the strengthening ratio increases, the ultimate deflection and the displacement ductility of the strengthened specimen decrease. The pre-yielding flexural stiffness, which was calculated using the slope of the linear portion of the load-deflection diagram, was 75.5 kips/in. The pre-yielding and the post-yielding flexural stiffness were increased by 13.2 & 183.3 percent respectively. The pre-yielding flexural stiffness was improved slightly whereas the post-
yielding flexural stiffness increased significantly using this strengthening system due to the stiffness of the CFRP strands. Adding more layers of high tensile CFRP strands sheets did not have a significant effect on the pre-yielding flexural stiffness however it considerably increased the post yielding flexural stiffness. The measured strains at the top surface of the steel bottom flange and at the bottom surface of the CFRP strands are shown in Figure 4-16. The load-strain relationship of the steel bottom flange indicates a yield load of 39 kips which corresponds to the yield strain of the steel at the bottom flange. The yield load was increased by five percent as the number of the high tensile CFRP layers increased from three to four. The strains of the CFRP strands and the steel bottom flange are compatible through all loading stages and the minimal difference between the strains is due to the thickness of the bottom flange. The strain of the CFRP strand sheets at failure was 0.0125 in/in which shows a 73 percent utilization of the high tensile CFRP strands. The CFRP strands of specimen S4-HT are 6.5 percent less utilized compared to specimen S3-HT. Increasing the strengthening ratio resulted in a decrease in the degree of utilization of the strands. The load-strain diagram of the top surface of the concrete slab shown in Figure 4-17 reflects a strain of 0.005 in/in at failure. The load-strain diagram of the top flange of the steel beam is shown in Figure 4-18. The change from compressive strain to tensile strain is due to shifting of the neutral axis towards the concrete slab. As the steel bottom flange yielded, the neutral axis began to shift upwards in order to satisfy the force equilibrium within the section. The reduction in the tensile strain of the steel top flange is due to the localized crushing of the top compression zone of the concrete slab which caused the downwards movement of the neutral axis.
Figure 4-14: Crushing of the Concrete Slab of Specimen S4-HT

Figure 4-15: Predicted versus Measured Load-Deflection Diagram of Specimen S4-HT
Figure 4-16: Load-Strain Diagram of the Steel Bottom Flange & the CFRP Strands of S4-HT

Figure 4-17: Load-Strain Diagram of the Top Surface of the Concrete Slab of Specimen S4-HT
4.2.4. Test Results of Specimen S3-IM

Test specimen S3-IM was strengthened with three layers of intermediate elastic modulus CFRP material which has a higher elastic modulus and is less ductile in comparison to the high tensile CFRP material. The specimen was loaded under static loading to failure. Failure was brittle in nature and occurred due to rupture of the CFRP strands as shown Figure 4-19. The measured load-strain behavior of the top surface of the steel bottom flange and the load strain diagram of the CFRP strands are shown in Figure 4-20. The load-strain relationship of the steel indicates a yield load of 38 kips which corresponds to the yield strain of the steel at the bottom flange. The yield load increase of specimen S3-IM is five percent higher than specimen S3-HT. The strain of the steel bottom flange is compatible with the strain of the
CFRP strands before yielding of the steel at bottom flange. After yielding of the steel bottom flange, the strains of the steel and the CFRP strands began to deviate significantly. Test results indicate the presence of shear lags between the intermediate modulus CFRP strands and the steel. The steel at the bottom flange lost its stiffness after yielding while the intermediate modulus CFRP strands had the same elastic modulus. The significant stiffness difference between the steel and the intermediate modulus CFRP strands after yielding resulted in a shear lag. The predicted load-deflection behavior of specimen S3-IM is compared to the measured load-deflection relationship in Figure 4-21. The measured load at failure was 74 kips and the corresponding deflection at failure was 1.58”. The results shows that an increase of 54.2 percent in the ultimate flexural capacity was achieved using this strengthening system. The ultimate flexural capacity increase of specimen S3-IM was 5.2 percent more compared to the ultimate flexural capacity increase of specimen S3-HT. The displacement ductility of specimen S3-IM was calculated as 3.5 in/in. Using the intermediate modulus CFRP material, which has a higher elastic modulus compared to the high tensile CFRP strands, resulted in a lower deflection at failure and displacement ductility compared to specimen S3-HT. The pre-yielding flexural stiffness, calculated using the slope of the linear portion of the load-deflection diagram, was 87.4 kips/in. The pre-yielding and the post-yielding stiffness were increased by 31.1 & 228 percent respectively. The results show that the intermediate modulus CFRP strands had a greater effect on the flexural stiffness compared to the high tensile CFRP strands. Both pre-yielding and post-yielding flexural stiffness were significantly increased using the three layers of intermediate modulus
strengthening system. After rupture of the CFRP strands, which is shown as a sudden drop of the load carrying capacity, the beam was still capable to carry load as an unstrengthened steel-concrete composite beam and the failure occurred due to crushing of the concrete at the top zone of the concrete slab. The load-deflection behavior of the beam after rupture of the strands was very similar to the behavior of the control specimen. The predicted load-deflection diagram was considerably different from the measured load-deflection relationship. The analytical model significantly underestimated the load-deflection behavior of specimen S3-IM. This is mainly due to the linear strain profile assumption that the model was based on. The strain profile of the specimen S3-IM is not linear after the yielding of the steel at bottom flange as it was discussed in the load-strain diagram comparison of the steel and the intermediate modulus CFRP shown in Figure 4-20. The load-strain diagram of the top surface of the concrete slab is shown in Figure 4-22. The strain of the top surface of the concrete slab at failure was 0.003 in/in. The load-strain diagram of the steel top flange is shown in Figure 4-23. The change of the strain of the steel top flange from compression to tension reflects the movement of the neutral axis towards the top concrete slab. The neutral axis started to shift upwards as the steel at the bottom flange yielded. The loss in the stiffness of the steel bottom flange resulted in movement of the neutral axis in order to satisfy the force equilibrium in the section. The sudden drop in the tensile strain of the steel top flange is due to the rupture of the intermediate modulus CFRP strands at the bottom flange of the steel.
Figure 4-19: Rupture of the Intermediate Modulus CFRP Strands of Specimen S3-IM

