STRENGTHENING OF SHEAR DEFICIENT BEAMS WITH CFRP LAMINATES
WITH DIFFERENT TYPES OF ANCHORAGE SYSTEMS

by
Khalid Mustafa Elradi Mohamed

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Approval Signatures

We, the undersigned, approve the Master’s Thesis of Khalid Mustafa Elradi Mohamed. Thesis Title: Strengthening of Shear Deficient Beams with CFRP Laminates with Different Types of Anchorage Systems.

<table>
<thead>
<tr>
<th>Signature</th>
<th>Date of Signature (dd/mm/yyyy)</th>
</tr>
</thead>
</table>
| Dr. Jamal Abdalla  
Professor, Department of Civil Engineering  
Thesis Advisor | |
| Dr. Rami Hawileh  
Professor, Department of Civil Engineering  
Thesis Co-Advisor | |
| Dr. Adil Al-Tamimi  
Professor, Department of Civil Engineering  
Thesis Committee Member | |
| Dr. Wael Abuzaid  
Assistant Professor, Department of Mechanical Engineering  
Thesis Committee Member | |
| Dr. Robert Houghtalen  
Head, Department of Civil Engineering | |
| Dr. Ghaleb Husseini  
Associate Dean for Graduate Affairs and Research  
College of Engineering | |
| Dr. Richard Schoephoerster  
Dean, College of Engineering | |
| Dr. Mohamed El-Tarhuni  
Vice Provost for Graduate Studies | |
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Dedication

To my beloved parents,
To my Siblings,
To my fiancé.
Abstract

Retrofitting and repairing deteriorating structures have been achieved using several techniques. Strengthening of Reinforced Concrete (RC) members in shear with externally bonded fiber reinforced polymer (FRP) plates and sheets has been commonly accepted. FRP de-bonding from the concrete substrate is one of the most common types of failure in shear strengthening of RC beams. Many shear strengthening methods have used different anchorage systems to solve the problem of the de-bonding of FRP laminates. The most common types of anchorage in use include full wrapping, U-wrapping, FRP-spikes, in addition to other types of mechanical anchorages. This study explores the use of groove-epoxy and bore-epoxy anchorages. In this investigation, 15 shear deficient rectangular RC beams were strengthened with carbon (CFRP) sheets and plates bonded by groove-epoxy anchorages of different widths and bore-epoxy anchorages of different depths and spacing. The beams were tested under four-point bending. The aim of this study is to investigate the feasibility of using epoxy-anchorages, specifically groove-epoxy and bore-epoxy to reduce or eliminate FRP de-bonding failure and increase the FRP strength that will lead to an increase in shear strength of aging beams. Both methods have shown an increase in the shear capacity when compared with the control beams and with the externally bonded reinforcement (EBR) strengthening method without anchorage. In the groove-epoxy anchorage method, the two medium grooves of 10 mm width showed the best performance among the groove widths while in bore-epoxy anchorage method, the large bores of 30 mm diameter showed the best performance among the bore diameters. Groove-epoxy anchors have increased the shear capacity by 112 % over the control beam and 52 % over the EBR strengthened beam. Bore-epoxy anchors have increased the shear capacity up to 68 % over the control beam and 20 % over the EBR strengthened beam. The shear strength of three specimens were predicted using the relevant codes of practice (ACI-440.2R-08, CAN/CSA-S806-02, FIB 14 and TR55). The prediction showed that CAN/CSA-S806-02 is the most accurate when compared with the other codes.

Keywords: Shear strengthening, shear deficient RC beam, anchorage systems
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<th>Abbreviation</th>
<th>Description</th>
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<tr>
<td>CFRP</td>
<td>Carbon Fiber Reinforced Polymer</td>
</tr>
<tr>
<td>EBR</td>
<td>Externally bonded reinforcement</td>
</tr>
<tr>
<td>Ef</td>
<td>Tensile modulus of elasticity of FRP (MPa)</td>
</tr>
<tr>
<td>ffe</td>
<td>Effective stress level in FRP (MPa)</td>
</tr>
<tr>
<td>εfu</td>
<td>Effective strain level in FRP (mm/mm)</td>
</tr>
<tr>
<td>φ</td>
<td>Nominal strength reduction factor</td>
</tr>
<tr>
<td>Vn</td>
<td>Nominal shear strength (N)</td>
</tr>
<tr>
<td>Vu</td>
<td>Ultimate shear strength (N)</td>
</tr>
<tr>
<td>Vc</td>
<td>Nominal shear strength provided by concrete (N)</td>
</tr>
<tr>
<td>f′</td>
<td>Concrete compressive strength (MPa)</td>
</tr>
<tr>
<td>bw</td>
<td>Web width (mm)</td>
</tr>
<tr>
<td>d</td>
<td>Effective depth for tension reinforcement (mm)</td>
</tr>
<tr>
<td>Vs</td>
<td>Nominal shear strength provided by steel stirrups (N)</td>
</tr>
<tr>
<td>Av</td>
<td>Area of shear reinforcement (mm2)</td>
</tr>
<tr>
<td>Fy</td>
<td>Yield strength of reinforcement (MPa)</td>
</tr>
<tr>
<td>S</td>
<td>Spacing of transverse reinforcement, in. [mm]</td>
</tr>
<tr>
<td>Vf</td>
<td>Nominal shear strength provided by FRP stirrups (N)</td>
</tr>
<tr>
<td>Afv</td>
<td>Area of FRP shear reinforcement (mm2)</td>
</tr>
<tr>
<td>Ffe</td>
<td>Effective stress in FRP; stress level attained at section failure (MPa)</td>
</tr>
<tr>
<td>α</td>
<td>Angle of inclination of FRP wraps (degrees)</td>
</tr>
<tr>
<td>df</td>
<td>Effective depth of the FRP strengthening, measured from the top of the FRP to the tension reinforcement (mm)</td>
</tr>
<tr>
<td>Sf</td>
<td>Spacing of FRP shear reinforcement (mm)</td>
</tr>
<tr>
<td>N</td>
<td>Number of plies of FRP reinforcement</td>
</tr>
<tr>
<td>tf</td>
<td>Nominal thickness of one ply of FRP reinforcement (mm)</td>
</tr>
<tr>
<td>w f</td>
<td>Width of the FRP reinforcing plies (mm)</td>
</tr>
<tr>
<td>εfe</td>
<td>Effective strain level in FRP reinforcement (mm)</td>
</tr>
<tr>
<td>k1</td>
<td>Modification factor applied to κ v to account for the concrete strength</td>
</tr>
<tr>
<td>k2</td>
<td>Modification factor applied to κ v to account for the wrapping scheme</td>
</tr>
<tr>
<td>Le</td>
<td>Active bond length of FRP laminate (mm)</td>
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Chapter 1. Introduction

This chapter provides a short introduction about the use of CFRP in strengthening of shear deficient beams. It presents the significance of the research, objectives and contributions. Finally, general organization of the thesis is presented.

1.1. Overview

Over the years buildings age and consequently show pressing need for additional strengthening and retrofitting in order to increase their strength, durability and lifetime. A significant number of existing concrete buildings have inadequate shear capacity, this due to many reasons such as deterioration in material, changes in design codes, increase of the pay loads, changes in use and function, poor initial design, corrosion problems in reinforcing steel, seismic deflections, and cracks. As a result, researchers started to seek efficient, effective and economical methods for retrofitting and repairing of these structures rather than demolishing and reconstructing them. One of these solutions is retrofitting by externally bonding strengthening materials to structural members of the structure [1]. They introduced many materials that can be used as strengthening material such as steel, aluminum and fiber reinforced polymers (FRPs) [2, 3]. FRPs are composite materials, which are made of fibers in a polymeric resin and they are the most widely used strengthening material. This is due to its high desirable features. Such features include its high tensile strength, lightweight, high fatigue resistance, and ease of installation and corrosion resistance. FRPs outperformed the use of steel and aluminum as strengthening materials. Nevertheless, their major disadvantage is the lack of ductility compared to steel and aluminum. The use of FRPs as externally bonded strengthening materials started in 1980s in both Europe and Japan, this use has showed dramatic increase during the past 20 years from a few to thousands of applications and has become very popular nowadays. There are many types of FRPs that are currently in use including carbon (CFRP), glass (GFRP), and aramid (AFRP) [4]. The resin is usually used and it surrounds and encapsulates the fibers to work as a bonding material, also to protect them from damage, to maintain their alignment and to allow distribution of load among them. The most common types of polymeric resin are epoxy, polyester and vinyl ester. Carbon fiber reinforced polymers are used either to increase the shear strength when building a new structure or for strengthening and
rehabilitation of existing structures when it shows deficiencies in strength due to the above reasons [5-9].

Since early 1990s, researchers started doing tests on the shear strength of concrete beams and how to increase their shear capacity using externally bonded FRP materials. Many researchers have published analytical models and shear design equations in order to predict the increase in shear capacity due to externally bonded FRP materials [10]. American Concrete Institute (ACI 440) has published reports showing guidance for the selection, design, and installation of FRP materials for externally strengthening of concrete elements. Data with respect to material properties, design, installation, quality control, and maintenance have been introduced in ACI440. Engineers can use this data select and design FRP system in order to increase the strength and stiffness of reinforced concrete elements [11, 12].

ACI committee report has mentioned many areas that need to be studied such as effective strain of FRP systems that do not completely wrap around the section, and anchoring of FRP systems. This research will investigate strengthening of reinforced concrete (RC) beams deficient in shear with externally bonded CFRPs using different anchorage systems.

