2016

Behavior of Concrete Beams with Corroded Reinforcement Retrofitted with Carbon Fiber Reinforced Polymer

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By

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A research project submitted in partial fulfillment of the
requirements for the degree of

MASTER OF SCIENCE
IN
CIVIL & ENVIROMENTAL ENGINEERING

Research Project Advisor:
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Portland State University
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Abstract

Severe premature deterioration has been reported in a large number of reinforced concrete (RC) structures in corrosive environments. Many concrete structures built in the past few decades are already showing signs of deterioration due to the corrosion of steel reinforcement. This premature deterioration can diminish structural integrity and safety of the structure.

There are several options available for retrofitting the structural members of existing reinforced concrete (RC) structures. Bonding thin steel plates is one of the common methods of retrofitting. Though the technique is successful in practice, the added steel plates are susceptible to corrosion, which leads to an increase in future maintenance costs. Therefore, attention has shifted to the use of carbon fiber reinforced polymer (CFRP) as alternative material. Based on previous studies, bonding CFRP sheets to the damaged members helps increase load carrying capacity, ductility, and stiffness of the damaged structure. Such a technique is an effective way to improve the flexural and shear performance of the RC damaged structure. In this experimental study, CFRP materials were used for structural strengthening. CFRP materials do not corrode because they are a combination of carbon fibers and an epoxy resin matrix. Moreover, they have very high strength and rigidity in the fiber direction.

The project focused on retrofitting RC beams that contained corroded steel, considering an extreme case of corrosion. The steel in RC beams were assumed to be fully corroded, resulting in the most severe loss in steel cross-section and strength. Unidirectional CFRP sheets were used to strengthen the deteriorated RC beams. This type of retrofitting increases the load carrying capacity of the corrosion damaged RC beams. It also increases the flexural and fatigue strength of the damaged RC beam.

The experimental program included strengthening and testing five simply supported rectangular cross section RC beams. All beams had the same cross section, 4in. x 6in., and were 6ft. long. The oiled steel rebars were safely pulled out of the formwork after the concrete had cured for few hours, leaving voids. This technique was used to represent the total loss in steel cross section in an extreme corrosive environment.

The first specimen was a control RC beam, which had no corrosion. It was tested to compare against corroded and repaired members. The second specimen was a plain concrete beam, and the third an un-retrofitted deteriorated beam. The two remaining deteriorated beams were strengthened by externally bonding one and two layers of CFRP. The CFRP sheets were bonded in the longitudinal as well as the vertical direction of the beams, and were tested under third-point loading.

The effectiveness of the repairing technique was determined by evaluating the performance in terms of load carrying capacity, deflection, and ductility. Test results revealed that bonding two layers of CFRP to the deteriorated RC beams increased the load capacities to two times the control RC beam without corrosion. The load deflection response of specimens showed that for the retrofitted specimens had a higher stiffness under service load conditions.

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CHAPTER 1: **INTRODUCTION**

1.1.   Overview

Reinforced concrete (RC) is known to be the most widely used building material due to its extensive availability. It is used in different engineering applications worldwide such as buildings, bridges, dams, and newly as a foundation system for wind turbine towers. Due to the wide variety of reinforced concrete uses, RC structures are subjected to a range of different environmental exposures including marine, industrial, nuclear, and other extreme environments. As a result, many RC structures experience an unacceptable loss in serviceability or safety far earlier than anticipated. Severe premature deterioration has been reported in a large number of concrete structures in corrosive environments. Many concrete structures built in the past decades are already showing signs of deterioration due to the corrosion of steel reinforcement. This premature deterioration is a problem in terms of the structural integrity and safety of the structure that requires remedial attention.

The damage to RC structures resulting from the steel reinforcement corrosion is exhibited in the decrease of the steel cross section and the formation of rust (iron oxide) inside the concrete. As a result, an internal stress is induced in the concrete, which leads to the cracking and spalling of concrete. Concrete cover cracking due to reinforcement corrosion is widely accepted as a limit-state indicator in defining the end of functional service life for existing RC structures undergoing corrosion.

Different techniques have been developed and used to repair a variety of structural deficiencies. The conventional methods include steel jacketing of concrete columns, external post-tensioning, and bonding steel plates to concrete beams. However, these conventional techniques are not cost effective, and some common problems such as corrosion will also be

present after the repair. For economic benefits, innovative repair techniques have been developed and tested so that durability of the concrete structure can be improved and the service life prolonged. The alternative is to repair damaged reinforced concrete structures with externally bonded carbon fiber reinforced polymer (CFRP). This advanced composite material is lightweight, has high strength and stiffness to weight ratio, corrosion resistance, and has high fatigue strength. In addition, CFRP is flexible and can be rapidly applied to flat or curved surfaces. Due to its advantages, it has been applied in many areas such as aerospace, defense, marine, equipment, and automotive sector [1]. CFRP composites have been used as structural materials since World War II, when they were first used in the construction of British Spitfires [2]. These materials have mechanical and physical properties in excess of those of steel. More will be presented about the CFRP characteristics in the literature review.

1.2.   Research Significance and Objective

The primary objective of this experimental study is to investigate and gather knowledge on the performance of corroded reinforced concrete beams externally bonded with CFRP. Published research studies have provided valuable findings, particularly with regard to addressing the effect of CFRP on strengthening the flexural strength and stiffness of corroded beam. Most of the previous studies on corroded concrete beams were based on accelerating the corrosion in the system. The corrosion rate was varied between 5% (mild) and 20% (severe), which represents the fraction of loss in the cross-sectional area of the steel reinforcement. However, little research work has been devoted to study the feasibility of using CFRP laminates to improve the strength of fully corroded beams.

In this study, an extreme case of corrosion is considered. Extreme corrosion is defined in this study as fully corroded or fully ineffective steel reinforcement resulting in complete loss of

the bond between the concrete and the steel reinforcement. Also, the worst case scenario assumes a complete loss in the rebar cross sectional area. In other words, steel reinforcement effect is considered non-active and the reinforcement is eliminated in the experiment. Hence, this study is going to provide an insight into the effect of bonding CFRP on the stiffness and load carrying capacity of fully corroded reinforced concrete members.

The specific objectives of the project are:

- Provide an insight into the effect of bonding CFRP on the:

    ➤ Load carrying capacity of concrete members with fully corroded reinforcement (worst case scenario).

    ➤ Stiffness and deflection at service load

    ➤ Total energy absorbed

- Compare the total flexural capacity of retrofitted beam to original beam

- Assess the effectiveness of two layers of CFRP to enhance the capacity and stiffness of the corroded beams.

- Compare ACI 440.2R design guidelines calculations to the experimental results.

CHAPTER 2: **LITERATURE REVIEW**

2.1. Steel Corrosion in Reinforced Concrete Structures

The purpose of this section is to review and summarize the latest knowledge on various aspects of corrosion of steel reinforcement, including the primary causes of corrosion and its effects on the RC structure. It also summarizes different methods (conventional and innovative) that are used to extend the service life of deteriorated structures.

2.1.1. Overview

Corrosion of reinforcing steel in reinforced concrete structures is generally considered as the most widespread mode of premature distress and deterioration of structural concrete. Because corrosion is progressive and the resultant damage distributed in severity, repairs are needed continually. If such visual indicators are not addressed, public safety is at risk. This results in a clear need for both the industry and field of research to explore and study this issue.

Throughout the years, considerable efforts have been made to understand corrosion mechanisms in RC structures, causes, failure modes, and possible rehabilitation methods. A lot of work has been done to study factors affecting the rate of steel corrosion in RC members. In addition to that, a number of studies have been carried out to evaluate the effect of corrosion on the behavior of RC structural members, as well as on the mechanical behavior of steel bars and the bond between steel and concrete. Generally, the knowledge developed over the past decades has led to improvements in the protection of reinforcement and rehabilitation of damaged structures.

Concrete normally acts to provide a high degree of protection against corrosion of the embedded reinforcement. The concrete inherently provides a highly alkaline environment, with a pH level between 12.5 and 13, to the steel through the formation of a passive film of iron oxides

[3]. This protects the steel against corrosion. Concrete also provides a physical barrier that prevents the steel from coming in contact with the external environment. This prevents substances such as water, salt, or other damaging ions from reaching the iron atoms that make up the steel. However, corrosion will still result in structures that experience poor concrete quality, poor design, or construction, and/or harsh environmental conditions, especially structures located in the coastal marine environment [4]. Figure 1 summarizes the effect of corrosion on concrete structures.
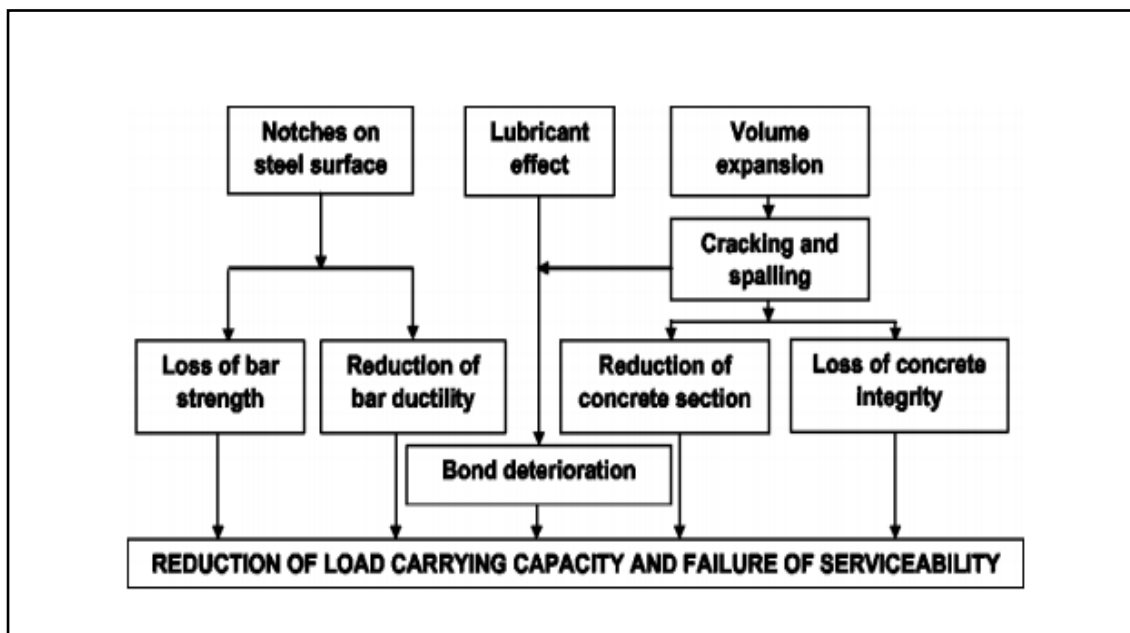
Figure 1: Effect of corrosion on structures [28]

2.1.2. Corrosion Process

The principal cause of steel corrosion is the presence of chlorides during the preparation of the concrete. In several places close to shore, sea sand is used as an aggregate in the mix. Also, some chemical admixtures, such as accelerators, can contain a high percentage of chlorides. De-icing salts used during wintertime can introduce chlorides to the reinforced steel as

well. According to the ACI Committee 222, steel corrosion in concrete is an electrochemical process where corrosion cells are generated due to differences in electrochemical potentials. Some areas of the steel bar become anodes, and some cathodes, as shown in Figure 2 [4].
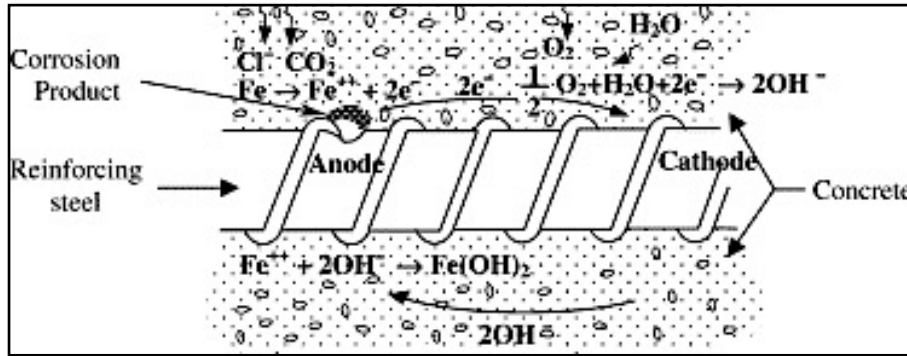
Figure 2: Illustrates a mechanism of corrosion process in steel bar [5].