Figure 4-20: Load-Strain Diagram of the Steel Bottom Flange & CFRP Strands of S3-IM
Figure 4-21: Predicted versus Measured Load-Deflection Diagram of Specimen S3-IM

Figure 4-22: Load-Strain Diagram of the Top Surface of the Concrete Slab of S3-IM
4.2.5. Test Results of Specimen S4-IM

Test specimen S4-IM was strengthened with four layers of intermediate modulus CFRP strand sheets and loaded to failure under static loading. Failure, which was brittle in nature, occurred due to rupture of the CFRP strands as shown in Figure 4-24. The load-strain relationship of the top surface of the steel bottom flange and the load-strain diagram of the intermediate modulus CFRP strands are shown in Figure 4-25. The relationship indicates that the yield load of specimen S4-IM is 41.5 kips which corresponds to the yield strain of the steel at the bottom flange. The yield load increase for specimen S4-IM is 11.7 more percent than the specimen S3-IM. This suggests that the intermediate modulus CFRP material has a greater effect on the yield load compared to the high tensile CFRP material. The strains of
The results reflect the presence of shear lag between the intermediate modulus CFRP strands and the steel surface. The steel at the bottom flange lost its stiffness significantly after yielding while the intermediate modulus strands had the same stiffness. The significant stiffness difference between the intermediate modulus strands and the steel bottom flange resulted in a shear lag. The predicted load-deflection relationship versus the measured behavior of specimen S4-IM is shown in Figure 4-26. After rupture of the CFRP strands, which was brittle in nature, a sudden drop of the load carrying capacity was observed and the beam was still capable to carry load equivalent to an unstrengthened steel-concrete composite beam and failure occurred due to crushing of the concrete at the top compression zone of the slab. The measured load at failure of specimen S4-IM was 80.5 kips and the corresponding deflection was 1.65”. An increase of 66.7 percent in the ultimate flexural capacity was achieved using this strengthening system. The flexural capacity increase is 4.2 percent less compared to specimen S4-HT. The displacement ductility of specimen S4-IM was calculated as 3.45 in/in. It was observed that increasing the number of the IM CFRP layers from 3 to 4 did not have a significant effect on the displacement ductility and the ultimate deflection. The pre-yielding flexural stiffness was calculated using the slope of the linear portion of the load-deflection diagram as 90.6 kips/in. The pre-yielding and the post-yielding stiffness were increased by 35.9 and 236.3 percent respectively. Increasing the strengthening ratio of the IM CFRP strands did not significantly improve the pre-yielding and the post-yielding flexural stiffness. The predicted load-
deflection behavior is considerably different from the measured load-deflection diagram and the analytical model significantly underestimates the ultimate flexural capacity. This is mainly due to the linear strain profile assumption that the model is based on. The assumption is not valid for specimen S4-IM after yielding of the steel at the bottom flange as it was discussed in Figure 4-25. The load-Strain diagram of the top surface of the concrete slab is shown in Figure 4-27. The measured strain of the top surface of the concrete slab at failure is 0.0039 in/in. The load-strain diagram of the steel top flange is also shown in Figure 4-28. The relationship indicated that the neutral axis started to shift upwards because of the stiffness loss at the steel bottom flange due to the yielding of steel.