1.2. Research Problem and Significance

The rehabilitation of old structures which are deteriorated such as buildings, bridges, parking structures, marine structures has become a major issue during the last four decades. This issue has been addressed by many researchers to figure a way to repair these structures. Strengthening of these structures using CFRP is one of the most widely used solution method. Beams are one of major elements in building and bridges and other structures and they can fail in flexure or shear. This study will focus on shear strengthening of shear deficient beams using CFRPs plates and sheets. When strengthening RC beams against shear with FRP, the fibers do not reach their ultimate strength and mostly a de-bonding of the FRP happens. To increase the utilization of the FRP strength, end anchorage become necessary especially when the length of the FRP is limited and the bonded length after a critical section is not enough to reach the ultimate strength of the FRP. In addition, problem of premature peeling is also a concern especially when strengthening a T-section beam because the shear strengthening is only located on the web of the member and the FRP sheets may end below the position of the neutral axis. Many systems have been used in order to solve
the problem of de-bonding of CFRP sheets or plates by anchoring the sheet or plate to the element and they are called anchorage systems. When using CFRPs without an anchorage it’s called externally bonded reinforcement (EBR), so this study will particularly investigate experimentally two anchorage methods with different variables that have not been studied, these methods are externally bonded reinforcement on Grooves (EBROG) and Boring method anchors and compare them with the EBR method. The significance of this research is to explore the effect of these types of anchorages in increasing the shear capacity of shear deficient reinforced concrete beams.

1.3. Research Objective and Contribution

The objectives of this research are to:

1) Investigate experimentally the effect of using two sided CFRPs sheets on shear strength of shear deficient RC beams.

2) Study the effect of using two-sides wrapped CFRPs plates on shear strength of shear deficient RC beams.

3) Examine the effect of using Groove-epoxy and Bore-epoxy as anchorage mechanisms for attachment of the CFRP sheets and plates to the RC beams and study their effects on de-bonding and whether they increase the strain on CFRP and the beam shear capacity.

4) Study the mode of failure of the strengthened beams when subjected to four points load test.

5) Use the relevant codes of practice (ACI440, CSA, FIB 14 and TR55) to predict the shear strength capacity of the strengthened RC beams and compare their performance.

1.4. Research Methodology

The following steps were followed to fulfill the objectives of this research:

Step 1: The literature related to shear strengthening of RC beams using CFRP is reviewed.

Step 2: The problem in anchorage of CFRP sheets and plates is identified and test matrix is done.

Step 3: Experimental program for the test matrix has been conducted.

Step 4: Theoretical prediction is done using relevant codes of practice.
Step 5: The experimental results are evaluated, discussed and compared with the theoretical prediction.

Step 6: The conclusion and the major findings are stated, and possible future research is proposed.

1.5. **Thesis Organization**

In this chapter, an introduction about the shear strengthening of RC beams using CFRP is presented. The rest of the thesis is organized as follows: Chapter 2 Presents a comprehensive literature review on the anchorage systems and techniques that have been used is externally bonded CFRP laminates. The experimental program including preparation of specimens, instrumentation and testing are discussed in Chapter 3. Chapter 4 presents the experimental results. Chapter 5 presents comparison between the predicted results done by the codes and the experimental. Chapter 6 summarizes the major findings and provides suggestions for future work.
Chapter 2. Literature Review

In this chapter, we discuss the design philosophy of FRP, the related work that has been done in shear strengthening with FRP and the different anchorage systems that is used in FRP applications.

2.1. FRP Shear Strength and Design Philosophy

In the last few decades, researchers performed an extensive research on how to increase the shear strength of RC beams using the FRP either plates or sheets and the methods used to strengthen these beams. The beams which are weak in shear can be strengthened using FRP through common techniques: two-sided, U-wrapping or full wrapping as in Figure 2-1. Full wrapping is used mostly in columns in which all sides are reachable, and this does not apply in beams where the slab makes is impossible then the three sides wrapping is the solution which called U-wrap, or bonding in two sides of the beam can be done. A full wrap is the most powerful technique followed by U-wrap and two sides wrap. In all these techniques, the fiber can be installed either continuously along the span of the beam or as discrete strips. An epoxy resin is used for applying The FRP materials to the surface of the concrete member after enough surface preparation of the concrete, this surface preparation is done by sandblasting, water jetting, and the application of a suitable primer. After applying the FRP materials it must be cured for seven days in order to achieve the full bond strength of the system [13, 14].

Figure 2-1: Shear Strengthening Wrapping Methods [3]
2.2. **Shear strengthening of RC beams with CFRP**

This section summarizes some work associated with the external Shear strengthening of RC beams with CFRP.

Challal et al. [15] have experimented how reinforced concrete beams behave when strengthened against shear by using unidirectional carbon fiber polymers side strips. Eight beams were tested which exposed to four loading patterns and divided into three types: full strength in shear, under reinforced in shear and strengthened in shear targeting the same strength in the full-strength beams. The unidirectional CFRPs were bonded externally using epoxy in a shape either perpendicular or diagonal to beam's longitudinal axis acting as external shear reinforcement. The outcome was that the CFRPs maintain and increase shear capacity which leads to less shear cracking, diagonal strips outstrip the vertical ones besides proving that strips are better than plates.

Khalifa et al. [16] have carried out an experimental program examining the shear performance of six T-section RC beams which strengthened externally using CFRP. One beam treated as a control beam and five beams with multiple arrangements. They investigated various parameters: wrapping types, CFRP alignment, CFRP amount and CFRP end anchorage. One beam with 90°–0° direction continuous U wrapped fiber, one with 90° two-sided Strips fiber, one with 90° U wrapped strips fiber and one with 90° continuous U wrapped fiber with anchoring in the flanges using U-anchors system. They conclude that 0° ply fiber does not increase shear capacity and using U-anchor system can improve CFRP performance. Generally, the CFRP increased the shear capacity by 35 % up to 145 %, U wrapped outperformed two-sided wrapped fibers. They resulted that there is an optimum quantity of CFRP, despite that CFRP strips and continuous sheets have the same effectiveness but strips are not practical in fields besides that they are not as safer as sheets when damaged. The authors compared their experimental results with calculated one’s using proposed design method by ACI and found it satisfactory.

Bousselham et al. [17] conducted an experimental program studying the performance of 22 RC beams strengthened against shear using externally bonded CFRP, they used continuous sheets of fibers attached to the beams in form of U-wrap. Their objective exactly was to know the effect of the numbers of CFRP, the internal transverse steel amount and the ratio of the shear length to the depth of the beam. In
addition, they investigated the mode of failure and compared between experimental results and predicted results from different design codes. They concluded that as the transverse steel increased the shear strength capacity decrease, but this capacity is higher in deep beams than slender beams, also this capacity is unrelated to the CFRP thickness. They found that strengthening with CFRP didn’t change the Crack pattern as well as crack angle, but the addition of transverse steel in slender beams did change these parameters. One of their findings is that the strain of the transverse steel is higher in beams without CFRP, therefore, the transverse steel has yielded in most of the beams and this was predicted by the design codes. moreover, when comparing their lab results with calculated results, it showed that the value from f ACI 440.2R-02 (ACI Committee 440 2002), CSA S806-02 (Canadian Standards Association 2002), and fib TG9.3 (2001) does not consider fiber thickness, addition of transverse steel and the ratio of the shear length to the depth of the beam.

Mofidi et al. [18] conducted an experimental study on 14 T-section RC beams strengthened against shear using CFRP epoxy bonded. they questioned that which one is more effective CFRP strips or continuous sheets, what is the ideal rigidity of the CFRP strips which means the effective width to strip spacing ratio, where to put CFRP strips considering the transverse internal reinforcement, what is the effect of changing strip width and how does the internal reinforcement affect the amount of shear carried by CFRP. They found that the CFRP strips showed more strain and beams strengthened with CFRP strips showed more deflection which means CFRP strips outstripped continuous sheets. Also, they resulted in that the CFRP can exhibit more shear strength capacity when there is no internal transverse steel than when there is internal transverse steel, the wider CFRP strips displayed more shear strength capacity than smaller strips. As a result of locating CFRP strips in the middle between stirrups rather than installing it parallel to stirrups, the maximum failure load, and stiffness has increased significantly, but showed a less flexural behavior.

Baggio et al. [19] carried out a comparison investigation exploring the feasibility of carbon FRP, Glass FRP, and fiber reinforced cementations matrix (FRCM) when used in strengthening RC rectangular beams against shear, in addition, they explored the effect of FRP anchors. Their major finding was that the CFRP is advantageous over other FRP types. They found that all the tested beams showed the same flexural stiffness without considering the FRP properties. Without using any
anchors, the authors found that the increase in shear capacity was 34 %, 36 %, 50% and 75 % over the reference beam when strengthening with FRCM, full depth GFRP, partial depth GFRP and CFRP respectively. They evidenced that by using GFRP anchors there is 13% increase in shear strength capacity over the reference beam. The FRP anchors stopped the FRP debonding which avoid the beam from brittle shear failure mode despite that they did not reach their maximum strain of rupture. Moreover, using CFRP has changed the failure mode from brittle shear to flexure.

Hawileh et al. [20] performed an experimental work investigating how the externally bonded CFRP continuous sheets installed in the soffits of RC beams affect the shear strength capacity of the beam. Without using any stirrups, with various longitudinal steel reinforcement ratios and with different layers of CFRP sheets. The authors tested 13 beams divided into three groups with reference beam in each group and they found that due to CFRP strengthening there is an increase in concrete shear capacity over the reference beams ranges from 10 % up to 70 % depending on longitudinal steel ratio, they noticed that as the longitudinal steel ratio increases the shear capacity increases too. Moreover, all the tested beams failed with diagonal tension crack mode. In addition, they evidenced that the beams that had more CFRP layers experimented more shear strength, therefore, as the CFRP layers numbers increase, the percentage increase in shear strength decreases. When they compared their experimental results with predicted theoretical results using different codes and method they found that CSA-2004 shear design provisions are advantageous over ACI318-11 simplified and detailed shear strength provisions and another shear strength model that are based on the cracked neutral axis depth. The authors conducted many studies investigating the nonlinear behavior of shear deficient RC beams strengthened with NSM glass fiber reinforcement under cyclic loading, the effect of flexural CFRP sheets on shear strength of reinforced concrete beams, Besides CFRP laminates other materials have been used for shear strengthening of RC beams [20-26].