The anodic reaction is the oxidation process, which results in the loss of metal. The cathodic reaction is the reduction of dissolved oxygen creating hydroxyl ions. The released hydroxyl ions at the cathode travel through the electrolyte to react with the ions at the anode, producing rust. The common anodic and cathodic reactions of steel in concrete are iron dissolution (equation 1) and oxygen reduction (equation 2) reactions.

$$Fe \rightarrow Fe^{2+} + 2e^- \hspace{3cm} Equation 1$$

$$2H_2O + O_2 + 4e^- \rightarrow 4(OH)^- \hspace{3cm} Equation 2$$
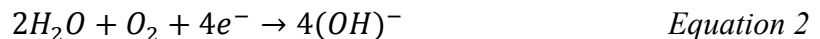
With the anodic reaction presented in equation 1, the cross section of the steel bar is reduced and the rebar could eventually lose its capacity and become non-active in a member.

2.1.3. Causes of Steel Corrosion

Differences in concrete parameters and the environmental factors, which can result in changes of the concrete properties, would be directly and indirectly responsible for the different

forms of corrosion damage to the RC structures. Thus, the corrosion behavior of steel in concrete is a function of steel and concrete parameters of steel and concrete and the properties of their interactional zone. The factors affecting corrosion of steel in concrete is classified into two major categories: external factors and internal factors.

2.1.3.1. External Factors (Environmental Factors)

The problem of steel corrosion is very prominent in parking garages and highway bridges where snow contaminated with deicing salt is frequently splashed during the winter time. A research done by Al-Ibweini et al. [6] indicates that steel corrosion is also noticeable in structures built in coastal areas where the ocean salts, which are primarily sodium chloride and other compounds, accumulate on the metal surfaces and accelerate the electrochemical reactions that cause rusting and other forms of corrosion. Therefore, environmental conditions play a key role in the formation of corrosion in reinforced concrete members. Oxygen, moisture, and chlorides must be found at the steel level in the concrete member to initiate the corrosion process. A certain mixture of these elements will ensure the continuation of corrosion activity. Among all the environmental factors, the presences of chloride ions and the penetration of carbon dioxide (carbonation process) have been responsible for most corrosion of steel in reinforced concrete structures according to the ACI Committee 222 [4].

Carbonation Process

Concrete carbonation results from the chemical reaction between the hydrated cement components (i.e. calcium hydroxide) and atmospheric carbon dioxide. This reaction lowers the pH of the concrete, and therefore the passive film around the rebar will be lost, causing the initiation of corrosion [7]. Table 1 shows the effect of lowering the pH level in the concrete and the state of corrosion in the reinforcement.

Table 1: State of reinforcement corrosion at various pH levels [5].

pH. level of Concrete	State of reinforcement corrosion
Below 9.5	Initiation of steel corrosion
At 8.0	Passive film on the steel surface disappears
Below 7	Catastrophic corrosion occurs

This carbonation concept is presented in concrete exposed to different environments such as bridges and structures underwater. A study by Ngala et al. finds that there is a reduction in the total porosity and redistribution of pore sizes as a result of carbonation [8]. This can affect the diffusion of chloride in concrete through changing the pore structure of concrete. In general, if the pH level reaches a low value, active corrosion of rebar takes place.

Chloride Attack

A literature search has shown that chloride-induced corrosion can have an extensively damaging effect on reinforced concrete structures. Chloride maybe introduced to the concrete in its initial mixing state based on the type of aggregate or water used in the initial composition of concrete. Also, admixtures that are used sometimes in concrete mixing contain a significant amount of chloride [4]. Additionally, chloride ions can be diffused into concrete in ways such as in the use of de-icing salts on many bridges in the United States during the winter. In general, the rate of corrosion increases with the increase of chloride content. Table 2 shows the risk of corrosion in both carbonated and non-carbonated concrete containing chlorides [9].

Table 2: Corrosion risk due to chlorides

Total chloride (wt% of cement)	Condition of concrete adjacent to reinforcement	Corrosion risk
Less than 0.4%	Carbonated	High
	Un-carbonated	Moderate
0.4% - 1.0%	Carbonated	High
	Un-carbonated	High
More than 1.0%	All cases	High

Other External Factors

In addition to these factors, relative humidity and temperature play a significant role in the corrosion process. A study by Hussain [10] reports that it is important to mention that the rate of chloride-induced corrosion and the process of carbonation are influenced by temperature and humidity, which may vary from one place to another. High humidity and high temperature are often found in gulf marine environments, which is a very serious threat for the durability of reinforced concrete structures. Also, the increase in temperature leads to the increase in the rate of all these processes, and consequently an increase in corrosion rate.

Additionally, in areas where there is extreme heat such as in industrial plants, the concrete cover may develop thermal cracks [10]. Cracked concrete structures are exposed to the surrounding environmental conditions, after which the process of corrosion starts. Similarly, in cold regions, the moisture in the pores of concrete freezes and may expand. This results in the development of cracks, which will lead to corrosion of reinforcement under the previously described conditions. It is important to note that the corrosion of steel in concrete is not determined by a single factor, which makes studying the influence of these factors complicated.

2.1.3.2.   Internal Factors (Concrete Quality Parameters)

Concrete Properties

Concrete properties including the composition of the concrete mix, water/cement ratio, type of cement used, workability, curing, and the quality control at construction site are all factors that affect the permeability of concrete. Higher porosity and large pore sizes lead to severe corrosion damage in reinforcement. Chlorides, water, and oxygen can get inside the pores. Thus, permeability directly affects the rate of corrosion. The porosity of concrete and its pore size distribution is dependent on the water/cement (w/c) ratio in the concrete. Low water/cement ratio decreases the permeability, which in turn reduces the chloride and carbonation penetration and oxygen diffusion in concrete.   In the same study by Kumar et al [9], it is observed that the permeability of hardened cement paste is increased 100 fold by increasing the w/c ratio from 0.35 to 0.45.

It is also reported that cement containing fly ash and silica fume has improved durability in the marine environment [9]. The incorporation of silica fume in concrete mix reduces water absorption and permeability. Thus, the chloride diffusion and water penetration become more difficult. In addition, many studies reported other factors that cause serious corrosion problems such as admixture and impurities in aggregates. If these factors could be well controlled, the corrosion performance of reinforced structures would be much improved.

Concrete Cover

The thickness of the concrete cover surrounding the reinforcement has a remarkable effect on the rebar corrosion due to penetration of chloride or carbonation. A study by the National Association of Corrosion Engineers (NACE) [11] reports the risk of reinforcement

corrosion with low cover thickness. However, once the corrosion starts, the rate of corrosion is independent of the cover thickness as shown in Figure 3. The service life of reinforced concrete structures can be extended greatly by simply increasing the thickness of the concrete cover.
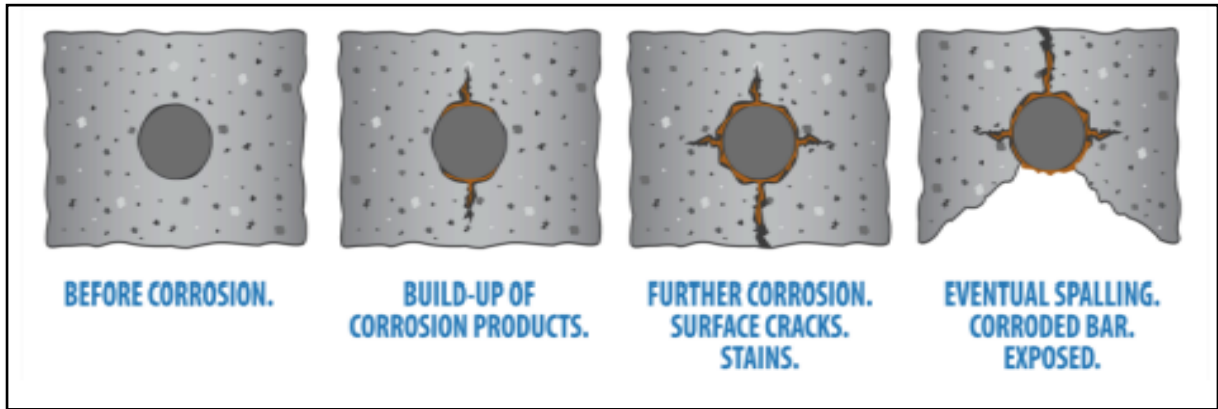
Figure 3: Progress of corrosion in concrete and eventual spalling [3]

2.1.4. Structural Effect and Damage Due to Corrosion

The cost of repairing or replacing deteriorated structures due to corrosion has become a major obligation and liability to clients. In a study by the U.S Federal Highway Administration in cooperation with the National Association of Corrosion Engineers (NACE), it was estimated that the direct cost of corrosion is $ 276 billion dollars on an annual basis. These costs include the cost of corrosion- control methods, equipment, repair, etc [11]. Considerable resources have to be allocated to restoring and extending the service life of deteriorating RC structures. In addition to the monetary costs, corrosion can cause catastrophic failures of structures. Other costs, such as loss in serviceability, the reduction of steel cross sections, cracking, etc, are equally significant.

As mentioned before, corrosion of reinforcement is the principle cause of deterioration of reinforced concrete members. In a structural journal by Du et al. [10], it is stated that

deterioration affects the stiffness and the strength of the structure, in addition to the rust stains and cracks that will be present on the structure. Corrosion may also affect the residual strength, such as when the reduction is on the concrete cross section due to corrosion induced cracking and spalling, loss of bond strength, and most importantly the loss of reinforcement.

Generally, the deterioration of reinforced concrete structures can be defined as a two-phase process: initiation and propagation. The deterioration of reinforced concrete versus time is illustrated in Figure 4.
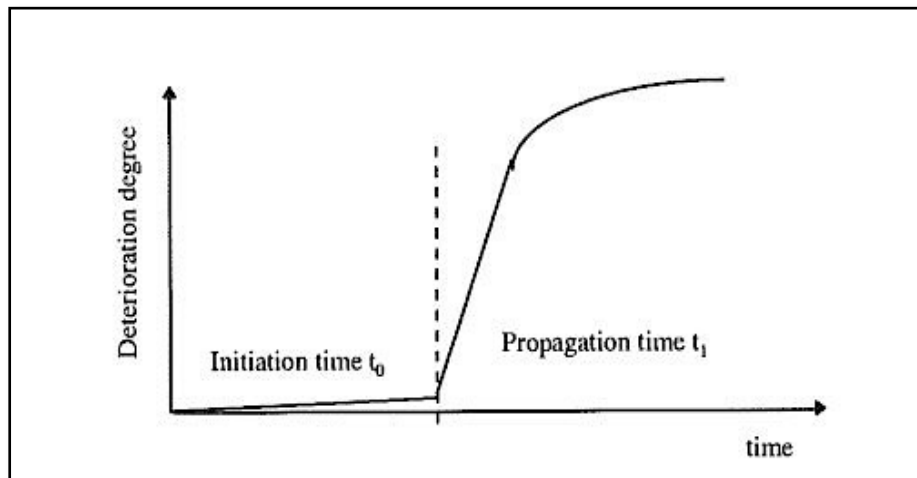
Figure 4: Deterioration degree of reinforced concrete vs. time [11].

As shown in Figure 4, the initiation period represents the time required for CO_2 or chloride ion to diffuse to the steel and activate corrosion. The propagation period represents the time between corrosion initiation and corrosion cracking. If the corrosion cracking can be delayed or prevented, structural strength is maintained for a longer period.

According to the same study done by NACE, the most commonly observed deterioration failure modes are: the rupture of the bottom tensile reinforcement; concrete crushing, shear, or by shear combined with anchorage failure of tensile bars depending on the location and level of

corrosion [9]. In general, the bending moment strength was found to decrease due to (1) reduced area of tensile bars; (2) reduced bond strength between bars and concrete, especially after the formation of longitudinal cracks along the bars; (3) flexural concrete crushing caused by the concentrated vertical cracks.

2.1.5. Steel Reinforcement Cross Section

Reinforced concrete uses steel to provide the tensile properties that are needed in structural concrete. It prevents failure of concrete structures that are subjected to tensile and flexural stress due to dead and live loads, wind, snow, or traffic. However, when the reinforcement corrodes, the formation of rust will cause a loss in the bond between the steel and the concrete resulting in delamination and spalling. Al-saidy [14] concludes in his study that as steel corrodes, there is a corresponding loss in cross sectional area and as a result, a reduction in the flexural strength capacity of concrete as shown in Figure 5.