Figure 4-24: Rupture of the CFRP Strand Sheets of Specimen S4-IM
Figure 4-25: Load-Strain Diagram of the Steel Bottom Flange & CFRP Strands of S4-IM

Figure 4-26: Predicted versus Measured Load-Deflection Diagram of Specimen S4-IM
Figure 4-27: Load-Strain Diagram of the Top of the Concrete Slab of Specimen S4-IM

Figure 4-28: Load-Strain Diagram of the Steel Beam Flange of Specimen S4-IM
4.2.6. Test Results of Specimen S4-HT-IM

Test Specimen S4-HT-IM was strengthened using both high tensile and intermediate modulus CFRP strands and loaded to failure under static loading. Three layers of high tensile CFRP strand sheets were attached to the bottom surface of the steel bottom flange while two layers of the intermediate modulus strand sheets were attached with a height of 2.1” to each side of the web and covered the top surface of the steel bottom flange as shown in Figure 4-29. Failure, which was brittle in nature, occurred due to rupture of the intermediate modulus CFRP strands at the web as shown in Figure 4-30. The intermediate modulus CFRP strands have a relatively low ultimate strain capacity compared to the high tensile strands. As a result, the failure occurred at the top surface of bottom flange where the intermediate modulus CFRP strands reached their ultimate strain capacity. The load-deflection behavior of specimen S4-HT-IM is shown in Figure 4-31. The measured load at failure is 85.3 kips and the corresponding deflection at failure is 1.85”. An increase of 77.7 percent in the ultimate flexural capacity is achieved using this strengthening system. Specimen S4-HT-IM had the largest ultimate flexural capacity among phase I specimen but it also had the biggest strengthening ratio among the phase I specimens. The displacement ductility of specimen S4-HT-IM was calculated as 3.56 in/in. The results reflect that specimen S4-HT-IM was more ductile than the specimens strengthened only with intermediate modulus CFRP strands and it was less ductile compared to the specimens strengthened only with the high tensile CFRP material. The pre-yielding flexural stiffness was calculated using the slope of the linear portion of the load-deflection diagram as 78.8 kips/in. The pre-yielding and the post-yielding
stiffness were increased by 18.2 & 242.4 percent respectively. Using a combination of IM and HT CFRP strands for strengthening did not improve the behavior efficiently considering the high strengthening ratio used for this specimen. The load-strain behaviors of the steel bottom flange, the high tensile CFRP strands and the intermediate modulus CFRP strands at the bottom flange are shown in Figure 4-32. The load-strain relationship of the steel bottom flange reflects a yield load of 41 kips which corresponds to the yield strain of the steel bottom flange. The strains of the steel bottom flange and the high tensile CFRP strands are compatible at all load levels whereas the strain of the intermediate modulus CFRP strands deviates from the strain of the steel bottom flange after yielding of the steel. The high tensile CFRP strands have a relatively low modulus compared to the intermediate modulus strands. After yielding, the steel lost its stiffness but the strain of the steel remained compatible with the strain of the high tensile CFRP strands due to its low elastic modulus. On the other hand, the intermediate modulus CFRP strands, which have a relatively high elastic modulus, started to show a different behavior after yielding of the steel. This reflects the presence of shear lag between the intermediate modulus CFRP strands and the steel bottom flange due to the significant stiffness difference after yielding of the steel. The load-strain diagram of the top surface of the concrete slab is shown in Figure 4-33. The measured strain of the concrete at failure is 0.004 in/in.
Figure 4-29: Strengthening Configuration for Specimen S4-HT-IM

Figure 4-30: Rupture of the IM CFRP Strands at the Web of Specimen S4-HT-IM
Figure 4-31: Load-Deflection Diagram of S4-HT-IM Specimen

Figure 4-32: Load-Strain Diagram of the Steel, HT & IM Strands at the Bottom Flange
4.3 TEST RESULTS OF PHASE II