Hongming et al. [27] examined the shear behavior of RC beams with corroded stirrups when retrofitted with CFRP. They targeted the investigation of shear capacity, failure mode, and deflection at midspan. A total of 16 beams divided into two groups were tested, one group is beams with corroded stirrups and not strengthened and the other one is beams with corroded stirrups and strengthened with CFRP. For the included stirrups, they subjected it into an accelerated corrosion. They observed that all the
beams experienced shear compression failure besides that the corrosion amount has a small effect on failure mode under specific span to effective depth ratio, moreover, they noticed that for all beams there is a reduction in the beam stiffness and the CFRP retrofitting was useless for stiffness. Regarding the corroded unstrengthened beams, their shear capacity showed a little increase when the corrosion degree is less than 5 % then the shear capacity decreased for corrosion more than 5 %. While for the corroded beams strengthened with CFRP the shear capacity has increased up to 15 % over the corroded unstrengthened beams. The authors proposed a model that can predict the shear capacity of both strengthened and unstrengthened beams and they found the results acceptable when compared with the experimental results.

Osman et al. [28] carried out an experimental program by testing a seven RC rectangular section beams strengthened with CFRP sheets using different wrapping types, two sides wrap, U-wrap and full wrap, investigating the effect on shear strength capacity of different shear span to the beam depth ratio, longitudinal steel ratio and stirrups ratio. A comparison has been made between experimental results; ACI code predicted results and finite elements model results. They found that the increase in longitudinal steel has a little effect on shear strength capacity, in addition, as the stirrups amount increase the shear strength capacity increases and crack depth decreases. For all beams, the results of the effect of reinforcement ratio on shear strength and deflection obtained by ACI code were less than those obtained from the experimental program and finite elements model, besides that the last two showed a variation of only 5 %. When comparing the ACI results with finite elements model results in catching the contribution of CFRP with different wrapping systems, it showed a variation of 10 to 19 %, moreover, the model predicted the same load capacities for beams which mean that the model is valid.

Finite elements models have been developed to simulate the behavior of the CFRP laminates and aluminum alloys on the shear and flexural strength of RC beams, these models are able to predict the increased capacity of the beams when strengthened with these materials [29-36]

2.3. Failure modes in shear strengthened RC beams

The most common failure mode noticed in shear strengthening occur from intermediate crack-induced interfacial FRP debonding and FRP tensile rupture [37]. The intermediate crack-induced interfacial FRP debonding starts at the end of a
maximum diagonal tension shear crack. As the diagonal tension shear crack becomes wider, high stress concentrations cause debonding of the FRP sheet along with a thin layer of concrete. This mode of failure is common when diagonal tensile shear cracks occur at locations near the top of the beam because it is very difficult to develop the strength of the FRP laminate at this location.

Failure by FRP tensile rupture may also occur at the edge of the maximum diagonal tension shear crack. As the diagonal tension shear crack becomes wider, the strain in the FRP increases until the FRP reaches its ultimate strain at which time rupture occurs. This brittle mode of failure occurs most often at the lower end of the maximum diagonal tension shear crack the location where the crack width and tensile force are the greatest (Chen and Teng [37])

FRP tensile rupture is the preferred mode of failure as the load-bearing capacity of the FRP sheet has been utilized. Debonding of the FRP sheet is detrimental because it does not allow the full development of the shear capacity of the FRP shear strengthened member. This mode of failure is often brittle which reveals few warning signs. ACI accounts for the debonding failure mode by imposing a maximum strain of 0.4% on the FRP for the design of strengthening applications. The maximum strain threshold of 0.4% also prevents FRP rupture from occurring.

2.4. Anchorage systems

When strengthening RC beams against shear with FRP the fibers do not reach their ultimate strength and mostly a debonding for the FRP happens, this makes end anchorage is an important issue specially when the length of the FRP is limited and the bonded length after a critical section is not enough to reach the ultimate strength of the FRP. in addition, problem of Premature peeling is also a concern especially when strengthening a T-section beam because the shear strengthening is only located on the web of the member and the FRP sheets may end below the position of the neutral axis [38-40].

Many systems have been used in order to solve the problem of debonding of CFRP sheets or plates when used for shear strengthening by anchoring the sheet or plate to the element and they are called anchorage systems and all of them are reviewed below. These systems have been divided into three groups:

1. Anchorage systems using mechanical elements (Mechanical anchors).
2. Anchorage systems using the FRP materials (FRP anchors).
3. Anchorage systems by surface preparation only (Epoxy anchors)

2.4.1. Mechanical anchors.

2.4.1.1. Steel plates and bolts. Steel plates and bolts were used for end anchoring and it’s proved that they are effective besides increasing the shear capacity of RC beams about 20% over the unanchored specimens, bolted anchorage system is used by H.N. Garden et al. [41]. but both of them have downsides, steel plates found not to be practical in the fields not as well as in the laboratory and when using carbon FRP the corrosion happens due to the contact between the steel and carbon fiber, also the bolts cause discontinuity of the FRP by drilling.

2.4.1.2. Plate anchors. Plate anchors are studied by Jin et al. [42] who introduced a new method by anchoring a precast fiber-reinforced cementitious composites (FRCC) plate on top of the FRP sheets that is used for strengthening the RC member. They used a concrete prism for the test and found that the debonding load capacity and deformation load capacity are increased up to 100% over the control samples. Wu el [43] developed a new hybrid anchorage system that combines adhesive bonding like epoxy and a new type of mechanical fastening which is done by making the same method for epoxy bonded FRP sheets and then another epoxy resin is applied on top of the FRP sheet or strip then the mechanical fasteners (thin steel plate plus two small nails) are installed longitudinally along the FRP sheet at specific spacing, they tested one reference sample that was strengthened with epoxy bonded FRP, and three other samples that were strengthened with hybrid bonding-FRP system. They concluded that the bond strength for Hybrid bonding FRP was increased by 7.5 times that for epoxy bonded FRP.

2.4.1.3. Bolted angles. Angles made of Steel or aluminum has been used as FRP anchorage systems at 90° joints by many authors. It’s made by putting the FRP around the joint then bonding the angle to the FRP in the joint and bolting it to the concrete either through or around the FRP sheet. steel angles have disadvantages like corrosion possibility, the 90 corner in the angle makes stress concentrations in the FRP which can cause premature failure [44].

2.4.2. FRP anchors

2.4.2.1. U anchors (near surface mounted-NSM). All above mentioned information leads Khalifa et al. [45] to introduce a new system of anchorage called U-
anchorage, its concept is embedding a bent port of the end (or near the end) of the FRP into a groove made in the concrete, and filling this groove with a viscous paste such as epoxy including or excluding an FRP bar. This type of anchorage can be used either with sheets or pre-cured laminates, fully bonded to concrete or unbonded, it can be applied away from the corner, after the corner or before the corner directly, in addition, can be used for continuous FRP sheets as well as discontinuous FRP sheet (Strips) with inside or outside lap. The authors performed an experimental program by testing three T-Section RC beams, one as a benchmark, second beam was strengthened with a single ply U-wrapped CFRP sheets perpendicular to the beam longitudinal axis and without end anchorage. The third beam was similar to the second beam but with end anchorage in the two sides of the flange (after corner) in addition to a deformed Glass FRP bar of 10 mm diameter. Regarding the failure mode, the first beam experienced a diagonal shear cracks, the second beam experienced a shear compression failure preceded with FRP debonding and it showed a shear capacity increase of 72% over the first beam while the third beam failed with a flexural failure mode which is preferred, no debonding was noticed, the fibers was ruptured after the beam failure besides showing a shear capacity of 145% and 42 % of the first and second beam respectively. All these advantages prove the feasibility of U-anchor system in the third beam. Eshwar et al. [38] studied the performance of two different anchorage systems which used for strengthening of RC beams with CFRP against shear, the two systems are: near surface mounted (NSM) end anchor which called U-anchor and Spike anchor which also called FRP anchors, FRP dowels, or fiber anchors. Considering the U-anchor, they tested 16 beams investigating the location of the end anchor, its groove size, and the anchor bar diameter. They found this end anchor failed with FRP rupture or anchor pullout, moreover anchors before the corner showed more strength than those after the corner by an increase of 40%, also evidenced that the groove size has an effect on the ultimate shear capacity, it increases the capacity by 5 to 20 %, they suggested a minimum groove size to be from 1.5 to 2.5 times the bar anchor diameter knowing that as the groove size becomes larger it turns to be not practical, in order to have no stress concentration and avoid premature peeling the minimum radius for the corner of the groove should be a minimum of 13 mm and the glass FRP bar should be at least 10 mm.

2.4.2.2. FRP anchors or spike anchors. Another type of anchors called carbon fiber anchors, which firstly introduced by the Shimizu Corporation in Japan is studied
by L. Orton et al. [46]. A carbon fiber anchor is constructed by cutting a strip of the CFRP material, imbedding it into a predrilled hole, then fanning the ends of the anchor over the CFRP sheet as in Figure 2-2, saturated with a resin (epoxy) and inserted immediately after the sheet is placed. They resulted in improving the tensile strength capacity.

![Figure 2-2: FRP Spike anchors [24]](image)

Ozbakkaloglu et al. [47] developed FRP anchors to avoid or delay the delamination of the fibers, they conducted a comprehensive test on a total of 81 specimens under direct pullout. The specimens are made by rolling fiber strips that were cut from carbon fiber sheets and inserting it into a predrilled hole in a cylindrical concrete sample with an epoxy adhesive. The authors investigated different parameters: FRP anchors length, diameter, the angle of inclination and concrete compressive strength, the majority of the FRP anchors fail with the pullout of the anchors. They evidenced as the length and diameter of FRP anchors increases, the average bond strength decreases, also it decreases with the increase in the anchor's angle of inclination. Meanwhile, the compressive strength of the concrete showed a little effect on the bond strength of the anchors. They suggested that an adequate amount of FRP should be included in the anchors in order to get away from the fibers rupture. Kim et al. [48] conducted a test on RC T-beams strengthened with CFRP and anchored with CFRP anchors against shear, parameters like shear span to depth ratio, CFRP amount, concrete surface preparation, CFRP anchors layout and CFRP strip layout. They found that using CFRP anchors resulted in eliminating or delaying the CFRP debonding and making the Bond between the CFRP and concrete surface, not a critical parameter. In addition, they found that the highest CFRP contribution to shear happened when the shear span-depth ratio is 3.0. The bond between the CFRP laminates and concrete surface was not a critical parameter when CFRP anchors were adequately provided.
The strength was not dependent on the adhesive bond stress between the CRFP and the concrete. However, bond did increase the overall stiffness because it decreased the length of the CFRP sheet over which strains were distributed and tended to reduce the width of cracks.