Figure 5: Corroded steel bar in comparison with noncorroded steel bar [14]

Other experimental studies reported in the literature show that the rust occupies a volume of up to twelve times greater than the volume of the original steel rebar [15]. The formation of

rust will reduce the cross sectional area of steel, which will reduce the rebar flexural and shear strength capacity.

2.1.6. Steel Reinforcement Tensile Properties

The experimental study by Almusallam [3] also notes a decrease in the tensile strength of steel bars with an increasing degree of reinforcement corrosion utilizing the actual area of cross-section. Table 3 summarizes the experimental tensile strength data for 6 mm (0.23 in.) diameter bars with varying degrees of reinforcement corrosion. The experimental results data indicated that the actual load carried by the bars decreased with an increasing level of reinforcement corrosion.

Table 3: Tensile strength of 6mm (0.23 in.) diameter steel bars [3]

Specimen #	Corrosion (%)	Average Diameter (mm)	Ultimate Load (kN)
C1	0	5.9	21.76
C2	0.88	5.85	21.01
B2	1.1	5.8	20.49
A1	1.45	5.81	20.53
A2	1.45	5.89	21.09
D1	1.63	5.85	20.762
B3	11.64	5.25	16.521
G1	17.83	4.95	13.05
G2	19.4	4.95	15.03
D1	24.95	4.3	10.79
H2	32.02	3.9	9.266
A2	40.7	4.1	10.156
S2	48.25	4.1	10.134
S3	75	3	4.877

2.1.7.   Bond Between Concrete and Steel Reinforcement

The concrete-steel bond is responsible for the rebar anchorage in the RC member. The rust formed by the accumulated corrosion products on the rebar surface may reduce the friction component of the bond strength. According to the previous study by Sulaimani et al. [17], corrosion causes an initial increase in rebar to concrete bond strength due to the increased rebar surface roughness, but further corrosion results in a loss in bond strength. That loss is explained by the deterioration of the rebar ribs of the deformed rebars causing a significant reduction of the interlocking forces between the ribs of the rebars and the surrounding concrete.

2.1.8.   Corrosion Induced Cracks

The accumulated corrosion products on the bar surface cause longitudinal cracking of the concrete cover. Loss of concrete cover implies a loss of confinement and a reduction in bond strength at the interfacial zone between the steel and concrete. As a result, the bond strength is significantly reduced and becomes negligible. A number of researchers have attempted to study the corrosion-induced crack width and corrosion crack patterns using accelerated corrosion techniques. A study by Badawi and Soudki [18], investigated different corrosion configuration in eight specimens at three different degrees of corrosion (5%, 10%, and 15%). One of their conclusions was that corrosion-induced crack width increases with time as corrosion activity progresses. A larger crack width is presented at a higher rate of corrosion assuming uniform corrosion as shown in Figure 6.
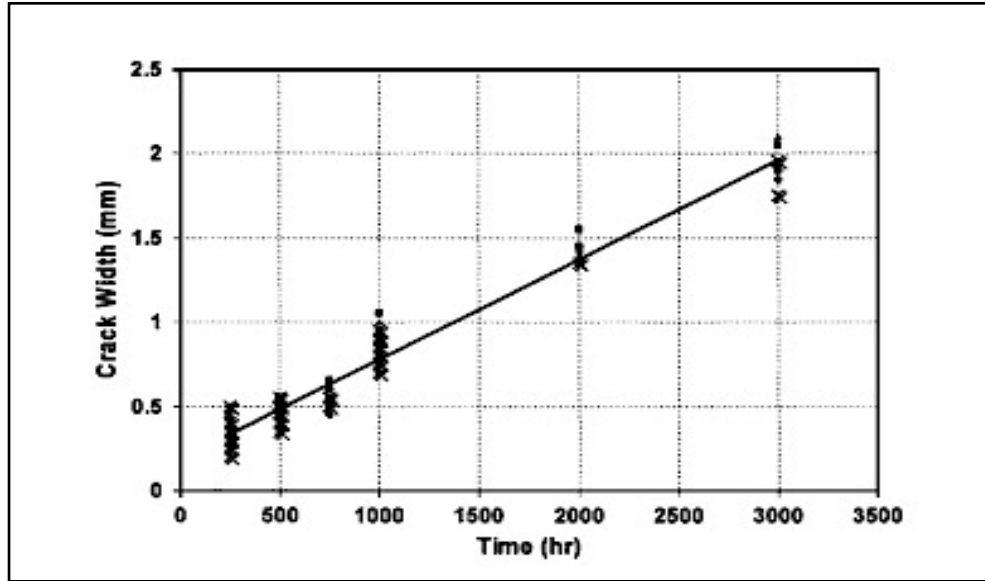
Figure 6: Corrosion crack width vs. time [16]

Also, degradation of the concrete through cracking can eventually lead to the concrete falling away from the structure. This more extensive form of cracking is known as spalling. Cracking and spalling are signs of degradation that can be observed by the naked eye. These warning signs, however, are advanced phases of corrosion damage. Once the concrete begins to spall away from the structure, the structural integrity of the concrete member will be compromised.

Additionally, reinforcement corrosion may have other effects on the concrete member. For instance, an experimental program done by Al-Saidy and Al-Jabri [19] tested rectangular reinforced concrete specimens after they were exposed to accelerated corrosion. The corrosion rate varied between 5% to 10%. The corroded beams showed lower stiffness and strength than non-corroded beam (control specimen). Corrosion will cause a loss of stiffness in a concrete member, which will experience greater cracking. Also, as the stiffness decreases due to the corrosion in a concrete member, the member will exhibit higher deflection. This situation may lead to serviceability failure of the structure. Moreover, since corrosion happens non-uniformly,

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the structure may become unsymmetrical after deterioration. Cross-sectional asymmetry leads to large eccentricity and moment on the deteriorated columns [17].

2.1.9. Rehabilitation Techniques

In recent years repair and retrofit of existing structures such as buildings, bridges, etc., have been among the most important challenges in civil engineering. The primary reason for strengthening structures includes upgrading structural resistance to withstand underestimated loads, increasing the load carrying capacity for higher permit loads, eliminating premature failure due to inadequate detailing, and restoring the lost load carrying capacity due to corrosion or other types of degradation caused by aging, etc.

Several rehabilitation techniques for concrete members have been identified during the last three decades. For instance, concrete members have been repaired by jacketing them with new concrete in conjunction with epoxy-bonded steel plates. However, steel plates have a durability problem because of their vulnerability to corrosion. This adversely affects the bond at the steel plate/concrete interface. Special heavy equipment is also needed to install these heavy plates. As a result, alternative innovative materials have been sought by structural engineers.

During the development of advanced materials in the 1990s, the use of fiber reinforced polymer (FRP) sheets as material to strengthen structural members was becoming more popular due to the high corrosive resistance and high strength to weight ratio. Strengthening with FRP has shown applicability to many kinds of structures. Currently, this method has been applied to strengthen such structures such as columns, beams, walls, slabs, etc. Many studies and research programs have been conducted to investigate this innovative method to enhance the performance of deteriorated RC members.

A laboratory study carried out by Badawi and Soudki [18], which included sixteen small-scale reinforced concrete beams (100 x150x 1200 mm) and twenty large-scale beams (152 x254 x 3200 mm). The specimens were exposed to different accelerated corrosion levels (5%, 10%, and 15%). Different CFRP strengthening schemes (directions and shapes) were used on the small and the large beams. The authors reported that all strengthened beams exhibited increased stiffness over un-strengthened specimens and a marked increase in the yield and ultimate strength.

Moreover, Bonacci and Maalej [20] carried out an experimental program to provide a realistic assessment of the potential use of CFRP materials in the repair and strengthening of reinforced concrete flexural members exposed to a corrosive environment. Seven specimens (270 x 400 x 4350 mm) were tested. Four of the seven RC beams were reinforced externally with one or two layers of CFRP composite, and were tested under sustained and monotonic loading. CFRP external reinforcement increased beam load carrying capacities by 10–35% and reduced deflection by 10–32% with respect to the control specimen. The results showed that the use of CFRP sheets for strengthening corroded reinforced concrete beams was an efficient technique that could maintain structural integrity and enhance the behavior of such beams.

Additionally, the use of near surface mounted (NSM) fiber reinforced polymer (FRP) rods to strengthen RC beams has been recognized as a promising technology for increasing flexural and shears strength of deficient RC members. A study by Nurbaiah et al. [21] reported that the percentage of stiffness increase was 55% to 85% for beams strengthened with NSM glass fiber reinforced polymer (GFRP) bars, and that they mostly failed in flexure after the longitudinal steel reinforcement yielded. With the limited number of studies of corroded RC beams strengthened with FRP, there is a need for further investigation.

2.2.   Carbon Fiber Reinforced Polymer (CFRP)

This section of the report is an introduction to fiber reinforced polymer (FRP) materials as an external reinforcement to strengthen existing structures. More specifically, the review will only cover one type of fibers: carbon fiber reinforced polymer (CFRP). The use of CFRP as reinforcement for strengthening and repairing structural members, as well as the advantages and disadvantages of this technique, will be discussed briefly.

2.2.1. Overview

The term fiber reinforced plastic/polymer (FRP) describes a group of advanced composite materials composed of synthetic or organic fibers embedded in a resin. In advanced composite materials, the fibers are oriented at high volume fractions in the directions of significant stress in order to maximize the utility of the fibers. The most common FRPs consist of continuous fibers of glass, aramid, or carbons embedded in a polymer resin matrix such as polyester or epoxy and are called carbon FRP (CFRP), aramid (AFRP), and glass FRP (GFRP).

In recent years, there has been a surge of activities in the civil engineering research community to test and demonstrate the viability of these new materials for the construction of more durable structures, and for the repair and rehabilitation of existing structures. Many creative applications of fiber composites have been developed by researchers around the world, such as reinforcing and re-stressing concrete structures, seismic retrofitting of concrete and unreinforced masonry structures, and strengthening of buildings, bridges, and etc. The efforts of these researchers have resulted in many successful demonstration projects. Therefore, the use of FRP for externally bonded reinforcement (EB-FRP) to rehabilitate and strengthen existing structures and materials of RC elements is becoming a widely accepted practice [22]. Of the three types of FRP, mentioned here, CFRP has the highest tensile properties.

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Over the past few years, external strengthening using CFRP composites gained popularity over steel for several reasons, including material cost, lower weight, corrosion resistance, and ease of application (Figure7).
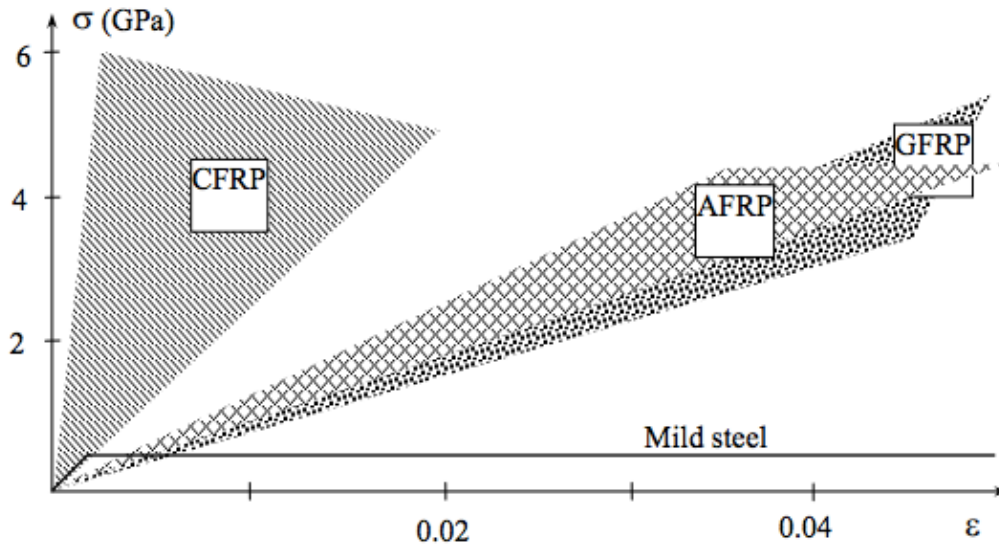
Figure 7: Stress-strain diagrams for different unidirectional FRPs and steel [22]

If the service life of a structure is shorter than anticipated, investments related to maintaining the structure can be justified. The maintenance can be categorized into two types, repair (retrofit) and strengthening (upgrading) of a certain structure [23]. Strengthening with CFRP sheets has shown to be a beneficial alternative to structural elements that have had a change in function. It has been shown from past studies that CFRP sheets can be used to enhance the capacity of both flexural and shear. Table 4 presents some material data for the most common materials.