Test Specimen S4-HT-C was strengthened with four layers of high tensile CFRP strands and loaded to failure under cyclic loading. The loading protocol for specimen S4-HT-C is described in section 3.5. Failure occurred due to crushing of the concrete at the top compression zone of the concrete slab as shown in Figure 4-34. The load-deflection diagram of specimen S4-HT-C is shown in Figure 4-35. The measured load at failure is 78 kips and the corresponding deflection at failure is 2.55”. An increase of 62.5 percent in the ultimate flexural capacity was achieved using this strengthening system. The ultimate flexural capacity increase of specimen S4-HT-C is 8.3 percent less than the increase in specimen S4-
HT. The pre-yielding flexural stiffness was calculated using the slope of the linear portion of the load-deflection curve as 73.5 kips/in which is almost equal to the flexural stiffness of specimen S4-HT. No change in the pre-yielding flexural stiffness was observed under the effect of the cyclic loading. Test results indicated that there is not a significant change in the behavior of the strengthened specimen under the effect of the cyclic loading. The residual deflection before yielding of the specimen is negligible while it increases significantly after yielding of the bottom flange. The load-strain behaviors of the steel bottom flange and the high tensile CFRP strands are shown in Figure 4-36. The load-strain relationship of the steel bottom flange shows a yield load of 36.2 kips/in. The strains of the steel bottom flange and the CFRP strands are compatible at all stages of loading. No shear lag was observed between the high tensile CFRP strands and the steel bottom flange under the effect of the cyclic loading since the load-strain behaviors of the high tensile CFRP strands and steel bottom flange were almost identical. The small difference in the strains of the high tensile CFRP strands in comparison to the steel is due to the strain measurement locations which are different by the thickness of the flange. The strain of the CFRP strands at failure was 0.01295 in/in which shows a 76 percent utilization of the CFRP Strands. The utilization of the CFRP strands was determined based on comparing the measured strain in the CFRP at failure to the ultimate tensile strain measured from the tensile coupon tests. The load-Strain diagram of the top surface of the concrete slab is shown in Figure 4-37. The measured strain of the top surface of the concrete slab at failure is 0.00365 in/in. Test results indicate that the selected strengthening system was effective for cyclic loading.
Figure 4-34: Crushing of the Concrete Slab for Specimen S4-HT-C

Figure 4-35: Load-Deflection Diagram for S4-HT-C Specimen
Figure 4-36: Load-Strain Diagram of the Steel Bottom Flange and the CFRP Strands

Figure 4-37: Load-Strain Diagram for the Top Surface of the Concrete Slab of S4-HT-C
4.4 TEST RESULTS OF PHASE III

Specimen S4-HT-F was strengthened by four layers of high tensile CFRP strands and it was subjected to fatigue loading. Test specimen S4-HT-F was subjected to two million cycles of fatigue loading before it was loaded to failure under static loading. Duration of testing was approximately two weeks and the fatigue loading frequency was 1.55 Hz. The data was recorded for seven seconds in every 20000 cycles of loading with a rate of 100 points per second. The maximum load level of the fatigue loading was 29 kips which is equivalent to 80 percent of the yield load of the strengthened section and the minimum load level was 7.8 kips as described in section 3.5. The load-deflection behavior of specimen S4-HT-F under the effect of fatigue loading is shown in Figure 4-38. Flexural stiffness degradation was observed during the two million cycles of fatigue loading which was due to the fatigue creep of the concrete in the compression zone. The residual deflection due to fatigue loading was 0.06”. At the end of two million fatigue load cycles, the specimen was unloaded and then loaded statically to failure. Failure occurred due to crushing of the concrete slab as shown in Figure 4-39. The load-deflection behavior under static loading is shown in Figure 4-40. The load at failure of specimen S4-HT-F was 75 kips and the corresponding deflection at failure was 2.59”. An increase of 56.2 percent in the ultimate flexural capacity was achieved using this strengthening system under fatigue loading. The ultimate flexural capacity increase of specimen S4-HT-F is 14.6 percent less than the capacity increase of specimen S4-HT. The pre-yielding flexural stiffness was calculated using the slope of the linear portion of the load-deflection curve shown in Figure 4-40 as 61 kips/in. Results indicate that there is not a
drastic change in the flexural stiffness and the flexural capacity under the effect of fatigue loading and the small differences due to fatigue loading is related to the fatigue creep of the concrete in the compression zone of the slab. The load-strain relationships of the steel bottom flange and the CFRP strands are shown in Figure 4-41. The load-strain relationship of the steel bottom flange reflects a yield load of 36.5 kips. The strains of the steel bottom flange and the CFRP strands are compatible at all loading stages. No shear lag was observed between the high tensile CFRP strands and the steel bottom flange since their strains were nearly identical. The small difference between the strains is due to the strain measurement locations which are different by the thickness of the steel flange. The strain of the CFRP strands at failure of the concrete slab was 0.01264 in/in which shows a 74 percent utilization of the CFRP material. The utilization of the CFRP strands was determined based on comparing the measured strain in the CFRP at failure to the ultimate tensile strain measured from the tensile coupon tests. The load-strain diagram of the top surface of the concrete slab is shown in Figure 4-42. The strain of the concrete at failure was 0.00454 in/in. Test results indicate that the selected strengthening system was effective for fatigue loading.
Figure 4-38: Load-Deflection Diagram of Specimen S4-HT-F under Fatigue Loading