Another study conducted by Kim et al. [49] where he examined nine deep T-section beams with the depth of 1220 mm strengthened with CFRP strips with CFRP anchors with a hole diameter of 11.1 mm. The author concluded that FRP anchors increased the useable strain in CFRP strips from what could be achieved through the bond between CFRP and concrete alone. CFRP anchors were, therefore, able to maximize the shear strength contribution of CFRP strips. Also, for the same surface area of CFRP material, there is no significant strength benefit derived from inclining CFRP strips from the normal to a member’s axis, and although the area of CFRP anchors was doubled when doubling the CFRP material in strips, the strength of the strips was not developed. A larger-than-proportional increase in CFRP anchor material is needed for a given increase in CFRP strip strength.

Eshwar [38] spike anchors, they are strands of bundled fibers with one end embedded in the composite matrix and the other end embedded in the concrete member and they can be made by hands, they are applied either at 90 degrees where the axis of the embedded part of the anchor is perpendicular to the FRP plane or 180 degrees where the anchor and the FRP are in the same plane, samples of total 19 beams were tested investigating the embedment of the spikes and its location, Spikes were installed before the web flange intersection. The spikes failed with a sudden bonding and showed an increase in the ultimate shear strength of 25%. The authors tried two embedment lengths 50 mm and 75 mm, the second one experienced an increase of 10% in the ultimate capacity that proves that the embedment depth has little effect. Moreover, the FRP with two spikes outstripped those with one spike shown a significance increase about 200% in the ultimate shear strength. Koutas et al. [50] examined an experimental investigation on the effectiveness of various types of spike anchors in combination with U-shaped FRP jackets for shear strengthening of RC beams. They investigated the orientation, the number and spacing of anchors, and the type of fibers. They concluded that Spike anchors increase substantially the effective strains in U-shaped jackets. Anchors placed inside the slab (that is, nearly vertical) are many times more effective than those placed horizontally.
inside the web and Increasing the number of anchors in the shear span results in no proportional increase in shear resistance because those anchors not above shear cracks are not activated. Table 2-1 below shows a summary of these studies.

Table 2-1: Summary of studied Spike anchors

<table>
<thead>
<tr>
<th>FRP Type</th>
<th>No. of Spikes</th>
<th>Spike Diameter (mm)</th>
<th>Spike Spacing (mm)</th>
<th>Spike Embedment (mm)</th>
<th>Spike Length</th>
<th>Spike Angle</th>
<th>Author</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strips</td>
<td>1,2</td>
<td>10</td>
<td>50</td>
<td>50,75</td>
<td>-</td>
<td>90</td>
<td>Eshwar [38]</td>
</tr>
<tr>
<td>Strips</td>
<td>2</td>
<td>10</td>
<td>-</td>
<td>102</td>
<td>152</td>
<td>90</td>
<td>Kim [49]</td>
</tr>
<tr>
<td>Strips</td>
<td>1</td>
<td>12.7</td>
<td>-</td>
<td>102</td>
<td>152</td>
<td>90,20</td>
<td>Kim [48]</td>
</tr>
<tr>
<td>Continuous</td>
<td>-</td>
<td>12</td>
<td>100,167</td>
<td>70</td>
<td>80</td>
<td>25</td>
<td>Koutas [50]</td>
</tr>
</tbody>
</table>

2.4.2.3. Dry U-wrapped CFRP. Galal et al. [51] proposed a new method for strengthening RC T-section beams against shear by replacing epoxy bonded carbon FRP with mechanically anchored unbonded dry carbon fiber CF sheets. The dry CFRP is U-wrapped and rounded to two steel rods which should be made of a corrosion resistant steel, then the steel rods are anchored to the corners of the web-flange intersection of the T-beam with mechanical bolts. By this, we effectively use a higher tensile strength and modulus of elasticity of the dry CFRP than the epoxy bonded CFRP. A total of three beams were tested. first one as a reference beam, the second beam is strengthened with externally U-wrapped epoxy bonded CFRP and the third beam was strengthened with anchored U-wrapped dry CFRP sheet, the third one showed a significant increase in shear strength capacity of 48 % over the first beam and 27 % increase over the second beam, a little increase in the flexural stiffness, besides, it sustained the load until the flexural tensile steel reinforcement almost yielded. In addition, they evidenced that the unbonded dry carbon fiber CF sheets prevent the CFRP from debonding thus its full capacity is used, moreover, this method needs less hard work and time when compared to other methods.

2.4.2.4. CFRP straps. The use of CFRP straps started in 1997 and then in 2002 Lees et al. [52] used Nonlaminated prestressed CFRP straps to strengthen T-section RC beams deficient in shear. The unstrengthened control beam failed in shear while the CFRP strengthened beam failed in a ductile mode after the longitudinal yielding of the main reinforcement with an increase of 40% in the shear capacity. Kesse et al. [53] tested 12 cantilever beams deficient in shear, four as control beams and the
rest strengthened with CFRP straps, they targeted to know the effect of strap spacing, strap stiffness, the initial strap prestressing force, the existence of precracks in addition to the failure mode. They found that the strengthened beams failed with a ductile mode in flexure while the control beams failed with a brittle shear failure with a load of 90% higher than the control beam. Moreover, they found that the straps spacing has no effect on the shear capacity and mode of failure, besides evidencing that there are a minimum and maximum limit for the CFRP straps prestressed force. The presence of cracks affected the stiffness of the beam but did not have an effect on the ultimate shear capacity.

2.4.2.5. CFRP U-straps. Yalim et al. [54] performed a test on 26 RC T beams strengthened with wet layup or precured FRP sheets and anchored with different anchorage levels in addition to different concrete surface roughness. They found that anchorage level affects the failure mode, the beams without FRP straps failed with FRP debonding, the beams with 4 and 7 failed with debonding too, while the beams with 11 and full straps failed with FRP rupture. Also, they stated that Concrete surface roughness has no significant influence on the performance of both the wet layup or precured FRP sheets, with or without anchorage, and whether the failure was by FRP rupture or debonding, and they recommended intermediate smooth surface.

Neil A. Hoult et al. [55] performed an experimental program for shear strengthening of T-section RC beam with CFRP U straps instead of strips or continuous sheets. They developed a system where no need to work on the top flange of the beam is required and this is important for existed building where the top flange is for the slab, in addition, increasing the shear strength capacity. They did this by testing seven T-section RC beams where the CFRP straps were inserted through holes that were previously drilled from below the flange, thus no need for access to the top surface. They tested the CFRP straps penetration depth into the compression flange, the concrete strength, the CFRP strap spacing, the presence of holes in the compression flange, and the size of the loading pads were all found to affect the shear capacity. They concluded that the CFRP straps should have enough depth into the compression flange in order to work effectively, the concrete strength and CFRP strap spacing have an influence on the strain level on the CFRP straps which leads to an effect on the load sharing between materials. They evidenced that CFRP strap strengthened beams resulted in a shear load capacity 50% higher than control beams.
2.4.2.6. **U-jackets and full wrapping.** In many cases beams are not only subjected to shear but also torsion and in this case the failure is brittle, Deifalla et al. [56] tested six T-section RC have scale beams under combined shear and torsion two as reference beams and four strengthened with CFRP using different types of anchorage: U-jacket where the top of the flange of the beam is inaccessible and the CFRP is bonded to the web 50 mm below the flange and anchored with steel rods. extended U-jacket is similar to U-jacket but with the fibers bonded to the full web and anchored into the flange in addition into a flange in the corner between web and flange, full wrapping where the full access to all faces of the beam is possible and the fibers wrapped around the beam and anchored into flange, and Combined Full Wrapping with extended U-jacket. They noticed that the strengthened specimens increased the shear and torsion carrying capacity by 71% over the reference beam as well as stiffness and deformability of the beam. The U-jacket anchorage showed the least capacity while the full wrap showed the highest capacity, regarding the extended U-jacket, it has delayed the premature failure.

2.4.2.7. **FRP strips.** Fiber reinforced polymer strips are one of anchorage systems where we put it over an FRP sheet that is used for strengthening RC member. They are installed in the same plane of the FRP sheet and perpendicular to the force direction in FRP. It's evidenced that this type is not relatively effective comparing with other anchoring systems [44].

2.4.3. **Epoxy anchors.**

2.4.3.1. **Grooving method (EBROG and EBRIG).** Grooving method (GM) includes drilling grooves into tension face of concrete beams and filling them with epoxy resin then bonding the FRP sheets over these filled grooves. It is called externally bonded reinforcement on grooves (EBROG). Davood et al. [57] applied the EBROG on a set of small scale beams, eight as a reference beam and the others divided to four groups, one group of beams were strengthened against shear with FRP strips without surface preparation, with surface preparation, with EBR method, and with EBROG method. They found that the Beams strengthened with EBR method showed an increase in the load capacity of 10-13% but the failure still happened with FRP debonding, while beams strengthened with EBROG method showed an increase in the load capacity of 17-23% and the failure has changed to flexural failure. Another study has been done by Davood et al. [58] using EBROG method for Shear strengthening of
small RC rectangular beams, but by using CFRP sheets instead of CFRP strips in the previous study, they concluded that strengthening with CFRP sheets using EBROG method resulted in a 23% increase in the flexural strength of the beams over the control beams. This refers to the increase of contact surface between the surface of concrete and the epoxy. Moreover, premature debonding has been eliminated and the failure mode has changed from shear to flexural failure with ductile behavior. Aiming for a comparison investigation for the bond strength between FRP and concrete with externally bonded reinforcement (EBR) and externally bonded reinforcement on grooves EBROG techniques, Davood et al. [59] carried out a single shear bond test on six prism specimens. They concluded that the EBROG method on prisms has increased the load capacity by 55.5% when compared it with conventional EBR method.