Table 4: Mechanical properties of common strengthening materials [2]

	Modulus of elasticity [GPa]	Compressive strength [MPa]	Tensile strength [MPa]	Density [kg/m^3]
Concrete	20-40	5-60	1-3	2400
Steel	200-210	240-690	240-690	7800
Carbon fiber	200-800	NA	2500-6000	1750-1950

2.2.2. Application of CFRP

For structural applications, CFRP is mainly used in two areas. The first application is the use of CFRP bars instead of steel reinforcing bars or pre-stressing strands in concrete structures. The second, and the more common, method of strengthening deficient RC members is by adhesive bonding thin, prefabricated sheets or strips of composite laminates known otherwise as CFRP sheets/strips to the surfaces of RC beams or slabs to increase their capacity [24]. This method has been established around the world as an effective method applicable to many types of concrete structural elements. The performance of these strips depends on several variables: the bonding strength of the adhesives used, the state of stress at the interface of the concrete and the FRP strips; the failure modes of the concrete; methods of curing; and the material preparations needed [24]. These factors, among a host of other considerations, form the bases for design and safety concepts.

2.2.3. CFRP Advantages and Disadvantages

The advantages and disadvantages of FRP materials are summarized and listed in Tables 5 and 6. The tables as presented are a collection of relevant points from sources [22, 31].

Table 5: Advantages of CFRP

Advantages of CFRP
▪ High ultimate strength (2-3 times greater than steel)
▪ Lower density than steel
▪ Strength to weight ratio is higher than for steel
▪ Requires little maintenance
▪ Excellent durability
▪ Excellent corrosion resistance
▪ Good flexibility
▪ Handling and installation is significantly easier than for steel

Table 6: Disadvantages of CFRP

Disadvantages of CFRP
▪ High cost
▪ Long-term durability is not yet available
▪ The transverse strength is low

2.2.4. Failure Modes

Tests on reinforced concrete beams with CFRP sheets bonded to the tension face showed that although the CFRP reinforcement was effective in enhancing both stiffness and strength, catastrophic failures occurred when the beam load capacities were reached [25]. Failure of CFRP strengthened beams may occur by either CFRP rupture, steel yield and CFRP rupture, concrete

compression failure, shear failure, delamination of CFRP, or debonding of the composite attachment.

The debonding of an externally bonded CFRP sheet/ strip can be predicted by considering the different bond failure modes, which can occur under any of the following occurrences: bond-critical failure modes (end debonding or intermediate crack debonding); cohesive and adhesive strengths of the concrete; ultimate strength for end debonding (concrete rip-off); ultimate strength for intermediate rip-off; and, interfacial stresses for the serviceability limit state [22] as shown in Figure 8. Failure in the case of RC beams may take place through concrete crushing before yielding of the reinforcing steel, steel yielding followed by FRP rapture; steel yielding followed by concrete crushing, cover delamination; and/or FRP debonding.
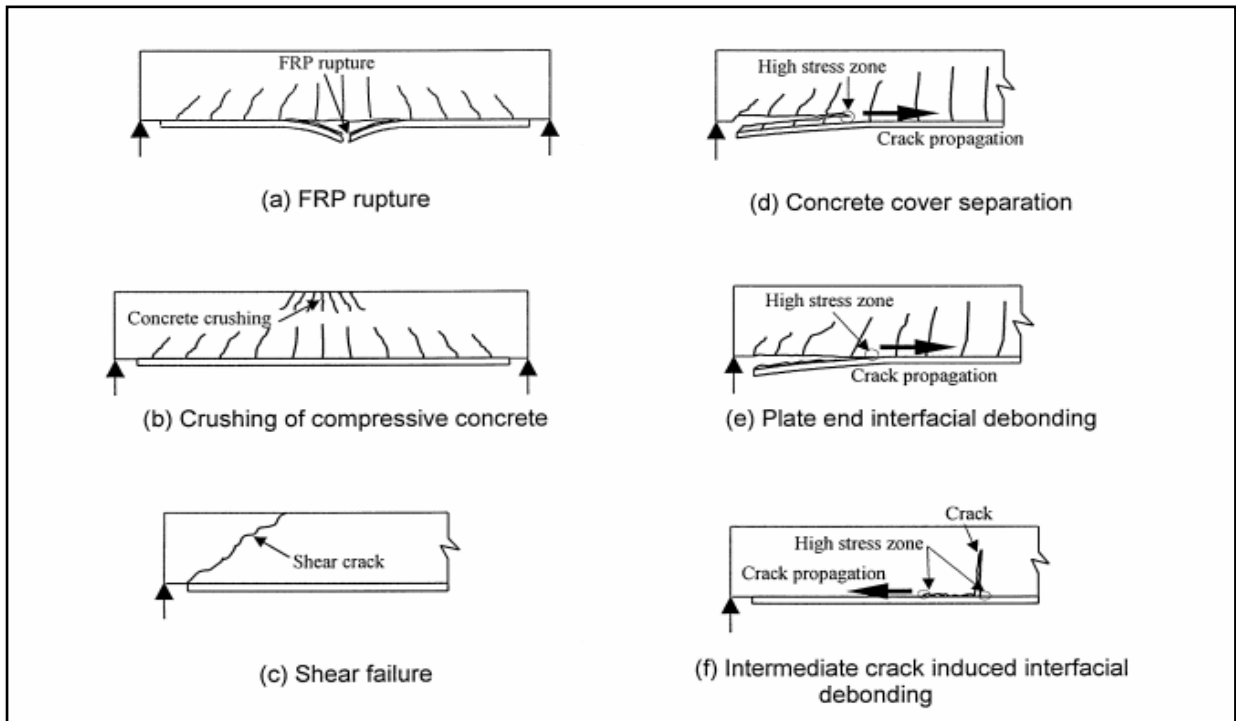
Figure 8: Failure modes of FRP wrapped RC beams

2.2.5. Experimental Investigation

Balamuralikrishnan et al [26] conducted an experimental study on beams to evaluate the performance of RC beams bonded with single and double layer CFRP fabric at the soffit of the beam under static and cyclic loading. A total of ten RC beams, all having a size of 6in. x10in. x125 in., were cast and tested over an effective span of 3000 mm up to failure. The beams were designed as under-reinforced concrete beams. The authors concluded that CFRP fabric properly bonded to the tension face of RC beams can enhance the flexural strength substantially. The strengthened beams exhibited an increase in flexural strength of 18% to 20% for a single layer and 40% to 45% percent for two layers, during both static and compression cyclic loading. In general, the strengthened beams exhibited increased flexural strength, enhanced flexural stiffness, and composite action until failure.

AI-Ham et al [27] investigated the effect of a mild level of corrosion of steel reinforced concrete on flexural and bond fatigue strength under repeated loading. This investigation was carried out on thirty beams of sized at 6in. x10in. x79in. To attain the required level of corrosion within a reasonable time an accelerated corrosion technique was used. Results showed that a mild level of corrosion (5% mass loss) caused on average 10% and 20% reduction in flexural and bond fatigue strength, respectively. The effect of the addition of carbon fiber reinforced polymer (CFRP) sheets on the fatigue life of corroded RC beams was also assessed. The authors reported that repairing with CFRP sheets increased the fatigue capacity of the beams with corroded steel reinforcement beyond that of the control unrepaired beams with non-corroded steel reinforcement.

EI Maaddawy et al [28] presented results of an experimental study designed to evaluate the performance of severely corroded reinforced concrete beams repaired with carbon fiber

reinforced polymer (CFRP) sheets. Eight RC T-beam specimens were constructed and tested to failure under four-point load configurations. Seven beams were pre-subjected to accelerated corrosion for five months that corresponded to an average tensile steel mass loss of 22%. The authors found that corrosion damage significantly reduced the flexural capacity and ductility of the unrepaired beam. Also, they concluded that the carbon fiber reinforced polymer (CFRP) system fully restored the capacity of the corroded beams.

Shihy et al [29] reported that strengthening composite beams and concrete slabs strengthened with CFRP sheets increased the load carrying capacity of the beam by 15%. This increase was related to the thickness of the CFRP sheet; doubling the sheet thickness increased the ultimate capacity of the beams to 21%. The load carrying capacity of the strengthened beams with corrugated sheet predicted by the experimental data is higher than that of the control beams by 12%. The ductility of the strengthened beams had a range of 2.4 to 2.5, compared to 3.5 for the control beam. The low ductility of strengthened beam indicates that the addition of CFRP as reinforcement greatly reduced the deforming ability at the ultimate stage of loading.

Obaidat et al [3] presented the results of the experimental study conducted to investigate the behavior of structurally damaged full-scale reinforced concrete beams retrofitted with CFRP laminates in shear or flexure. The main variables considered were the internal reinforcement ratio, position of retrofitting, and the length of CFRP. The stiffness of the CFRP-retrofitted beams increased compared to that of the control beams. Employing externally bonded CFRP plates resulted in an increase in maximum load. The increase in the maximum load of the retrofitted specimens reached values of about 23% for retrofitting in shear and between 7% and 33% for retrofitting in flexure. Moreover, retrofitting shifted the mode of failure to brittle

behavior. The results showed that the main failure mode was plate debonding, which reduced the efficiency of retrofitting.

AI-Hammoud et al [31] investigated the flexural behavior of thirty, 6in. x10in. x79in. corroded steel reinforcement beams repaired with CFRP sheets under repeated loading. They concluded that, repairing with a double flexural CFRP sheet at a high corrosion level increased the flexural fatigue capacity of corroded beams by 42% at 50000 cycles and 17% at 750000 cycles compared to the corroded beams. Further, they found that there was no difference in strength between repairing the beams with a single layer and a double layer of CFRP sheets. When severely cracked beams were repaired with FRP, their life was extended by about ten times, suggesting that beams in service could be effectively rehabilitated using FRP. High-modulus FRP sheets have excellent tensile and fatigue strength properties but little global ductility.

CHAPTER 3: **EXPERIMENTAL WORK**

3.1.   Beam Design

To meet the goals of this study, five one-third-scale simply supported beams specimens were designed and tested. The scale of the specimen was selected to accommodate the limitations of laboratory space, instrumentation, and access to rebars with characteristics similar to those for the full-scale test. The beams were designed and analyzed in compliance with the specifications given by the American Concrete Institute ACI 318-14 and ACI 440.2R-08. All five beams have the same cross section of 4 in. x 6 in and span length of 6 feet, and were tested under third-point loading.

3.1.1. Description of Beam Specimens

Beam No.	Description
Beam #1	Deteriorated
Beam #2	Plain concrete beam
Beam #3	Control RC beam, un-corroded
Beam #4	Deteriorated beam + One layer of CFRP
Beam #5	Deteriorated beam +Two layers of CFRP

Beam #1 is used to represent an extremely corroded and deteriorated concrete beam. The deterioration was represented by pulling out the temporary reinforcement shortly after the beams were set to cure. The voids represent the loss in bond to the concrete and the loss in steel cross section. Figure 9 shows the mold for Beam #1.

Figure 9: Deteriorated beam mold

The purpose of Beam #2 is to compare the load carrying capacity of a plain concrete beam to a deteriorated concrete beam (Beam #1). Figure 10 shows the mold for Beam #1.

Figure 10: Plain concrete mold

The mold and reinforcements of control (un-corroded) Beam #3 are shown in Figure 11. Beams #4 and #5 are the same as Beam #3 and are externally bonded and wrapped with one and two layers of CFRP sheets.

Figure 11: Reinforced concrete beam mold

3.2. Beam Construction

3.2.1. Construction materials

The fabrication of the beams formwork as well as mixing the concrete was done at the South Green House at Portland State University. The concrete was cast and cured outside the South Green House with the help of other undergraduate and graduate students. The flexural and shear reinforcement of the control beam (Beam #3) consisted of 2 - #3 longitudinal steel bars and #9-gauge wire stirrups. The experimental steel yield stresses (f_y) were determined in a previous experiments done by a graduate student colleague and are shown in Table 7.

Table 7: Steel reinforcement yield strength

Steel bas size	Yield strength, f_y (ksi)
#3 bars	74
#9 gage wire stirrups	30

The formwork consisted of plywood to provide good finishing of substrates. Two beams were cast at a time, and ten cylinders 6in. x 12in. were retained as samples for compressive strength testing (Figure12). The concrete was mixed using a small rotary mixer and shoveled into the formwork, and a steel rod was used to minimize air voids in concrete members as in Figure 13.