Figure 4-39: Crushing of the Concrete Slab of Specimen S4-HT-F
Figure 4-40: Load-Deflection Diagram of Specimen S4-HT-F under Static Loading

Figure 4-41: Load-Strain Diagram of the Steel Bottom Flange and the CFRP Strands
4.5 DISCUSSION OF TEST RESULTS

This section includes a discussion of test results of the three phased experimental program completed in this study. A comparative study is conducted among different strengthening systems to examine the effect of the each parameter on behavior of the strengthening system. The criteria used to select the most effective strengthening system for the cyclic and the fatigue testing is also included in this chapter. Furthermore, the agreement between the predicted behavior and the measured values is discussed.
4.5.1. **Discussion of Test Results of Phase I**

The load-deflection behaviors of the test specimens in phase I are shown in Figure 4-43. The strengthened specimens showed a superior behavior in comparison to the control specimen. The load-deflection comparison of the phase I test specimens shows that the test specimens strengthened with the high tensile CFRP strands have a higher deflection at failure compared to the beams strengthened with the intermediate modulus CFRP strands. The failure mode for the specimens strengthened with the intermediate modulus CFRP material was rupture of the CFRP strands whereas the failure mode for the specimens strengthened only with the high tensile CFRP material was crushing of the concrete which is the most favorable failure mode for reinforced concrete structures. The displacement ductility of all specimens tested in phase I is shown in Figure 4-43. The ductility comparison of the phase I test specimens, shown in Figure 4-44, reflects that the specimens strengthened with the high tensile CFRP strands show a more ductile behavior in comparison to ones strengthened with the intermediate modulus strands. The ultimate flexural capacity increases of specimens tested in phase I are shown in Figure 4-45. Test specimen S4-HT-IM with combined high strength and intermediate modulus CFRP strands has the maximum increase in the ultimate flexural capacity followed by specimen S4-HT with high strength CFRP strands only. The pre-yielding and the post yielding flexural stiffness increases of phase I test specimens are shown in Figure 4-46 & Figure 4-47 respectively. Results show that the specimens strengthened with the intermediate modulus CFRP strands have a higher pre-yielding and post-yielding flexural stiffness compared to the ones strengthened with the high tensile CFRP strands. Test
Specimen S4-IM has the highest pre-yielding flexural stiffness among specimens tested in phase I.

*Figure 4-43: Load-Deflection Comparisons of Test Specimens of Phase I*
Figure 4-44: Displacement Ductility of the Phase I Test Specimens

Figure 4-45: Ultimate Flexural Capacity Increase of Phase I Specimens
Figure 4-46: Pre-Yielding Flexural Stiffness Increase of Phase I Specimens

Figure 4-47: Post Yielding Stiffness Increase of Phase I Specimens
Adding more layers of the CFRP strand sheets resulted in an increase in the ultimate flexural capacity, the pre-yielding flexural stiffness and the post-yielding flexural stiffness of the beams as it was predicted. Additionally, increasing the strengthening ratio and the elastic modulus of the strengthening material resulted in an increase in the yield load. The high tensile CFRP strands have a higher ultimate stress capacity but they were less utilized in comparison to intermediate modulus CFRP strands. This was the main reason for the similarity of the ultimate flexural capacity increases between two different CFRP materials. The Intermediate modulus CFRP strands provided more increase in the pre-yielding and the post-yielding flexural stiffness due to their relatively higher elastic modulus. The predicted load-deflection behaviors versus the measured load-deflection behaviors of phase I specimens are provided in Figure 4-48 & Figure 4-49 to examine the validity of the analytical model. As it can be seen from Figure 4-48, the analytical load-deflection predictions for specimens S3-HT and S4-HT are in a good agreement with the measured load-deflection diagrams. On the other hand, Figure 4-49 shows that the predicted behavior is not compatible with the measured load-deflection relationship and the model significantly underestimates the ultimate capacity of the beams strengthened with intermediate modulus CFRP strands. The main reason for the difference between the analytical prediction and the measured results for specimens S3-IM & S4-IM is the assumption of linear strain profile along the cross section used in the analysis at all loading stages. This assumption was found to be not perfectly valid for the specimens strengthened with the intermediate modulus CFRP strands due to the presence of shear lags after yielding of the steel bottom flange. The strain
The strain profiles of specimens S3-HT & S4-HT are linear up to failure whereas the strain profiles of S3-IM & S4-IM shows a non-linearity after yielding of the steel bottom flange. This is mainly due to the significant stiffness difference between the steel and the intermediate modulus CFRP strands after yielding of the steel. The shear-lag was observed in specimens S3-IM, S4-IM & S4-HT-IM. The shear-lag is mainly affected by the elastic modulus of the CFRP materials and the strengthening ratio. As the elastic modulus of the strengthening materials increases, the shear-lag becomes more noticeable since the gap between the elastic modulus of the CFRP material and the steel increases after yielding of the steel. However, an increase in the strengthening ratio results in a decrease in the shear lag. As the strengthening ratio increases, the tensile forces at the bottom flange increases. As a result, the neutral axis starts to move downwards to satisfy the force equilibrium in the section. So, for a assumed level of load, as the strengthening ratio increases, the curvature decreases. This results in smaller strain at the bottom flange resulting in a reduced shear lag. A summary of the test results for phase I test specimens is presented at Table 4-3.
Figure 4-48: Load-Deflection Diagram for Specimens S3-HT & S4-HT