Davood et al. [60] used another method called externally bonded reinforcement in grooves (EBRIG) for strengthening concrete beams against shear where the difference is that here the grooves are in direct contact with the fibers by inserting the fibers into the grooves (see Figure 2-3). The authors compared EBRIG with the conventional EBR method where they used two-sided, U-shape, and full wrapped FRP strips for strengthening. They resulted in that the EBRIG method combined with the full wrapping has the best performance among all the other with an increase in load capacity by 148% over control beams, in addition, they noticed that the EBRIG method has changed the failure mode from a brittle shear failure to flexural.

![Figure 2-3: EBROG and EBRIG methods [57, 61]](image)

2.4.3.2. **Boring method.** The boring method is a new technique developed by Eftekhar et al. [62] which is used for FRP anchoring, it transfers the surface of stresses to a concrete depth similar to grooving method, it’s done by drilling holes into the desired side of the concrete beam as in Figure 2-4. Eftekhar et al. tested a 42-unreinforced concrete rectangular beam under flexure, two rows of holes at a transverse spacing of 40 mm were drilled in the lower side of the concrete specimens. The holes
were 5, 10, 15, and 20 mm in depth and 8, 10, 12, 14, and 16 mm in diameter. The longitudinal spacing between two consecutive holes varied from 32 to 87 mm, and the total numbers of holes in the two rows were 10, 16, 18, or 24. They are divided to three groups, without surface preparation, with conventional surface preparation, and with boring technique. They concluded that the boring enhanced the rupture strength, the ultimate ductility, and increased the flexural capacity of the beams by 35% over the control beams.

Figure 2-4: Boring method [62]

2.4.3.3. **CFRP rope.** El-Saikaly et al. [63] introduced a new type of anchoring CFRP strips in shear strengthening of RC beams called CFRP Rope anchors, it’s made by bundling flexible CFRP strands that held together using a thin tissue net which makes the rope. Then drilling holes through the web at the web-flange intersection where the CFRP ropes are inserted and flared onto the two free ends of the U-wrap scheme, then U-wrap becomes similar to a full-wrap. They concluded that Rope technique increases the shear strength capacity. In addition, strengthening beams with CFRP sheets outperforms that beams strengthened with CFRP L-strips but by using the Rope technique the shear capacity increased by twice for the CFRP L-strips over the CFRP sheets besides eliminating the debonding failure.

2.5. **Addressed Gap**

As mentioned above Davood et al. [57] applied externally bonded reinforcement on grooves (EBROG) on small scale rectangular concrete beams the beam dimensions are 560 × 85 × 70 mm and the author concluded that the EBROG
method increased the shear load capacity more than the conventional EBR method and changed the failure mode from brittle shear failure to a flexural failure. Mostly the results of small-scale specimens do not necessarily represent the overall behavior of full-scale members, hence, this study proposes applying EBROG on large scale rectangular concrete beams deficient in shear.

Eftekhar et al. [62] applied the boring method as an anchorage system for flexural strengthening of small RC beams. In the literature research, this method has not been used in shear strengthening of RC rectangular beams applications. Thus, this study aims to apply boring method for shear strengthening of RC Beams. Table 2-2 summarizes all the anchorage systems that are reviewed above. Table 2-2 summarizes all the anchorage systems that are reviewed above.

Table 2-2: Anchorage systems used for shear strengthening

<table>
<thead>
<tr>
<th>Number</th>
<th>Used anchorage system</th>
<th>Author</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Steel plates and bolts</td>
<td>H.N. Garden et al.[41]</td>
</tr>
<tr>
<td>2</td>
<td>Plate anchors</td>
<td>Jin et al.[42]</td>
</tr>
<tr>
<td>3</td>
<td>U-anchors</td>
<td>Khalifa et al.[45]</td>
</tr>
<tr>
<td>4</td>
<td>FRP anchors or Spike anchors</td>
<td>L. Orton et al. [45, 47, 64]</td>
</tr>
<tr>
<td>5</td>
<td>Near Surface Mounted (NSM) end anchor</td>
<td>Eshwar et al. [38]</td>
</tr>
<tr>
<td>6</td>
<td>Dry U-wrapped CFRP</td>
<td>Galal et al [51]</td>
</tr>
<tr>
<td>7</td>
<td>CFRP straps</td>
<td>Lees et al [52]</td>
</tr>
<tr>
<td>8</td>
<td>CFRP U-straps</td>
<td>Yalim et al. [54]</td>
</tr>
<tr>
<td>9</td>
<td>U-jackets &amp; Full wrapping</td>
<td>Deifalla et al. [56]</td>
</tr>
<tr>
<td>10</td>
<td>FRP strips</td>
<td>Ortega.</td>
</tr>
<tr>
<td>11</td>
<td>Bolted Angles</td>
<td>Tanarslan and Altin</td>
</tr>
<tr>
<td>12</td>
<td>Grooving method (EBROG)</td>
<td>Davood et al.[58]</td>
</tr>
<tr>
<td>13</td>
<td>Boring Method</td>
<td>Eftekhar et al.[62]</td>
</tr>
<tr>
<td>14</td>
<td>CFRP Rope</td>
<td>El-Saikaly et al.[63]</td>
</tr>
</tbody>
</table>
Chapter 3. Experimental Program

The aim of this experimental program is to investigate the feasibility of using bore epoxy anchors and groove epoxy anchors in application of shear strengthening of RC beams. Fifteen RC beams have been tested with different parameters.

3.1. Test Specimens

Fifteen beams are designed and casted and they all designed in a way that the shear failure controls when tested under four point bending. They were divided into two groups having the same section and reinforcement but with different CFRP material type, first group (group S) is strengthened using CFRP sheets while second group was strengthened with CFRP plates, the test matrix for both groups are shown in Table 3-1 and Table 3-2. The samples are named with three letters, the first letter stands for the group name, the second letter is for the anchorage method and the third one describes the parameter that has been varied.

As mentioned above Davood et al. [57] applied externally bonded reinforcement on grooves (EBROG) on small scale rectangular concrete beams the beam dimensions are 560 × 85 × 70 mm and mostly the results of small-scale specimens do not necessarily represent the overall behavior of full-scale members, hence, this study proposes applying EBROG on large scale rectangular concrete beams deficient in shear. Table 3-1 shows the details of the proposed specimens.

In EBROG method, grooves of 10 mm depth, width of 5, 10, and 40 mm is done so that two grooves were placed under each of FRP sheet.

Eftekhar et al. [62] applied the boring method as an anchorage system for flexural strengthening of small RC beams, here, this study aims to use boring method for shear strengthening of RC Beams. In boring method, holes of 10 mm depth, diameter of 15, 25, and 35 mm, and 20 mm spacing (5 holes in raw). Figure 3-1 showes the configurations of the grooves and the holes under the FRP sheet.
Figure 3-1: Grooves and holes under the FRP laminate

Table 3-1: Test matrix for group S

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen</th>
<th>Strengthening technique</th>
<th>FRP Type</th>
<th>Width or diameter (mm)</th>
<th>FRP Width (mm)</th>
<th>FRP Spacing Sr (mm)</th>
<th>Wrapping method</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>C1</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>2</td>
<td>S-EBR</td>
<td>EBR</td>
<td>Sheets</td>
<td>---</td>
<td>50</td>
<td>125.0</td>
<td>Two-sided</td>
</tr>
<tr>
<td>3</td>
<td>SGT</td>
<td>EBROG</td>
<td>Sheets</td>
<td>5</td>
<td>50</td>
<td>125.0</td>
<td>Two-sided</td>
</tr>
<tr>
<td>4</td>
<td>SGM</td>
<td>EBROG</td>
<td>Sheets</td>
<td>10</td>
<td>50</td>
<td>125.0</td>
<td>Two-sided</td>
</tr>
<tr>
<td>5</td>
<td>SGD</td>
<td>EBROG</td>
<td>Sheets</td>
<td>40</td>
<td>50</td>
<td>125.0</td>
<td>Two-sided</td>
</tr>
<tr>
<td>6</td>
<td>SBN</td>
<td>Boring</td>
<td>Sheets</td>
<td>10</td>
<td>50</td>
<td>125.0</td>
<td>Two-sided</td>
</tr>
<tr>
<td>7</td>
<td>SBM</td>
<td>Boring</td>
<td>Sheets</td>
<td>20</td>
<td>50</td>
<td>125.0</td>
<td>Two-sided</td>
</tr>
<tr>
<td>8</td>
<td>SBL</td>
<td>Boring</td>
<td>Sheets</td>
<td>30</td>
<td>50</td>
<td>125.0</td>
<td>Two-sided</td>
</tr>
</tbody>
</table>

Notations:
C = Control, S-EBR = Sheets with EBR method, SGT = Sheets Grooving Thin, SGM = Sheets Grooving Medium, SGD = Sheets Grooving Dense, SBN = Sheets Boring Narrow, SBM = Sheets Boring Medium, SBL = Sheets Boring Large.
Table 3-2: Test matrix for group P

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen</th>
<th>Strengthening technique</th>
<th>FRP type</th>
<th>Width or diameter (mm)</th>
<th>FRP Width (mm)</th>
<th>FRP Spacing Sf (mm)</th>
<th>Wrapping method</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>P-EBR</td>
<td>EBR</td>
<td>plate</td>
<td>---</td>
<td>50</td>
<td>125.0</td>
<td>Two-sided</td>
</tr>
<tr>
<td>2</td>
<td>PGT</td>
<td>EBROG</td>
<td>plate</td>
<td>5</td>
<td>50</td>
<td>125.0</td>
<td>Two-sided</td>
</tr>
<tr>
<td>3</td>
<td>PGM</td>
<td>EBROG</td>
<td>plate</td>
<td>15</td>
<td>50</td>
<td>125.0</td>
<td>Two-sided</td>
</tr>
<tr>
<td>4</td>
<td>PGD</td>
<td>EBROG</td>
<td>plate</td>
<td>45</td>
<td>50</td>
<td>125.0</td>
<td>Two-sided</td>
</tr>
<tr>
<td>5</td>
<td>PBN</td>
<td>Boring</td>
<td>plate</td>
<td>10</td>
<td>50</td>
<td>125.0</td>
<td>Two-sided</td>
</tr>
<tr>
<td>6</td>
<td>PBM</td>
<td>Boring</td>
<td>plate</td>
<td>20</td>
<td>50</td>
<td>125.0</td>
<td>Two-sided</td>
</tr>
<tr>
<td>7</td>
<td>PBL</td>
<td>Boring</td>
<td>plate</td>
<td>30</td>
<td>50</td>
<td>125.0</td>
<td>Two-sided</td>
</tr>
</tbody>
</table>

Notations:
P-EBR = Plates Control, PGT = Plates Grooving Thin, PGM = Plates Grooving Medium, PGD = Plates Grooving Dense, PBN = Plates Boring Narrow, PBM = Plates Boring Medium, PBL = Plates Boring Large.