Figure 12: Concrete beams and cylinders

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Figure 13: Mixing concrete on sit

3.2.2. Concrete Properties

In order to maintain general applicability of these results, a typical unit weight and concrete compressive strength of 3000 psi were used. Experimental compressive strengths were obtained by testing standard 6x12 inch cylinders and standard flexural tests for each concrete batch as presented in Table 8. The concrete compressive strengths at testing ages were slightly higher than the mix design target strength. The average compressive strength and modulus of rupture are shown in Table 9.

Table 8: Concrete compressive strength

Test Date	Compressive strength f'_c (psi)
11-Mar	2476
16-Mar	3042
15-Apr	3679
20-Apr	3136
6-May	3767
Average	3220

Table 9: Concrete properties (average values)

Compressive strength f'_c (psi)	Modulus of rupture f_r (psi)	Modulus of elasticity E_c (psi)
3220.0	429.9	$3.3*10^6$

3.2.3. Casting of Beams

A single beam was fully reinforced in shear and flexure. The steel reinforcements were cut on-site to the required length and assembled in the cage ready for concrete casting. Plastic spacers were used in formwork as well as corner chamfers to provide beam specimens typical of those used in construction. For the deteriorated beams (Beams #2, #4, and #5), small sizes of reinforcements were cut and inserted after they were greased in the formwork as shown in Figure14. All five beams were cast and cured under similar conditions. After six hours of concrete curing, reinforcements were safely pulled out of three beams (#2, #4, and #5) leaving voids. The voids represent the area of the steel reinforcement after it has been completely corroded (Figure 15, 16 & 17).

Figure 14: Temporary reinforcement in deteriorated Beam #4 &5

Figure 15: Shear voids in the deteriorated beam

Figure 16: Voids representing deteriorated flexural reinforcement

Figure 17: Voids representing deteriorated shear reinforcement

3.2.4. Composite Material

The type of composite material used in this project is a unidirectional carbon fiber reinforced fabric. It is composed of a dense network of high strength carbon fibers held in a unidirectional alignment with a light thermoplastic glass fiber cross weave yarn. The properties of the used CFRP material are shown in Table 10. The bonding agent used was MasterBrace SAT 4500. After the beams were wrapped, they were cured for a minimum of one week prior to testing.

Table 10: CFRP tensile and physical properties

Property	Requirement
Ultimate Tensile Strength, f^*_{fu}	550 ksi [3800 MPa]
Tensile Modulus, E_f	33000 ksi [227 GPa]
Ultimate Rupture Strain, ε^*_{fu}	1.67%
Nominal Thickness, t_f	0.0065 in/ply[0.165 mm/ply]
Fiber Tensile Strength	720 ksi (4950 MPa)

3.2.5. Application of Carbon Fiber Reinforced Polymer

Special consideration was given to surface preparation before bonding the CFRP sheets to the concrete surface. Sandblasting was employed to remove the weak layer from the surface of the beam, and then the surface was cleaned with a high-pressure air jet. The beam corners were grinded and smoothed per ACI 440.2R-08 as in Figure18.

Figure 18: Beam surface and corner preparation

Strips of carbon fiber reinforced polymer sheets (CFRP) were cut to the proper dimension and bonded to the tension side (longitudinally) over the length of Beam # 4 and Beam #5. The strips were extended few inches at both ends as well as along the sides of the beams to reduce the risk of de-bonding failure. Also, a continuous sheet of CFRP was wrapped around the entire cross section of the beams. The deteriorated beams (Beam #4 &5) were wrapped with CFRP around the circumferential of the beams first, and then were strengthened in tension. The layout of CFRP strips and CFRP wrap will typically be as indicated in the application procedure. Figure 19 presents a better view for the actual application.

Figure 19: CFRP wrapped Beam #5

3.3.   Test Setup and Data Collection

All beams were tested to failure under two-point loading as shown in Figure 20. Prior to testing, beams were checked dimensionally, and a detailed visual inspection made with all information carefully recorded. The load was applied to the concrete beams though a steel plate and half rollers with a flat side. The rollers consisted of 1 in. radius steel rods on a flat steel plat that extended across the entire width of the beams. All load points in contact with concrete surfaces were distributed with steel plates to avoid stress concentration problems. Since the non-reinforced beams have a low tensile strength, the load was manually applied at a constant rate. A strengthened modulus beam in the testing machine is presented in Figure 21. Two channels of data were collected during the tests, the applied load and centerline displacement (measured with a National Instrument Data Logger), Figure 22.

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Figure 20: Testing beams suing two-point loading

Figure 21 : Retrofeted beam test setup in the Greenhouse lab.

Figure 22: Data logger

Figure 23: Beam test set-up
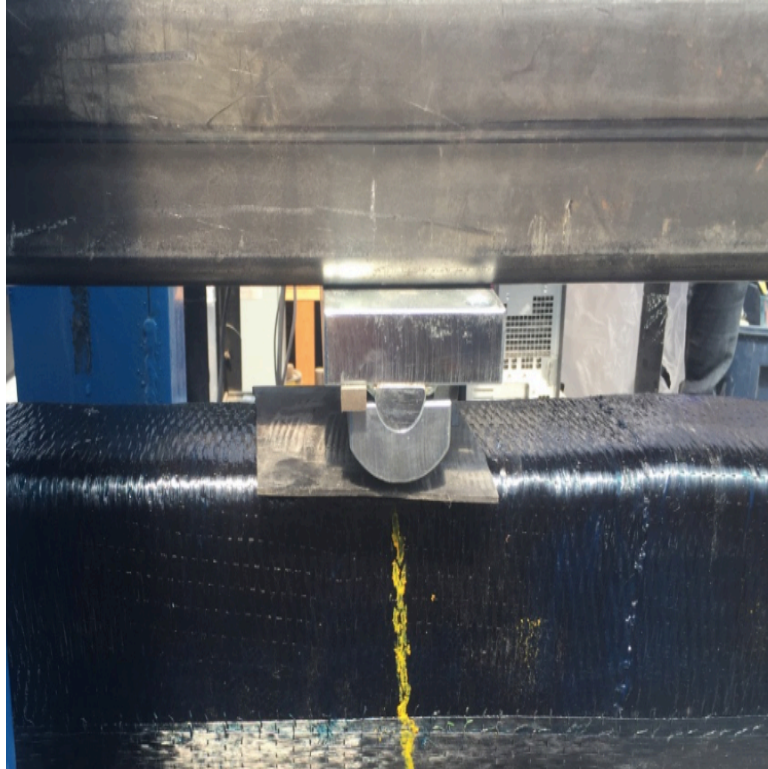
Figure 24: Point load

Figure 25: Testing CFRP-wrapped beam

CHAPTER 4: **EXPERIMENTAL RESULTS**

This chapter will summarize the experiment results in terms of ultimate applied load, maximum deflection, and failure modes for each beam. In addition to that, a comparison between all beam capacities and contribution of carbon fiber reinforced polymer sheets will be presented in this section.

4.1. Summary of Results

4.1.1. Beam #1 (Deteriorated Concrete Beam)

The deteriorated concrete beam experienced a classic brittle failure of concrete loaded in bending. The cracks observed on the concrete specimen had a high speed of propagation, then a sudden rupture of the specimen. This brittle failure is due to the fragility of concrete and the low tensile strength developed in the tension zone of the element. Figure 26 shows the specimen after failure. From the collected data, the maximum load capacity of the beam is 0.27 kips. The deflection corresponding to this load was 0.0071 in.

4.1.2. Beam #2 (Plain Concrete Beam)

This beam experienced a very similar brittle failure mode as beam #1 as shown in Figure 28. The load capacity was slightly higher than that of Beam #1 due to the full concrete cross section without voids. Beam #2 failed at a higher load of 0.54 kips and it had a slightly lager deflection value of 0.015 in.

Figure 26: Failure mode of beam #1

Figure 27: Beam #1 brittle failure

Figure 28: Beam #2 brittle failure

4.1.3. Beam #3 (RC Beam)

Beam #3 was the control beam with un-corroded reinforcement. This beam was used as a reference for the members strengthened with CFRP materials. The maximum load capacity of the beam was 6.8 kips. The deflection corresponding to this load was 1.55 in. This beam experienced a typical ductile failure mode of reinforced concrete beams. Central cracks propagated starting a load of 3kips and continued until a major failure in the shear zone occurred as expected. Yielding of steel was at a second stage until total crushing took place as shown in Figure 29, 30 & 31.

Figure 29: Concrete crushing in the compression zone

Figure 30: Crushing concrete cover

Figure 31: Yield of steel reinforcement in beam #3

4.1.4. Beam #4 (One Layer of CFRP)

This beam is similar to Beam #1 but with one layer of CFRP applied to the bottom of the beam for flexure, and around the circumference for shear. The maximum load was measured as 7.89 kip, with a maximum deflection of 1.78 in. at failure. The failure mode was a combination of rupture of the carbon fabric in shear and tension sides, and sudden brittle failure due to the lack of steel reinforcement as shown in Figure 32 & 33.
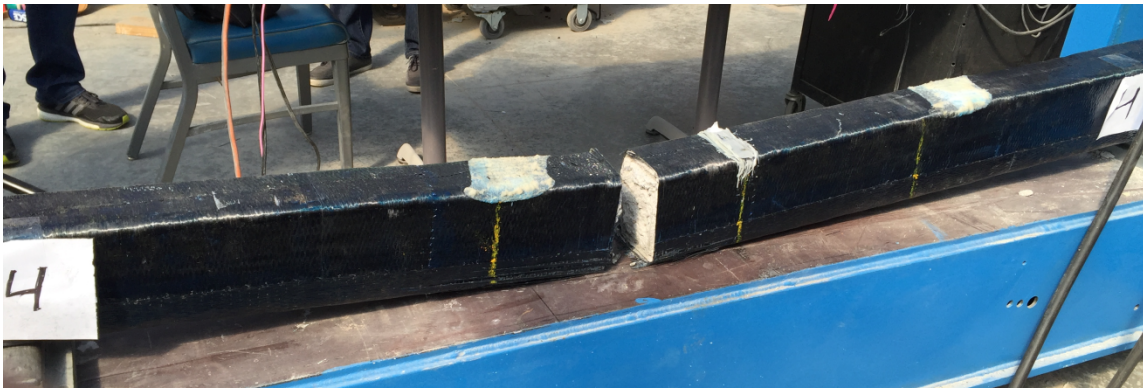
Figure 32: Beam #4 failure mode

Figure 33: Combined rupture and brittle failure

4.1.5. Beam #5 (Two Layers of CFRP)

This beam was the same as Beam #4, however; two layers of CFRP were externally bonded in flexure and shear. Due to the increase of number of layers and CFRP thickness, one can expect the beam capacity to increase at least twice that of Beam #4. Beam failure occurred at a maximum load of 14.58 kip. The measured deflection at failure was also higher at 3.05 in., Figure 34.

Figure 34: Beam #5 deflection capacity

Figure 35: Beam #5 shear failure

Figure 36: Beam failed half span distance

CHAPTER 5: **DISCUSSION OF RESULTS**

5.1.   Failure Load

Table 11 summarizes the maximum loads carried by the tested beams. Beam #5 exhibited the greatest load carrying capacity, which was about 2 times that of the control beam. Comparing the results of Beams #4 and #5 shows that the load-carrying capacity is doubled when a beam is strengthened with two layers of CFRP, both circumferentially and longitudinally covering the entire span. This was expected due to the higher strength and modulus of elasticity of the CFRP sheets used in Beam #4 and #5. Graphical representations of the beams behavior are shown in Figures 37, 38 & 39.