Figure 4-49: Load-Deflection Diagram of Specimens S3-IM & S4-IM
Figure 4-50: Strain Profile of S3-HT

Figure 4-51: Strain Profile of S4-HT

Figure 4-52: Strain Profile of S3-IM

Figure 4-53: Strain Profile of S4-IM
<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>$\text{EI}_{\text{pre-yield}}$ Increase %</th>
<th>$\text{EI}_{\text{post-yield}}$ Increase %</th>
<th>$\text{F}_u$ Increase %</th>
<th>Ductility</th>
<th>Shear Lag</th>
</tr>
</thead>
<tbody>
<tr>
<td>S3-HT</td>
<td>13.18</td>
<td>96.1</td>
<td>49</td>
<td>4.27</td>
<td>No</td>
</tr>
<tr>
<td>S4-HT</td>
<td>13.23</td>
<td>183.3</td>
<td>70.83</td>
<td>4.06</td>
<td>No</td>
</tr>
<tr>
<td>S3-IM</td>
<td>31.1</td>
<td>228</td>
<td>54.17</td>
<td>3.5</td>
<td>Yes</td>
</tr>
<tr>
<td>S4-IM</td>
<td>35.9</td>
<td>236.3</td>
<td>66.7</td>
<td>3.45</td>
<td>Yes</td>
</tr>
<tr>
<td>S4-HT-IM</td>
<td>18.2</td>
<td>242.3</td>
<td>77.7</td>
<td>3.56</td>
<td>Yes</td>
</tr>
</tbody>
</table>

Test specimen S4-HT-IM had the maximum increase in the ultimate flexural capacity followed by specimen S4-HT. Test specimen S4-IM provided the maximum increase in the pre-yielding flexural stiffness. However, the failure mode for specimens S3-IM, S4-IM & S4-HT-IM had a brittle nature. Specimens S3-HT and S4-HT showed ductile behaviors. The ductility of the specimens strengthened with the high tensile CFRP materials were higher than the ductility of the specimens strengthened with the intermediate modulus CFRP materials. Adding the intermediate modulus CFRP strand sheets to the web of specimen S4-HT-IM did not significantly improve the behavior considering the high strengthening ratio used for specimen S4-HT-IM. As a result, the S4-HT-IM strengthening system was
inefficient. Figure 4-54 presents the ratio of the strain at failure to the ultimate strain of the strands for specimens S3-HT & S4-HT. As the strengthening ratio increases, the CFRP materials utilization decreases whereas the materials are utilized more effectively using higher elastic modulus strands. Specimen S4-HT was determined as the optimal strengthening system since it provided a significant increase in the flexural strength while showing a ductile behavior. The failure mode was crushing of the concrete slab which is a typical failure mode in steel-concrete composite beams. As a result, specimen S4-HT was selected to be tested under cyclic and fatigue loading.

Figure 4-54: Degree of effectiveness Comparison for Specimens S3-HT & S4-HT
4.5.2. Discussion of Test Results of Phase II

The load-deflection behaviors of specimens S4-HT & S4-HT-C are shown in Figure 4-55. The specimen was tested under cyclic loading to study the effect of the cyclic loading on the behavior of specimen strengthened with high tensile CFRP strands. The load-deflection behavior indicates that specimen S4-HT-C has approximately the same behavior as specimen S4-HT. The slight differences in the ultimate load, the ultimate deflection and the post-yield stiffness can be attributed to possible creep in the concrete slab due to the cycling loading and the different concrete properties of two batches. As it was mentioned in chapter 3, two different batches of concrete were used at different dates for casting the slabs. Second concrete batch which included specimens S4-HT-C & S4-HT-F had a lower ultimate strength and lower elastic modulus. This can be another influencing factor in the slight differences between the load-deflection behaviors of specimens S4-HT & S4-HT-C. The strain profile of specimen S4-HT-C at failure is shown in Figure 4-56. A comparison is made between the degree of utilization of specimens S3-HT, S4-HT and S4-HT-C in Figure 4-57. The high tensile CFRP strands of specimen S4-HT-C were utilized to the same degree as they were utilized in specimen S4-HT. This confirms the fact that the slight difference in the load-deflection behaviors of specimens S4-HT-C & S4-HT is either due to the concrete creep or the relatively lower concrete qualities of the second batch used to cast S4-HT-C slab. The S4-HT-C strengthening system increased the ultimate flexural capacity by 62.5 % under cyclic loading. No shear lag was observed under the effect of the cyclic loading. Test results indicated that the selected strengthening system was effective for cyclic loading.
Figure 4-55: Load-Deflection Comparison between S4-HT & S4-HT-C