3.2. Specimens Details

3.2.1. First group (group S). This group includes seven beams that strengthened using CFRP sheets. The beams dimensions are $1840 \times 150 \times 250$ mm, beam span of 1690 mm, clear span of 1550 mm and shear span of 650 mm with shear span to depth ratio $(a/d)$ of 2.51. The beams were reinforced in flexure with $4\Omega16$ mm bars in the tension zone located at 259 mm from the top of the beam. In the compression zone, the beams are reinforced with $2\#12$ mm bars, they are internally reinforced against shear using stirrups in half of the beam and empty on the other side with stirrups of $\Omega8$ every 50 mm (see Figure 3-2 and Figure 3-3). The beams are strengthened with CFRP U wrapped sheets of 50 mm width. One is strengthened using EBR method (SEBR), three are strengthened with grooving method (SGT, SGM, and SGD), and three are strengthened with boring method (SBN, SBM, and SBL). Where SEBR stands for Sheets with EBR method. SGT for Sheets with Grooving method and Thin grooves, SGM for Sheets with Grooving method and medium grooves, SGD for Sheets with Grooving method and dense grooves, SBN for Sheets with Boring method and Narrow bores, SBM for Sheets with Boring method and medium bores, for Sheets with Boring method and large bores.
3.2.2. **Second group (group P).** The specimens here have the same section, reinforcement and strengthening methods except that two-sided CFRP plates are used instead of U-wrapped CFRP Sheets, stating that the letter (P) stands for plates. Figure 3-2 and Figure 3-3 below show the detailing of group S and P while Figure 3-4 illustrated the alignment of the CFRP sheets and plates.

![Figure 3-2: Beams details of group S and P](image)

![Figure 3-3: Reinforcement details](image)

![Figure 3-4: Alignment of the CFRP sheets and plates](image)
3.3. Materials

3.3.1. Concrete material. Three cubes and three cylinders for each sample were tested after 28 days and the compressive strength is shown in Table 3-3. Typical mode of failure for cubes and cylinders is shown in Figure 3-5 and Figure 3-6 respectively.

Table 3-3: Concrete compressive strength

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen</th>
<th>Cube compressive strength ( f_{cu} ) (MPa)</th>
<th>Cylinder compressive strength ( f'_c ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>C</td>
<td>52.7</td>
<td>38.7</td>
</tr>
<tr>
<td>2</td>
<td>S-EBR</td>
<td>57.5</td>
<td>43.2</td>
</tr>
<tr>
<td>3</td>
<td>SGT</td>
<td>55.4</td>
<td>39.8</td>
</tr>
<tr>
<td>4</td>
<td>SGM</td>
<td>52.2</td>
<td>39.2</td>
</tr>
<tr>
<td>5</td>
<td>SGD</td>
<td>58.7</td>
<td>47.0</td>
</tr>
<tr>
<td>6</td>
<td>SBN</td>
<td>51.2</td>
<td>39.9</td>
</tr>
<tr>
<td>7</td>
<td>SBM</td>
<td>45.1</td>
<td>40.1</td>
</tr>
<tr>
<td>8</td>
<td>SBL</td>
<td>45.1</td>
<td>40.1</td>
</tr>
<tr>
<td>9</td>
<td>P-EBR</td>
<td>48.1</td>
<td>42.1</td>
</tr>
<tr>
<td>10</td>
<td>PGT</td>
<td>55.4</td>
<td>37.8</td>
</tr>
<tr>
<td>11</td>
<td>PGM</td>
<td>52.2</td>
<td>38.2</td>
</tr>
<tr>
<td>12</td>
<td>PGD</td>
<td>57.5</td>
<td>43.2</td>
</tr>
<tr>
<td>13</td>
<td>PBN</td>
<td>52.7</td>
<td>39.7</td>
</tr>
<tr>
<td>14</td>
<td>PBM</td>
<td>52.0</td>
<td>39.0</td>
</tr>
<tr>
<td>15</td>
<td>PBL</td>
<td>48.1</td>
<td>42.1</td>
</tr>
</tbody>
</table>
3.3.2. **Steel bars.** Three specimens of the steel bars have been tested and showed mechanical properties as in Table 3-4 with an average yield strength and modulus of elasticity of 590.4 MPa and 199.9 MPa, respectively.

![Concrete cube failure mode](image1)

![Concrete Cylinders failure mode](image2)

**Table 3-4: Properties of steel bars**

<table>
<thead>
<tr>
<th>Specimen number</th>
<th>fy (N/mm²)</th>
<th>E (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>588.5</td>
<td>199.988</td>
</tr>
<tr>
<td>2</td>
<td>587.4</td>
<td>199.971</td>
</tr>
<tr>
<td>3</td>
<td>595.05</td>
<td>200.000</td>
</tr>
<tr>
<td>Average</td>
<td>590.40</td>
<td>199.98</td>
</tr>
</tbody>
</table>
3.3.3. **CFRP and epoxy materials.** Table 3-5 shows the mechanical properties of the CFRP sheets, plates, and epoxy used in this study as stated by the manufacture. CFRP Sheets and plates were bonded externally to the RC beams using epoxy adhesive Mapewrap 31 and Adesilex PG2 respectively. Figure 3-7 shows the CFRP sheet and plate used in this study.

![CFRP sheet](image1)

(a) CFRP sheet

![CFRP plate](image2)

(b) CFRP plate

Figure 3-7: CFRP Sheets and Plates

Adesilex PG2 is the epoxy adhesive used in this study. It consists of Two-component thixotropic epoxy adhesives with long workability time for structural bonds, part A and part B. When creating the adhesive mix, part A and part B needed to be mixed together with a ratio of 3:1 for 3-4 minutes until grey color emerge. After getting this color (see Figure 3-8: Specimens preparation), you must use the epoxy within 50 minutes which is the time needed for epoxy to dry. After 7 days, the epoxy completely hardened.
Table 3-5: mechanical properties of CFRP sheets and plates

<table>
<thead>
<tr>
<th>Material</th>
<th>Thickness (mm)</th>
<th>Modulus of elasticity (GPa)</th>
<th>Ultimate tensile strength (MPa)</th>
<th>Elongation at failure (%)</th>
<th>Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFRP Sheets</td>
<td>0.17</td>
<td>230</td>
<td>4800</td>
<td>2.0</td>
<td>1.79</td>
</tr>
<tr>
<td>CFRP plates</td>
<td>1.40</td>
<td>170</td>
<td>3100</td>
<td>2.0</td>
<td>1.61</td>
</tr>
</tbody>
</table>

3.4. Preparation and Strengthening of Specimens

For specimens preparation, the beam surface was cleaned with a brush, place where the CFRP sheets will be attached was marked, a grinder machine especially made for concrete was used to remove the thin layer of weak concrete on the sides of the beams to make a rough surface for bonding the CFRP sheet and plates, then an air blower was used to remove all the dust.

The wet layup method was then used for bonding the CFRP sheet and plates into the prepared concrete surface by applying a film of an appropriate resin (Mapewrap and Adesllesx PG2) before laying the FRP. The fibers were finally saturated by applying enough resin. Excess resin was removed from the surface to prevent its adverse effect on ultimate rupture strength.

Regarding EBROG and Boring method, the grooving was done using the grinder while the holed were drilled with a driller device. Then the grooves and holes were filled with the epoxy and epoxy layer of 2 mm approximately was done, finally, the CFRP sheet or plate was attached and an epoxy is made above these sheets and plates. The specimens were left for 7 days waiting for the epoxy to gain its strength and then the beams were tested after that. Figure 3-89 shows the specimens preparation steps.
Figure 3-8: Specimens preparation
3.5. **Testing Setup and instrumentation**

All the beams were loaded monotonically using a digitally controlled INSTRON 8806 Universal Testing Machine (UTM) As in Figure 3-9 that has a capacity of 2500 KN at a rate of 10 kN/min.

![Universal testing machine](image)

Figure 3-9: Universal testing machine

Strain gauges on concrete, steel and all the CFRP sheets and plates were attached to capture the strain values. Figure 3-10 shows the strain gauges locations.

![Strain gauges locations](image)

Figure 3-10: Strain gauges locations
Chapter 4. Results and Discussion

This chapter shows the results of the experimental program conducted in this research. All the samples were tested with two-point loading and the applied force and the deflection at the mid span were recorded and plotted, strain graphs of the data recorded from the strain gauges (concrete, steel and CFRP) are shown for each beam. In addition, photos of the crushed beams are provided to demonstrate the modes of failure.