Table 11: Experimental max failure load

Beam No.	Beam	Experimental Failure Load (kip)	Ratio $P_{exp.} / P_{RC}$
1	Deteriorated	0.274	0.040
2	Plain	0.538	0.078
3	RC (Control)	6.89	1.00
4	CFRP (One layer)	7.89	1.15
5	CFRP (Two layers)	14.58	2.12

Furthermore, comparing results of Beams #3 and #4 indicates that both beams experienced similar load carrying capacity, suggesting that using one layer of CFRP sheets or 2 - #3 steel rebars (f_y = 60 ksi) as a strengthening systems leads to the roughly the same load carrying capacity. CFRP helped the deteriorated beam restore its load carrying capacity.
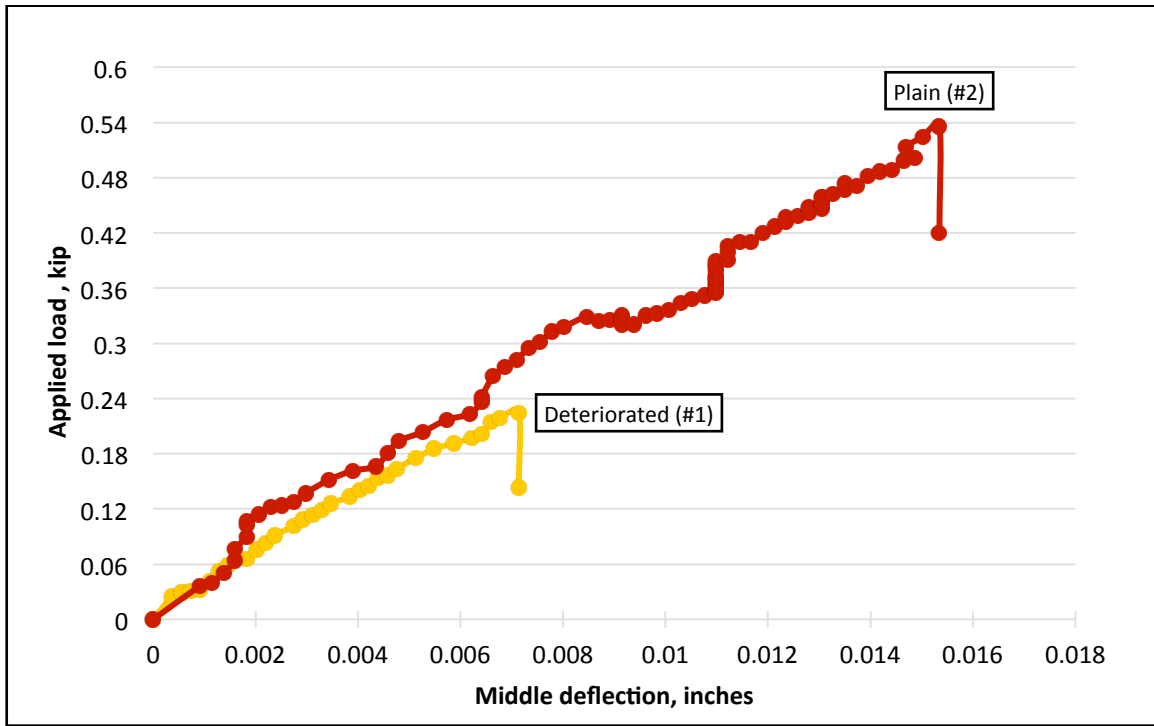
Figure 37: Load-Deflection relationship for un-reinforced beams
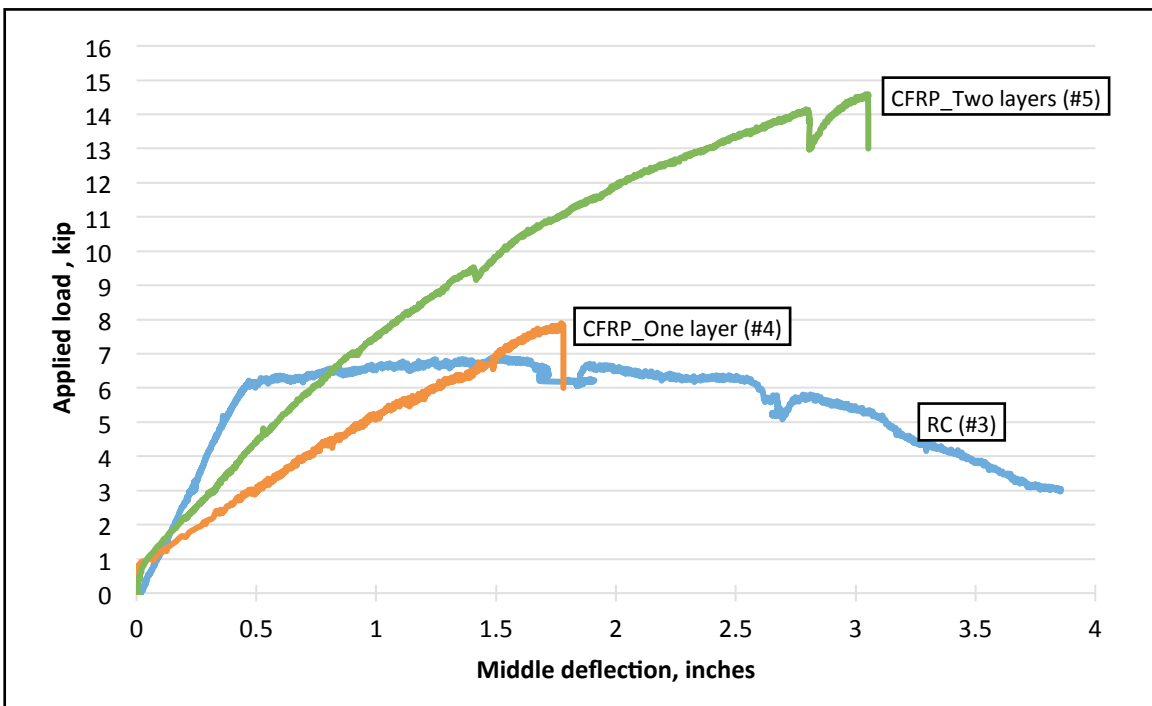
Figure 38: Load-deflection relationship for reinforced and retrofitted beam

Load-Deflection Relationship

CFRP_Two layers (#5)

CFRP_One layer (#4)

RC (#3)

Plain (#2) or
Deteriorated (#1)
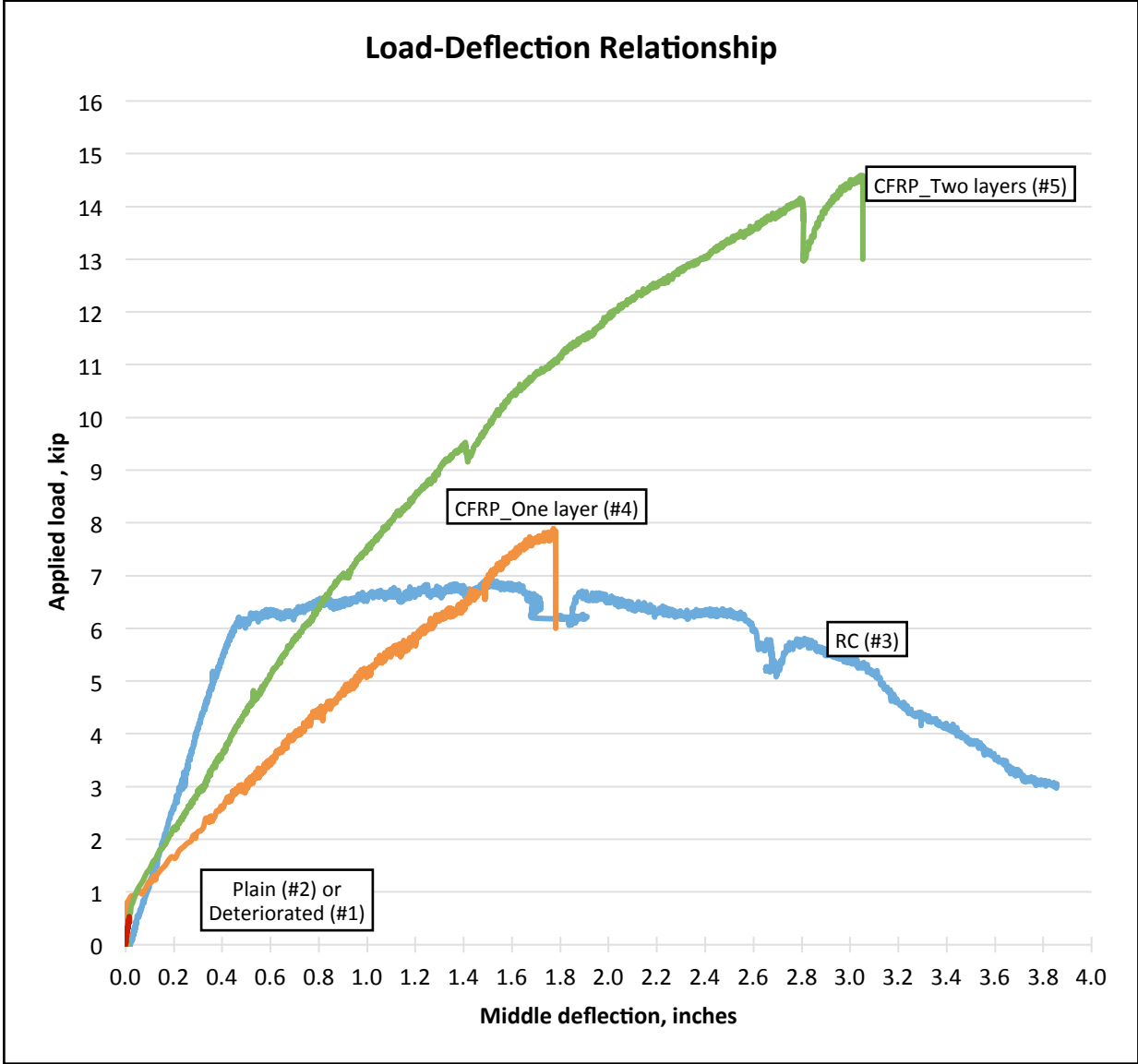
Figure 39: Load-deflection relationship for all beams

Plain and deteriorated beams had a small load carrying capacity relative to the other three beams, which is evidenced by their almost imperceptible load deflection curves (Figure 39). Thus, by comparing the load-deflection relationships for all five beams in Figure 39, it is obvious that strengthening the beams circumferentially and longitudinally with CFRP improved the

beams' load-carrying capacity. Better view of the comparison between all beams is shown in Figure 40. Note that Beam #1 and #2 are shown on a secondary axis.
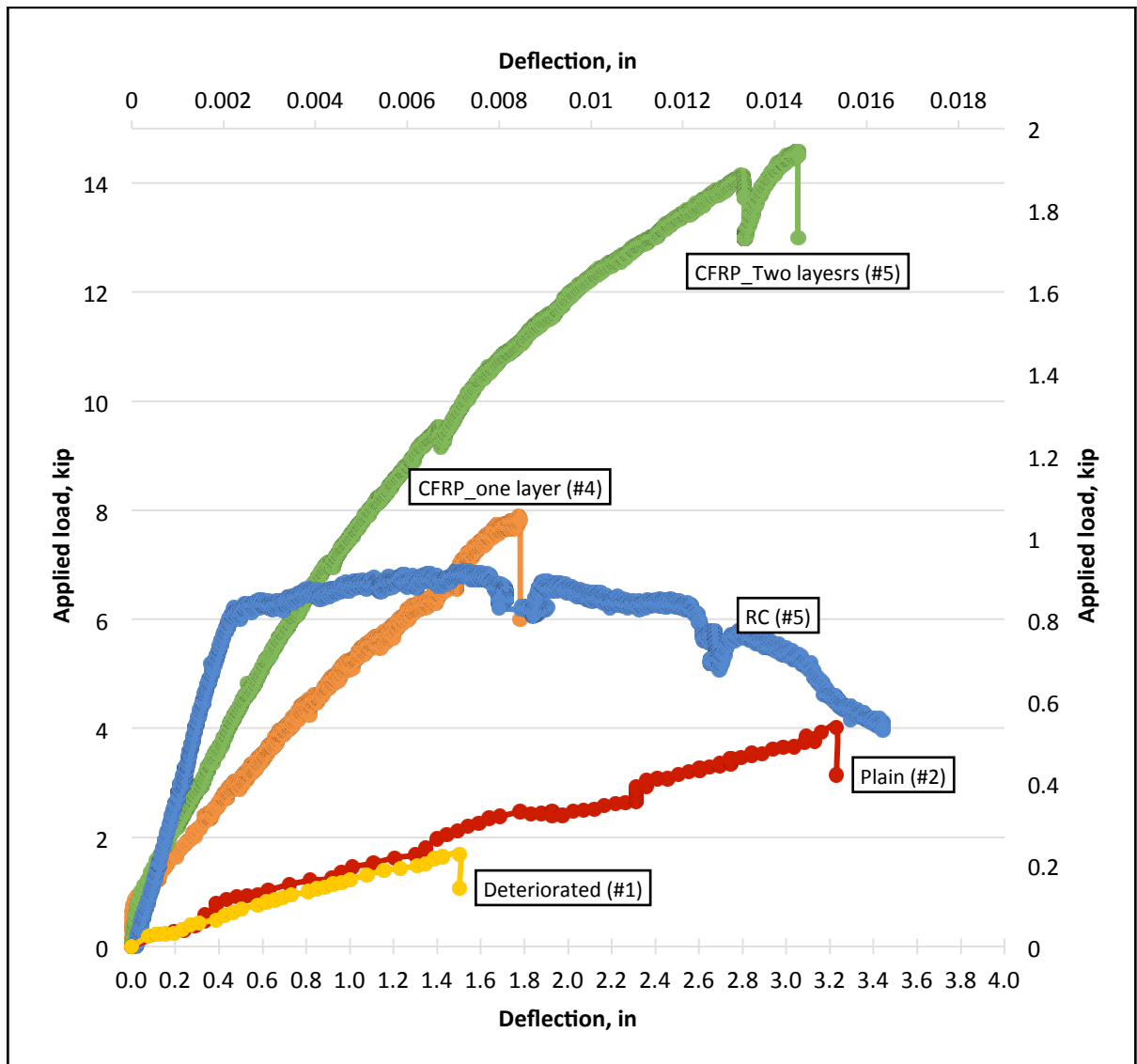
Figure 40: Load-deflection relation for all five beams on separate axis

5.2. Deflection

The deflection of each beam at the midpoint of beam span was recorded. Mid-span deflections of each strengthened beam are compared to the control beam. Table 12 shows the

mid-span deflections of all the beams at their failure loads. The largest deflection was experienced by Beam #5. This beam had a maximum deflection of 3.05 inches, twice that of the control beam. Conversely, Beams #1 and #2 experienced the lowest deflections at failure load, respectively. All the strengthened beams experienced deflections larger than those of the control beam at their failure loads, which proves that using CFRP increased the load carrying capacity and ductility of the deteriorated beams.