\[ \varepsilon_c = 0.00365 \]

Figure 4-56: Strain Profile at Failure for S4-HT-C Specimen

\[ \varepsilon_s = 0.012 \]
\[ \varepsilon_{CFRP} = 0.01295 \]
4.5.3. Discussion of Test Results of Phase III

The load-Deflection behaviors of specimen S4-HT-F which was strengthened with high strength CFRP strands and subjected to fatigue loading and specimen S4-HT are shown in Figure 4-58. The pre-yielding flexural stiffness of specimen S4-HT-F was calculated as 61 kips/in which is 19.5 percent less than the pre-yielding flexural stiffness of specimen S4-HT. The difference in the flexural strength and the flexural stiffness of specimens S4-HT-F & S4-HT is due to the fatigue creep in the concrete slab. Longitudinal cracks were observed at the load points during the test as shown in Figure 4-59. The longitudinal cracks were caused by the longitudinal moments from the point loads which were distributed along the width of the
slab. The strain profile of specimen S4-HT-F at failure is presented in Figure 4-60. The strain profile of the specimen S4-HT-F was linear at all loading stages. No shear lag was observed under the effect of the fatigue loading. Degree of effectiveness of the CFRP strands of all of the specimens strengthened with the high tensile CFRP strands are shown in Figure 4-61. Test results suggested that the CFRP strands in specimen S4-HT-F were utilized to the same level of utilization of CFRP strands in specimens S4-HT & S4-HT-C. This indicates the fact that the main reason of the flexural strength difference of the fatigue test specimen was the fatigue creep or the low concrete qualities of the second batch. Considering a yield load of 30 kips for the control specimen and a yield load of 36.5 kips for the fatigue test specimen, the service load capacity was increased by 22 percent using this strengthening system. Test results indicated that the selected strengthening system was effective for fatigue loading.
Figure 4-58: Load-Deflection Comparison of Specimens S4-HT & S4-HT-F

Figure 4-59: Longitudinal Cracks at Load Points for Specimen S4-HT-F
Figure 4-60: Strain Profile of Specimen S4-HT-F at Failure

\[ \varepsilon_s = 0.0045 \]

\[ \varepsilon_{off} = 0.0127 \]

Figure 4-61: Degree of Effectiveness of Strengthening with High Tensile CFRP Strands
5. SUMMARY & CONCLUSIONS

The research reported in this thesis presents an innovative strengthening system for steel structures and bridges. The proposed system includes a new material and new configurations of a carbon fiber reinforced polymer, CFRP, which is believed to have an advantage over the different strengthening techniques and materials presented in the literature review. The research findings are based on an experimental program conducted on eight scaled typical steel-concrete composite sections used for bridges. The experimental program included three phases in addition to the material properties testing to determine the mechanical properties and the fundamental characteristics of debonding of the proposed material from steel surfaces. Configuration of the CFRP material used in this research consists of small diameter strands provided in a sheet format to simplify the installation process. Three types of small diameter CFRP strands were considered for strengthening purposes including high strength, intermediate modulus and high modulus CFRP strands. The high strength and the intermediate modulus CFRP materials were selected for the first phase of the experimental program. In this phase, five scaled steel-concrete composite beams strengthened with high tensile and intermediate modulus CFRP strands were subjected to static loading. The first phase investigated the effect of the type of the CFRP material, strengthening ratio and strengthening configuration. The behaviors of specimens of phase I were analyzed and compared to the control specimen. Based on the measured results of the first phase, the most effective strengthening system was selected for the second and the third phase where the beams were subjected to cyclic and fatigue loading, respectively, to study the behavior under
service life of the proposed strengthening system. A series of double strap joints were strengthened with high strength CFRP material and tested in tension to determine the bond length of the high tensile CFRP material. This chapter presents a summary of the research findings of the study on strengthening steel structures using the proposed carbon fiber reinforced polymer material. The findings are based on the comparative experimental program undertaken to evaluate the behavior of the steel-concrete composite beams strengthened using different strengthening systems. The findings from the double strap joints tests are presented. The findings also address the observed shear lag phenomenon which is affected by the elastic modulus of the CFRP material. Recommendations for the possible future work are presented.