4.1. First Group (Group P)

4.1.1. Control beam (C).

4.1.1.1. Load deflection curves and failure modes. One control beam was tested under four points, it failed in shear as expected with a capacity of 116.96 kN. As shown in Figure 4-1 the shear crack happened in the half of the beam where there are no stirrups, it started from the point of load application up to the support of the beam with an angle of 45 degrees and that is expected as the span over depth ratio (a/d) is 2.51 which is between 2.5 and 5.

Figure 4-2 illustrates the plot of Applied load (kN) versus the deflection (mm) the beam shear capacity reached its maximum of 116.96 kN at 5.24 mm deflection. The failure load which is the drop of the ultimate load to 80% of its value was 93.3 kN with a deflection of 6.34 mm.

(a) Untested control beam
Figure 4-1: Failure mode for the control beam

Figure 4-2: Load-deflection relationship for beam C
4.1.1.2. **Strain results.** Figure 4-3 shows the load versus microstrain response curve for the concrete and steel reinforcement. The maximum strain reached by concrete and steel is 1046, 1044 micro strain which is less than yielding values for concrete and steel.

![Figure 4-3: Load-strain relationship for steel and concrete for beam C](image)

4.1.2. **P-EBR specimen**

4.1.2.1. **Load and deflection results.** This beam was strengthened in shear with CFRP plates using EBR method failed in shear with a diagonal tension crack experiencing higher load and displacement compared to control beams. The ultimate load was 162.90 kN with deflection of 7.10 mm (see Figure 4-4) which is 39.28 % higher than the control beam. At last, because of the weak interface between the CFRP plates and concrete beam, diagonal crack passed under CFRP sheets number 2 and 3 and sudden debonding of CFRP plates happened as shown in Figure 4-5.
Figure 4-4: Load-deflection relationship for beam P-EBR

Figure 4-5: Failure mode for beam P-EBR
4.1.2.2. **Strain results.** Figure 4-6 shows the load versus microstrain response curve for the concrete and steel reinforcement. The maximum strain reached by concrete and steel is 1155.81 and 1390.9 micro strain which is less than yielding values for concrete and steel.

![Load-strain relationship for steel and concrete in beam P-EBR](image)

Figure 4-6: Load-strain relationship for steel and concrete in beam P-EBR

Six strain gages were attached to the CFRP plates to record the strain while applying the load, they were attached at the center of every CFRP plate. CFRP plate number 3 and 2 showed maximum microstrain of 1895.84 and 823.21 which represents 9.47 and 4.11 respectively of the ultimate strain of the CFRP plate (20,000) (see Figure 4-7), this proves that they are the most stressed sheets because the diagonal crack passed under them leading them to deboned, while the other CFRP plates (number 1, 4, 5 and 6) showed a small microstrain values.
4.1.3. P-G-T specimen

4.1.3.1. Load and deflection results. This beam was strengthened in shear with CFRP plates using groove-epoxy anchorage with two thin grooves. It failed in shear with a diagonal tension crack experiencing higher load and displacement compared to control beams. The ultimate load was 241.68 kN with deflection of 8.77 mm (see Figure 4-8) which is 106.63 % higher than the control beam. At last, because of the weak interface between the CFRP plates and concrete beam, diagonal crack passed under CFRP plate number 2 and 3 and delamination of CFRP plate number 2 happened as shown in Figure 4-9.
4.1.3.2. Strain results. Figure 4-10 shows the load versus microstrain response curve for the concrete and steel reinforcement. The maximum strain reached by concrete and steel is 3073.1 and 2388.0 micro strain which is less than yielding values for concrete and steel.
Six strain gages were attached to the CFRP plates to record the strain while applying the load, they were attached at the center of every CFRP plate. CFRP plate number 3 and 2 showed maximum microstrain of 1133.69 and 443.3 which represents 5.67 and 2.22 respectively of the ultimate strain of the CFRP plate (20,000) (see Figure 4-11), this proves that they are the most stressed plates because the diagonal crack passed under them leading them to delaminate, while the other CFRP plates (number 1, 4, 5 and 6) showed a small microstrain values.
4.1.4. P-G-M specimen

4.1.4.1. Load and deflection results. This beam was strengthened in shear with CFRP plates using groove-epoxy anchorage with two medium grooves. It failed in shear with a diagonal tension crack experiencing higher load and displacement compared to control beams. The ultimate load was 248.30 kN with deflection of 7.79 mm (see Figure 4-12) which is 1012.629 % higher than the control beam, I also showed larger load than PGM sample which means that the groove width has an effect in delaying the debonding failure and At last, because of the weak interface between the CFRP plates and concrete beam, diagonal crack passed under CFRP plate number 2 and 3 and delamination of CFRP plate number 2 happened as shown in Figure 4-13.

![Figure 4-12: Load-deflection relationship for beam P-G-M](image.png)
4.1.4.2. Strain results. Figure 4-14 shows the load versus microstrain response curve for the concrete and steel reinforcement. The maximum strain reached by concrete and steel is 2154.2 and 1930.3 microstrain which is less than yielding values for concrete and steel.
Six strain gages were attached to the CFRP plates to record the strain while applying the load, they were attached at the center of every CFRP plate. CFRP plate number 3 and 2 showed maximum microstrain of 859.66 and 873.5 which represents 4.3% of the ultimate strain of the CFRP plate (20,000) (see Figure 4-15), this proves that they are the most stressed plates because the diagonal crack passed under them leading them to delaminate, while the other CFRP plates (number 1, 4, 5 and 6) showed a small microstrain values.
4.1.5. P-G-D specimen

4.1.5.1. Load and deflection results. This beam was strengthened in shear with CFRP plates using groove-epoxy anchorage with one dense grooves. It failed in shear with a crack started from the load point and going through the top of the beam until the third CFRP plate then it went down diagonally and continues through the beam bottom until it reached the edge experiencing higher load and displacement compared to control beams. The ultimate load was 176.30 kN with deflection of 7.72 mm (see Figure 4-16) which is 50.74 % only higher than the control beam but less than PGT and PGM which means that two thin or medium grooves performs better than one dense groove in delaying the debonding of CFRP plates. Eventually, delamination of CFRP plate number 2 and 3 happened as shown in Figure 4-17.

![Figure 4-16: Load-deflection relationship for beam P-G-D](image)

Figure 4-16: Load-deflection relationship for beam P-G-D
Figure 4-17: Failure mode for beam P-G-D
4.1.5.2. Strain results. Figure 4-18 shows the load versus microstrain response curve for the concrete and steel reinforcement. The maximum strain reached by concrete and steel is 1248.9 and 1390.9 microstrain which is less than yielding values for concrete and steel.

![Load-strain relationship for steel and concrete in beam P-G-D](image)

Six strain gages were attached to the CFRP plates to record the strain while applying the load, they were attached at the center of every CFRP plate. CFRP plate number 3 and 2 showed maximum microstrain of 2456.5 and 1070.9 which represents 12.3 and 5.4 % of the ultimate strain of the CFRP plate (20,000) (see Figure 4-19), this proves that they are the most stressed plates because the diagonal crack passed under them leading them to delaminate, while the other CFRP plates (number 1, 4, 5 and 6) showed a small microstrain values.
4.1.6. P-B-N specimen

4.1.6.1. Load and deflection results. This beam was strengthened in shear with CFRP plates using bore-epoxy anchorage with five narrow bores. It failed in shear with a crack started from the load point and going down until the third CFRP plate then continues through the beam bottom until it reached the edge experiencing higher load and displacement compared to control beams. The ultimate load was 178.1 kN with deflection of 6.185 mm (see Figure 4-20) which is 52.27 % only higher than the control beam which means that the narrow epoxy bores helped in delaying the debonding of CFRP plates. At last debonding of CFRP plate number 2 was noticed as in Figure 4-21.
Figure 4-20: Load-deflection relationship for beam P-B-N

(a) Untested beam

(b) Tested beam

Figure 4-21: Failure mode for beam P-B-N
**4.1.6.2. Strain results.** Figure 4-22 shows the load versus microstrain response curve for the concrete and steel reinforcement. The maximum strain reached by concrete and steel is 1098.8 and 2260.2 microstrain which is less than yielding values for concrete and steel.

![Figure 4-22: Load-strain relationship for steel and concrete in beam P-B-N](image)

Six strain gages were attached to the CFRP plates to record the strain while applying the load, they were attached at the center of every CFRP plate. CFRP plate number 3 and 2 showed maximum microstrain of 5502.0 and 171.8 which represents 27.5 and 0.9 % of the ultimate strain of the CFRP plate (20,000) (see Figure 4-23), this proves that they are the most stressed plates because the diagonal crack passed under them leading FRP to debond, while the other CFRP plates (number 1, 4, 5 and 6) showed a small microstrain values.

![Figure 4-23: Load-strain relationship for CFRP plates in beam P-B-N](image)
4.1.7. P-B-M specimen

4.1.7.1. Load and deflection results. This beam was strengthened in shear with CFRP plates using bore-epoxy anchorage with five medium bores. It failed in shear with a crack started from the load point and going down until the third CFRP plate then continues through the beam bottom until it reached the edge experiencing higher load and displacement compared to control beams. The ultimate load was 182.01 kN with deflection of 6.50 mm (see Figure 4-24) which is 55.62 % only higher than the control beam which shows that bore-epoxy anchorage is good in delaying the debonding of CFRP plates. Ultimately, delamination of CFRP plate number 2 and 3 happened as in Figure 4-25.

![Figure 4-24: Load-deflection relationship for beam P-B-M](image)

(a) Untested beam
4.1.7.2. Strain results. Figure 4-26 shows the load versus microstrain response curve for the concrete and steel reinforcement. The maximum strain reached by concrete and steel is 1487.2 and 1302.6 microstrain which is less than yielding values for concrete and steel.

![Figure 4-26: Load-strain relationship for steel and concrete in beam P-B-M](image)

Six strain gages were attached to the CFRP plates to record the strain while applying the load, they were attached at the center of every CFRP plate. CFRP plate number 3 and 2 showed maximum microstrain of 1836.9 and 360.5 which represents 9.2 and 1.8 % of the ultimate strain of the CFRP plate (20,000) (see Figure 4-27), this
proves that they are the most stressed plates because the diagonal crack passed under them leading FRP to delaminate, while the other CFRP plates (number 1, 4, 5 and 6) showed a small microstrain values.