Table 12: Experimental max deflection

Beam No.	Beam	Experimental Max. Deflection (in.)
1	Deteriorated	0.0071
2	Plain	0.0153
3	RC (Control)	1.55
4	CFRP (One layer)	1.78
5	CFRP (Two layers)	3.05

The beam deflections were also compared at the 4.85 kips service load of the control beam (Table 13); CFRP reinforced beams experienced significantly larger deflections. The service load was calculated based on the ultimate load applied on the control beam (Beam #3). This was expected since the presence of the CFRP sheets increases the strength of the deteriorated beams allowing the beams to deflect more. The stiffness of the strengthened beams was higher than that of the control beams. Increasing the numbers of CFRP layers generally reduced the mid span deflection at service load and increased the beam stiffness for the same value of applied load. The CFRP prevents the distribution of cracks, it can keep the original shape of beam and increase the deformability of the concrete and thus, the behavior becomes ductile instead of being fragile.

Table 13: Beam deflection at service load

	Deflection (in)	Beam 1	Beam 2	Beam 3	Beam 4	Beam 5
	0.27	0.0071	0.0068	0.0366	0.0021	0.0036
	0.54		0.0153	0.053	0.0021	0.0146
Service load	4.85			0.355	0.926	0.576
	6.8			1.55	1.5	0.86
	7.89				1.78	1.07
	14.58					3.05
	Failure load (kip)	0.27	0.54	6.8	7.89	14.58

5.3. Absorbed Energy

In calculating the energy absorption of the tested beams, load-displacement curves are used. The area under the curve yields the energy stored in each beam before it fails. Energy absorption rates of all beams were calculated using the computer software Mathcad. The amount of convertible energy is directly proportional with the length of the plastic region. As energy is the ability to do work, the amount of energy consumed has importance. In the load-displacement curve, energy consumption was found at the point where the maximum loading occurred and are shown in Table14.

Table 14: Beam absorbed energy at max load

Beam No.	Beam	Absorbed Energy @ max load
1	Deteriorated	0.000877
2	Plain	0.0043
3	RC (Control)	8.53
4	CFRP (One layer)	8.126
5	CFRP (Two layers)	28.08

The area under the load-deflection curve is used to estimate the energy-absorbing capacity or toughness of the material, Figure 41. Increase of the toughness also means improved performance under loading. From Figure 42, the energy absorption is larger for fiber-reinforced specimens than that for plain concrete specimens. This implies that the fiber-reinforced specimens require more energy to fracture than the plain concrete.
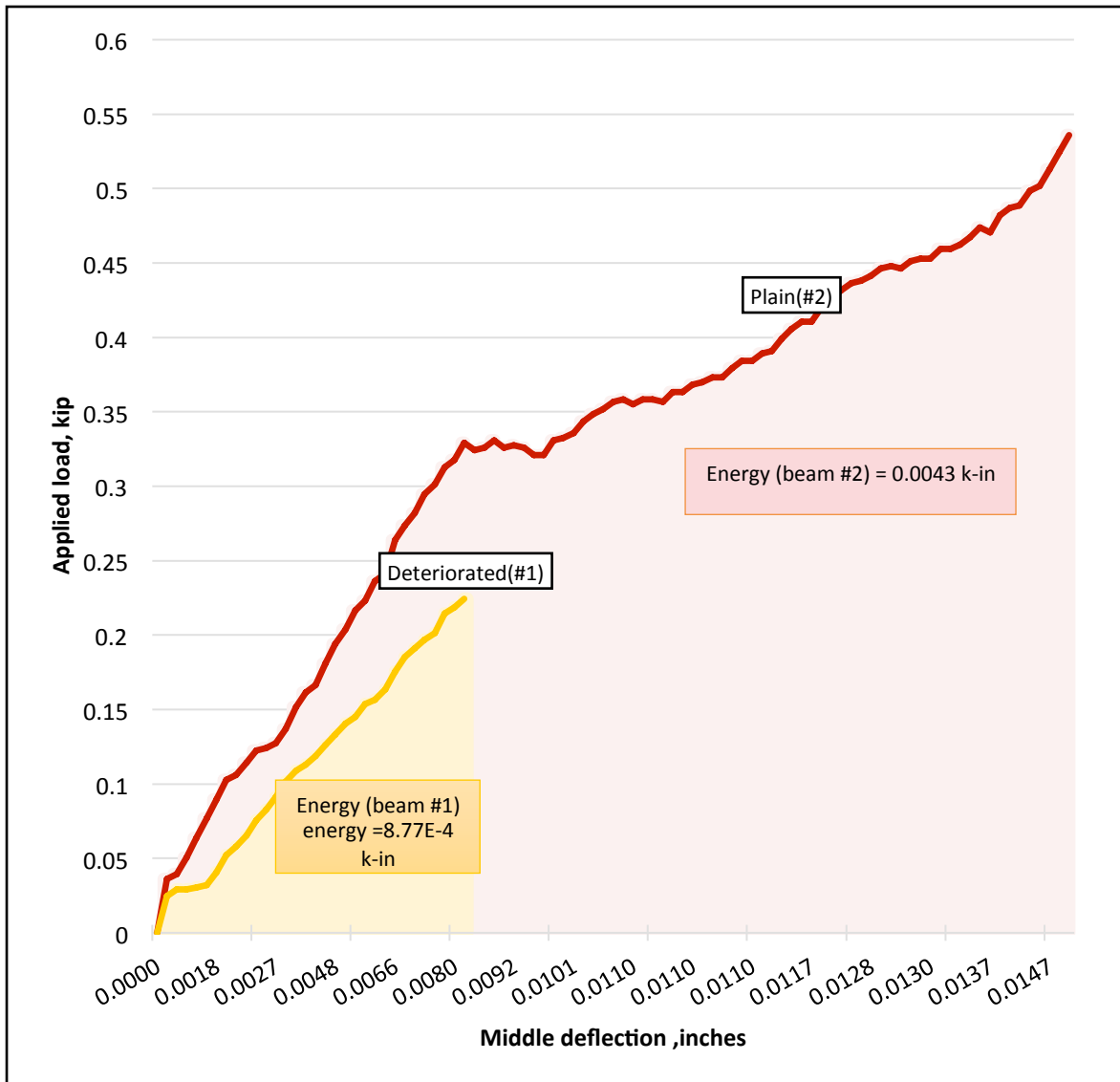
Figure 41: Energy absorption for beam #1 & #2
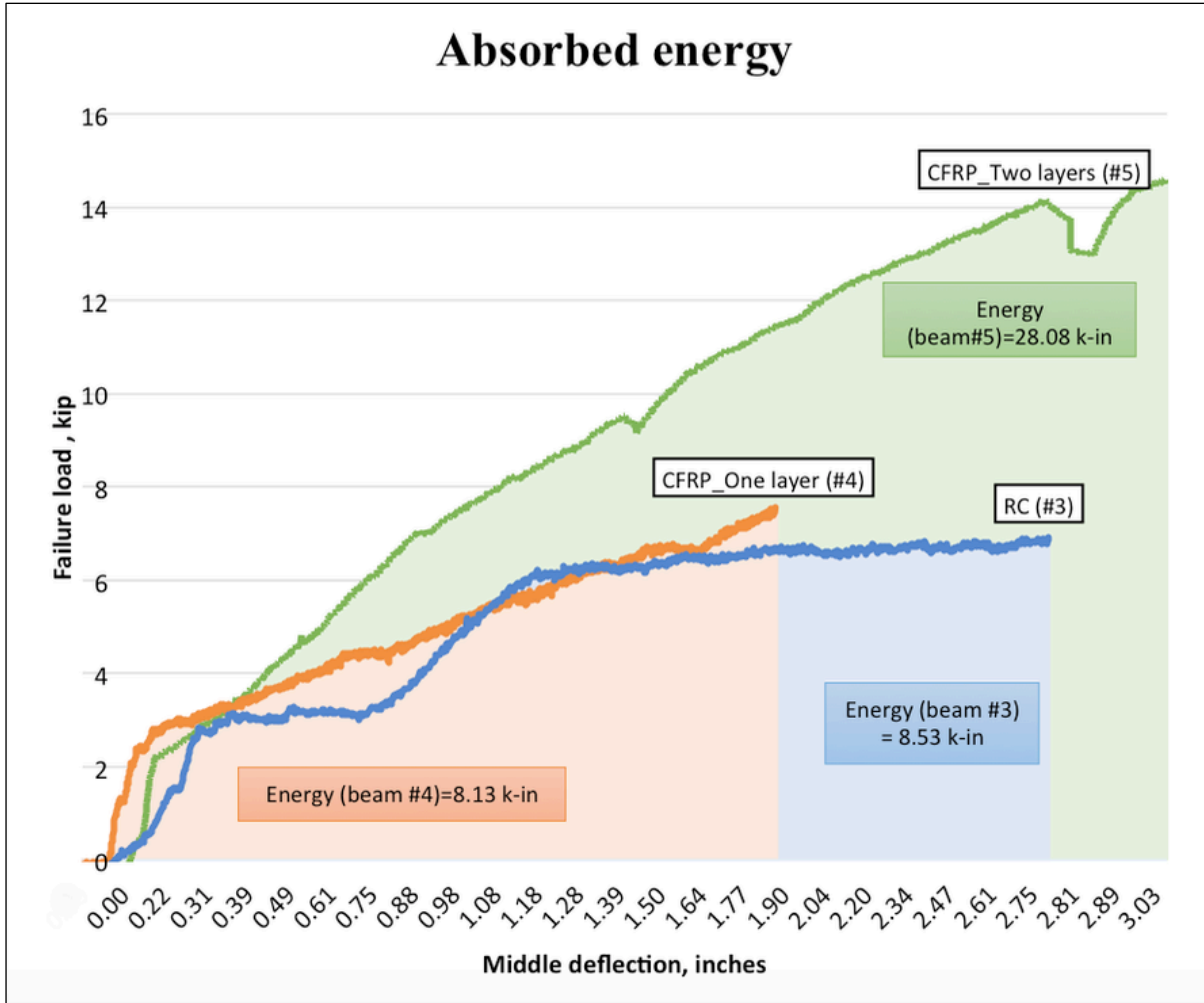
Figure 42: Energy absorption for beams #3, #4 &#5