In general, test results show that the proposed CFRP strands provided an excellent strengthening system for steel structures. The main findings of the research program are summarized below:

- Using high strength CFRP strands with an appropriate reinforcement ratio results in a failure due to crushing of the concrete in the compressive zone which is the most favorable mode of failure
- Using an appropriate design, peeling of the CFRP material can be totally eliminated and therefore solve the concern related to peeling of the CFRP laminates reported in most of the previous research dealing with strengthening of steel structures
• Preventing the debonding problems is due to the total coverage of the strands by the epoxy adhesive

• High tensile CFRP strengthening system is highly recommended for strengthening of steel structures due to its high strength and strain at failure

• High modulus CFRP strands can be effectively used in cases where strengthening is required to reduce deflection of the flexural members. Nevertheless, reinforcement ratio should be increased to avoid early rupture of the CFRP strands before the required increase of the ultimate strength of the beam

• High elastic modulus of IM CFRP material can cause shear lag due to the significant difference of its elastic modulus to the elastic modulus of the steel after yield. Increasing the load requires an increase in the internal tensile forces to satisfy the force equilibrium. However, due to the relatively higher elastic modulus of the intermediate modulus CFRP strands, the required strain in the CFRP is less than the corresponding strain of the steel.

• The intermediate modulus CFRP strands provide higher flexural stiffness of the strengthened beam before and after yielding

• Use of CFRP strengthening systems delays the yielding of steel to higher load levels. In addition, using intermediate modulus CFRP strands results in relatively higher yield load compared to high tensile CFRP strands

• Using high tensile CFRP strands results in a higher ductility compared to intermediate modulus CFRP strands
• Failure mode in the beams strengthened with the intermediate modulus CFRP strands was brittle due to rupture of the CFRP materials. Whereas, it was ductile in the beams strengthened with the high tensile CFRP materials due to crushing of the concrete in the compressive zone of the slab.

• Increasing the number of the high tensile CFRP layers decreases the efficiency of the CFRP materials used for the strengthening system.

• Adding the intermediate modulus CFRP strands to the web did not effectively improve the performance considering the high strengthening ratio used for specimen S4-HT-IM.

• The combined strengthening system is not efficient since the combined strengthened specimen did not provide significant improvement in its behavior compared to the other strengthened specimens.

• No shear lag was observed under the effect of cyclic and fatigue loadings.

• The selected strengthening system using high tensile CFRP performed successfully under cyclic and fatigue loadings.

• Clamping can significantly enhance the bond behavior of the CFRP strands.

• The development length of the high strength CFRP strands was found to be in a range of 10” to 12”.

The proposed future research may consider the following:

• Testing a significant number of strengthened steel-concrete composite beams to provide appropriate statistical data.
• Testing full scale steel-concrete composite beams strengthened with the new CFRP material to demonstrate its efficiency for real structures

• Perform additional testing and study the shear lag behavior and introduce an analytical model

• Study the effectiveness of the new CFRP material for shear strengthening of steel structures and bridges
REFERENCES

Alessio Pipinato, Carlo Pellegrino & Claudio Modena. “Fatigue Behavior of Steel Bridge Joints Strengthened with FRP Laminates”, *Modern Applied Science; Vol. 6, No. 10; 2012*


Amr Shaat and Amir Fam, M.ASCE. “Repair of Cracked Steel Girders Connected to Concrete Slabs Using Carbon-Fiber-Reinforced Polymer Sheets”, *Journal of Composites for Construction © ASCE 2008*


A. Monfared, K. Soudki, & S. Walbridge. “CFRP Reinforcing to extend the fatigue lives of steel structures”, *Fourth International Conference on FRP Composites in Civil Engineering (CICE 2008) Zurich, Switzerland*
Chao Wu, Xiaoling Zhao, Wen Hui Duan, Riadh Al-Mahaidi. “Bond characteristics between ultra-high modulus CFRP laminates and steel”, *Journal of Thin-Walled Structures* 51 (2012) 147–157


Trent C. Miller, Michael J. Chajes, Dennis R. Mertz and Jason N. Hastings. “Strengthening Of A Steel Bridge Girder Using CFRP Plates”, *Journal of Bridge Engineering 2001*

Ye Huwen, Christian König, Thomas Ummenhofer, Qiang Shizhong & Robin Plum. “Fatigue Performance of Tension Steel Plates Strengthened with Pre-stressed CFRP Laminates”, *Journal of Composites for Construction © ASCE 2010*