5.4. Theoretical correlation

5.4.1. American Concrete Institute

The guidelines suggested by ACI Committee 440.2R on calculations for shear strengthening effect using FRP to a reinforced concrete beam were used to predict the contribution of CFRP. The guidelines also present guidance on calculations on flexural

strengthening effect of adding longitudinal FRP reinforcement to the tension face of a reinforced concrete member [32].
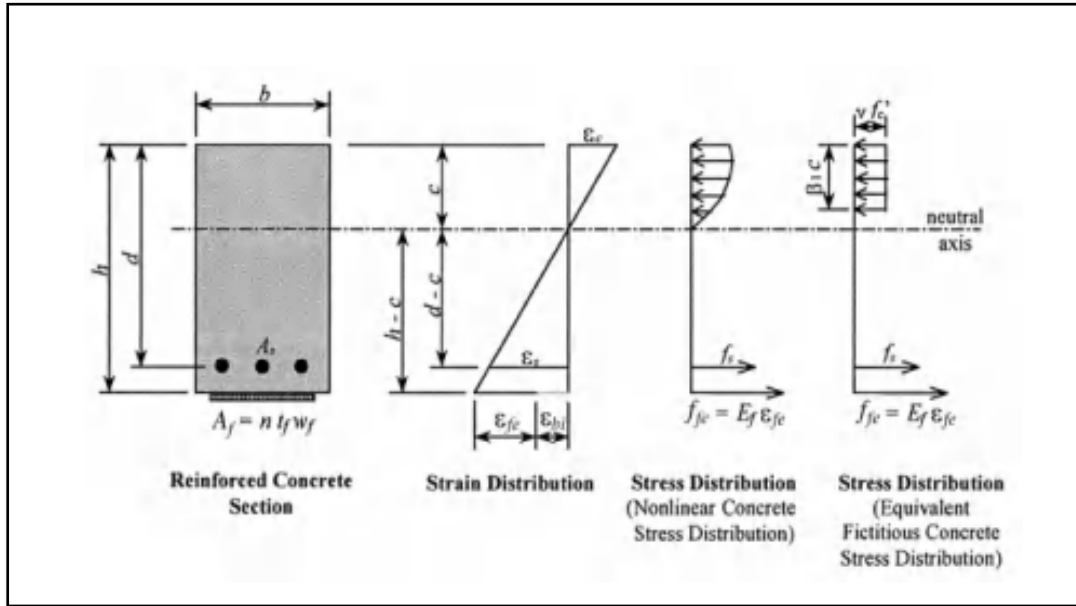
Figure 43 :Internal stress-strain relationship for tensile RC [32]

The beam theoretical load capacity, P_n, was obtained from Eq. 3

$$Pn = \frac{Mn \times 3}{L}$$
Equation 3

where M_n = theoretical moment capacity, and L = span length of the beam specimen.

The nominal flexural strength of a section with CFRP external reinforcement is computed from Eq.4

$$M_n = A_s f_s \left(d \frac{\beta_1 c}{2}\right) + \psi_f A_f f_{fe} (h \frac{\beta_1 c}{2})$$
Equation 4

The nominal shear strength of a CFRP-strengthened concrete beam can be determined by adding the shear resistance contribution of the FRP (V_f) to the steel stirrups contribution (V_s) and concrete shear resistance (V_c) according to Eq.5

$$V = V_c + V_s + V_f \qquad\qquad \textit{Equation 5}$$

Where V_c and V_s can be determined from design standard, such as ACI 318-08. The shear contribution of the FRP shear reinforcement can be determined by calculating the force resulting from the tensile stress in the FRP across the assumed crack. Therefore, FRP contribution to shear strength is based on the fiber orientation and the assumed crack pattern. The shear contribution of the FRP shear reinforcement can be determined by:

$$V_f = \frac{A_{fv} f_{fe} (\sin \alpha + \cos \alpha) d_{fv}}{s_f} \qquad\qquad \textit{Equation 6}$$

where the α is the inclination angle of the CFRP, s_f is the width of the CFRP and , A_f is the total FRP area.

The deflection at the mid-span of all beam were calculated using the maximum deflection equation

$$\Delta = \left(\frac{Pa}{24EI}\right) \times (3L^2 - 4a^2) \qquad\qquad \textit{Equation 7}$$

5.4.2. Comparison of Analytical Calculations with Experimental Results

Results from the experimental and analytical study are shown in Table 15. The maximum loads of all of the beams are calculated using the analytical procedure presented in the previous section and are compared with the experimental results. Compared to the experimental values for all systems, the design method provides reasonable accuracy. The ratio of P_{exp} / $P_{theo.}$ (Experimental study / Analytical study) are about 1.00 for all beams. Also, Figure 44 compared theoretical and experimental max load capacity for all beams. It is clear that the calculated values are very close to the experimental results.

Table 15: Theoretical max load

Beam	Beam No.	Experimental max load (kip)	Theoretical max load (kip)	Ratio P_{exp}/P_{theo}
Deteriorated	1	0.274	0.266	1.03
Plain	2	0.538	0.531	1.01
RC	3	6.89	6.64	1.04
CFRP (One layer)	4	7.89	7.60	1.04
CFRP (Two layers)	5	14.58	14.40	1.01
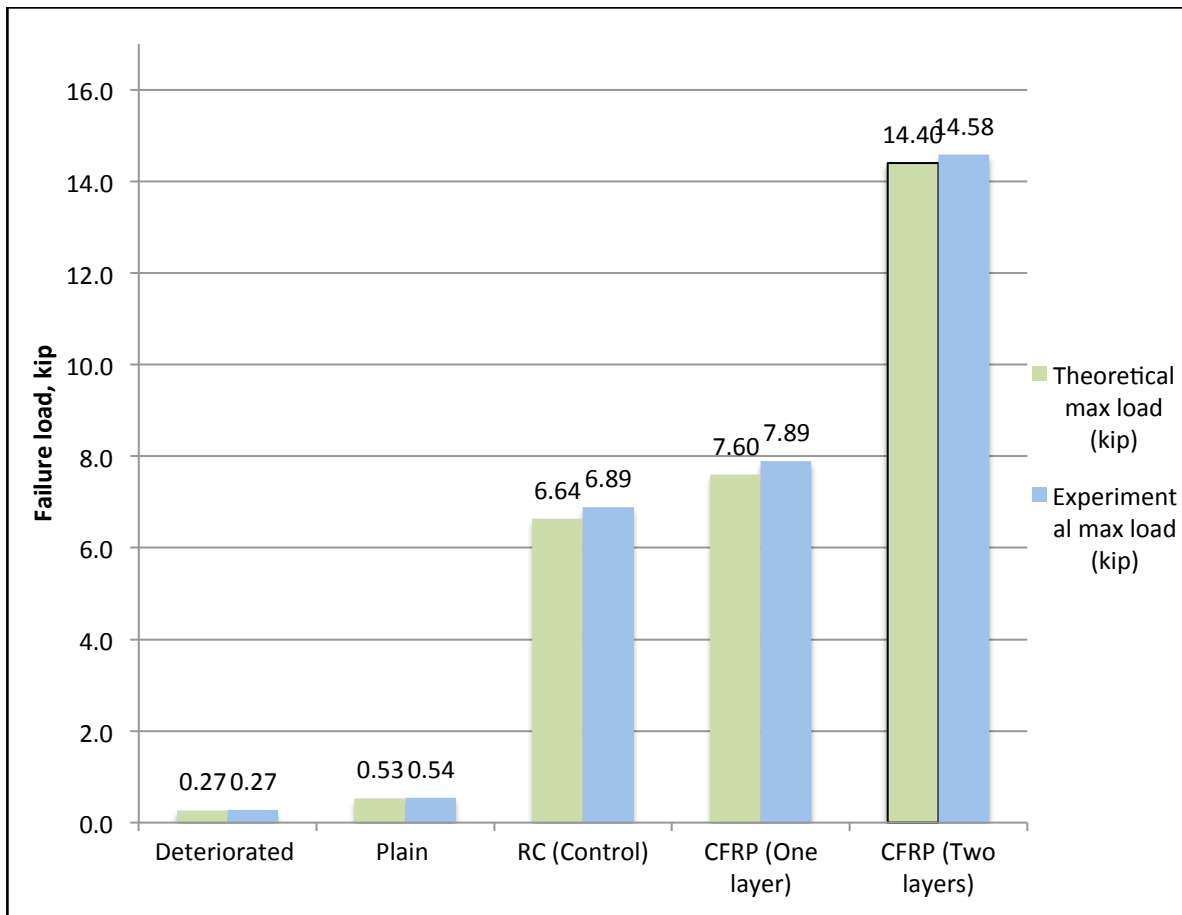
Figure 44: Theoretical vs. experimental load values

Also, Table16 presents the comparison between the experimental and theoretical deflection. The ratio between the experimental and theoretical values varies from 0.5 to 1.6.

Table 16: Beam experimental and calculated deflection values

Beam	Beam No.	Experimental Max Deflection (in.)	Theoretical Max deflection (in.)	Ratio $\Delta_{exp}/\Delta_{theo}$
Deteriorated	B1	0.0071	0.0152	0.5
Plain	B2	0.0153	0.0304	0.5
RC (Control)	B3	1.55	0.975	1.6
CFRP (One layer)	B4	1.78	1.11	1.6
CFRP (Two layers)	B5	3.05	2.11	1.4

CHAPTER 6: **Conclusions**

From the experimental and analytical study conducted in this research project, on beams strengthened in shear and flexure with externally bonded CFRP reinforcement the following can concluded:

- Carbon fiber reinforced polymer significantly improved the behavior of fully corroded reinforced concrete beams.

- The results show that CFRP laminates provides additional load carrying capacity.

- The capacity of the deteriorated beam with one layer was restored compared to the original beam.

- Using a proper combination of circumferentially and longitudinal fibers coupled with the proper epoxy can double the ultimate load carrying capacity of "original" beams without corroded steel.

- All the CFRP strengthened beams exhibited brittle behavior requiring a higher factor of safety in design.

- The number of the fiber layers was found to have an important effect, especially where two layers were applied. There was a greater strengthening effect and better control of the shear crack propagations.

- There was a consistency for the strengthened beams in failure mechanism, in terms of concrete crushing and fiber ruptures in the tension face of the beams.

- The energy absorption increased after bonding CFRP, which means that the beams have become stiffer and a big load is required to break the beams.

- The proposed ACI 440.2R design guidelines to estimate the flexural and shear capacity for beams strengthened gave promising results.

- These results also indicate that the application of CFRP laminates whenever needed, taking into consideration anchoring, rigidity, and stiffness, does actually results in an increase of strength of beams and provides additional load carrying capacity.

CHAPTER 7: **Limitation and Future Work**

This research is limited to investigate the application of CFRP material as external reinforcement. Based on the experimental results, the following recommendation are made

- There are some limitations in the corrosion consideration in the experiment. Future work may include investigating accelerated steel corrosion while increasing the corrosion rate to denote an extreme case of corrosion.

- Only load and deflection were collected and examined. Future work may investigate the stress and strain distribution in the strengthen beams.

- A finite element model may also be used to predict and verify the experimental results of beams retrofitted with CFRP.

- Only simply supported reinforced concrete beams strengthened with unidirectional was studied. Continues beams may be investigated.

- Most of the current experimental available work is for the case of CFRP wrapped entirely around the beam. Experimental studies are needed for case of the more practical U-jacket configuration.

- Investigate using high strength concrete and CFRP.

- Investigate using different orientation of CFRP sheets.

CHAPTER 8: **References**

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Beam	Notation	Size	Unit
Width	b_w	4	in
Height	h	6	in
Length	L	72	in
Loaded length	L	68	in
Concrete cover	c	0.5	in
Effective depth	d	5.75	in
Area of concrete	A_g	24	in^2
Concrete			
Compressive strength	f'_c	3220	psi
Modulus of elasticity	E_c	3456	ksi
Steel reinforcement			
Steel compressive bars diameter	$\varphi_{\#9}$	0.147	
Area of compressive reinforcement	$A_{\#9}$	0.0172	in^2
Yield strength in shear reinforcement	f_{ys}	30000	psi
Steel tensile bars	$\varphi_{\#3}$	0.375	in
Number of tensile bars		2	
Steel tensile reinforcement ratio	ρ	0.01	1%
Area of tensile reinforcement	$A_{\#3}$	0.11	in^2
Yield strength	f_y	72	ksi
Modulus of elasticity	E_s	29000	ksi
BFRP external reinforcement			
Thickness	t_f	0.0065	in
Width	d_f	20	in
Fiber alignment	α	0	
Modulus of elasticity	E_f	33000	ksi
Ultimate tensile strength	f_{fu}	522.5	ksi
Ultimate tensile strain	ε_{fu}	0.0159	in/in
Flexure strengthening calculations			
Flexure load capacity	M_n	79.6	kip-in
Shear strengthening calculations			
Effective length of FRP	L_e	2.02	in
Modification factor	k1	0.87	
Modification factor (U-wrapped)	k2	0.65	
Shear contribution of FRP	V_f	9.98	kip