Figure 4-27: Load-strain relationship for CFRP plates in beam P-B-M

4.1.8. P-B-L specimen

4.1.8.1. Load and deflection results. This beam was strengthened in shear with CFRP plates using bore-epoxy anchorage with five large bores. It failed in shear with a crack started from the load point and going down until the fourth CFRP plate then continues through the beam bottom until it reached the edge experiencing higher load and displacement compared to control beams. The ultimate load was 196.06 kN with deflection of 10.83 mm (see Figure 4-28) which is 67.63% only higher than the control beam, which proves that large epoxy bores was able to delay the debonding failure of CFRP plates and the bore diameter has an effect of increasing the load capacity. At last, delamination of CFRP plate number 2 and 3 happened as in Figure 4-29.
Figure 4-28: Load-deflection relationship for beam P-B-L

Figure 4-29: Failure mode for beam P-B-L

(a) Untested beam

(b) Tested beam
4.1.8.2. Strain results. Figure 4-30 shows the load versus microstrain response curve for the concrete and steel reinforcement. The maximum strain reached by concrete and steel is 2078.9 and 1929.9 microstrain which is less than yielding values for concrete and steel.

![Load-strain relationship for steel and concrete in beam P-B-L](image)

Six strain gages were attached to the CFRP plates to record the strain while applying the load, they were attached at the center of every CFRP plate. CFRP s plate number 3 and 2 showed maximum microstrain of 1489.9 and 1938.6 which represents 7.5 and 7.1 % of the ultimate strain of the CFRP plate (20,000) (see Figure 4-31), this proves that they are the most stressed plates because the diagonal crack passed under them leading FRP to delaminate, while the other CFRP plates (number 1, 4, 5 and 6) showed a small microstrain values.
Second Group (Group S)

4.2.1. S-EBR specimen.

4.2.1.1. Load and deflection results. This beam was strengthened in shear with CFRP sheets using EBR method failed in shear with a diagonal tension crack experiencing higher load and displacement compared to control beams. The ultimate load was 190.04 kN with deflection of 8.85 mm (see Figure 4-32) which is 62.48 % higher than the control beam. At last, because of the weak interface between the CFRP sheets and concrete beam, diagonal crack passed under CFRP sheets number 2 and 3 and sudden debonding of these sheets happened as shown in Figure 4-33.
Figure 4-33: Failure mode for beam S-EBR

(a) Untested beam

(b) Tested beam

(c) Tested beam
\textbf{4.2.1.2. Strain results.} Figure 4-34 shows the load versus microstrain response curve for the concrete and steel reinforcement. The maximum strain reached by concrete and steel is 1027.8 and 1709.9 micro strain which is less than yielding values for concrete and steel.

![Load-strain relationship for steel and concrete in beam S-EBR](image_url)

Six strain gages were attached to the CFRP sheets to record the strain while applying the load, they were attached at the center of every CFRP sheet. CFRP sheet number 3 and 2 showed maximum microstrain of 4740.36 and 1878.8 which represents 23.7 and 9.4 respectively of the ultimate strain of the CFRP sheet (20,000) (see Figure 4-35), this proves that they are the most stressed sheets because the diagonal crack passed under them leading them to deboned, while the other CFRP sheets (number 1, 4, 5 and 6) showed a small microstrain values.
4.2.2. S-G-T specimen

4.2.2.1. Load and deflection results. This beam was strengthened in shear with CFRP plates using groove-epoxy anchorage with two thin grooves. It failed in shear with a crack started from the load point and going down until the third CFRP sheet and then continued through the beam bottom until it reached the edge experiencing higher load and displacement compared to control beams. The ultimate load was 240.60 kN with deflection of 7.35 mm (see Figure 4-36) which is 105.71% higher than the control beam. It’s noticed that the thin groove-epoxy anchors helped in delaying the debonding of CFRP sheets. At last, diagonal crack passed under CFRP plate number 2 and 3 and delamination of CFRP sheet number 2 happened as in Figure 4-37.
Figure 4-36: Load-deflection relationship for beam S-G-T

(a) Untested beam

(b) Tested beam
4.2.2.2. Strain results. Figure 4-38 shows the load versus microstrain response curve for the concrete and steel reinforcement. The maximum strain reached by concrete and steel is 1987.6 and 2438.9 microstrain which is less than yielding values for concrete and steel.

Six strain gages were attached to the CFRP sheets to record the strain while applying the load, they were attached at the center of every CFRP sheet. CFRP sheet number 3 and 2 showed maximum microstrain of 4697.1 and 3255.1 which represents...
23.5 and 16.3 % of the ultimate strain of the CFRP sheet (20,000) (see Figure 4-39), this proves that they are the most stressed sheets because the diagonal crack passed under them leading FRP to delaminate, while the other CFRP sheets (number 1, 4, 5 and 6) showed a small microstrain values.

![Figure 4-39: Load-strain relationship for CFRP plates in beam S-G-T](image)

4.2.3. **S-G-M specimen**

4.2.3.1. **Load and deflection results.** This beam was strengthened in shear with CFRP sheets using groove-epoxy anchorage with two medium grooves. It failed in shear with a crack started from the load point and going down after the second sheet and started going down diagonally until the third CFRP sheet then continues through the beam bottom until it reached the edge experiencing higher load and displacement compared to control beams. The ultimate load was 281.89 kN with deflection of 10.93 mm (see Figure 4-40) which is 141.01 % higher than the control beam. It’s noticed that the medium groove-epoxy anchors helped in delaying the debonding of CFRP sheets. Ultimately, delamination of the second and third CFRP sheet happened as in Figure 4-41.
Figure 4-40: Load-deflection relationship for beam S-G-M

(a) Untested beam

(b) Tested beam
4.2.3.2. Strain results. Figure 4-42 shows the load versus microstrain response curve for the concrete and steel reinforcement. The maximum strain reached by concrete and steel is 2360.0 and 5077.7 microstrain which is less than yielding values for concrete and steel.

Figure 4-42: Load-strain relationship for steel and concrete in beam S-G-M
Six strain gages were attached to the CFRP sheets to record the strain while applying the load, they were attached at the center of every CFRP sheet. CFRP sheet number 3 and 2 showed maximum microstrain of 6118.9 and 15833.9 which represents 30.5 and 79.2% of the ultimate strain of the CFRP sheet (20,000) (see Figure 4-43), this proves that they are the most stressed sheets because the diagonal crack passed under them leading FRP to delaminate, while the other CFRP sheets (number 1, 4, 5 and 6) showed a small microstrain values.

4.2.4. S-G-D specimen

4.2.4.1. Load and deflection results. This beam was strengthened in shear with CFRP sheets using groove-epoxy anchorage with one dense grooves. It failed in shear with a crack started from the load point and going through the top of the beam causing the first third CFRP sheets to delaminate then it passed diagonally to the bottom then it continued through the beam bottom until it reached the edge experiencing higher load and displacement compared to control beams. The ultimate load was 214.56 kN with deflection of 8.85 mm (see Figure 4-44) which is 83.45% only higher than the control beam. It’s noticed that the dense groove-epoxy anchors helped in delaying the debonding of CFRP sheets. At last, all first three sheets delaminated as in Figure 4-45.
Figure 4-44: Load-deflection relationship for beam S-G-D

(a) Untested beam

(b) Tested beam
4.2.4.2. Strain results. Figure 4-46 shows the load versus microstrain response curve for the concrete and steel reinforcement. The maximum strain reached by concrete and steel is 1498.9 and 1983.7 microstrain which is less than yielding values for concrete and steel.

Six strain gages were attached to the CFRP sheets to record the strain while applying the load, they were attached at the center of every CFRP sheet. CFRP sheet number 3 and 2 showed maximum microstrain of 7800.9 and 820.9 which represents
39.0 and 4.1 % of the ultimate strain of the CFRP sheet (20,000) (see Figure 4-47), this proves that they are the most stressed sheets because the diagonal crack passed under them leading FRP to delaminate, while the other CFRP sheets (number 1, 4, 5 and 6) showed a small microstrain values.

![Graph showing load-strain relationship for CFRP plates in beam S-G-D](image)

Figure 4-47: Load-strain relationship for CFRP plates in beam S-G-D

### 4.2.5. S-B-N specimen

#### 4.2.5.1. Load and deflection results. This beam was strengthened in shear with CFRP sheets using bore-epoxy anchorage with five narrow bores. It failed in shear with a crack started from the load point and going down until the third CFRP sheet then continues through the beam bottom until it reached the edge experiencing higher load and displacement compared to control beams. The ultimate load was 214.70 kN with deflection of 5.50 mm (see Figure 4-48) which is 83.57% higher than the control beam. The narrow epoxy bores were able to help in delaying the debonding of CFRP sheets. Ultimately, the diagonal crack caused the CFRP sheets number 2 and 3 to delamination as in Figure 4-49.
Figure 4-48: Load-deflection relationship for beam S-B-N

(a) Untested beam

(b) Tested beam

Figure 4-49: Failure mode for beam S-B-N
4.2.5.2. Strain results. Figure 4-50 shows the load versus microstrain response curve for the concrete and steel reinforcement. The maximum strain reached by concrete and steel is 1238.5 and 1771.9 microstrain which is less than yielding values for concrete and steel.

![Figure 4-50: Load-strain relationship for steel and concrete in beam S-B-N](image)

Six strain gages were attached to the CFRP sheets to record the strain while applying the load, they were attached at the center of every CFRP sheet. CFRP sheet number 3 and 2 showed maximum microstrain of 3722.4 and 1022.7 which represents 18.6 and 5.2 % of the ultimate strain of the CFRP sheet (20,000) (see Figure 4-51), this proves that they are the most stressed sheets because the diagonal crack passed under them leading FRP to delaminate. While the other CFRP sheets (number 1, 4, 5 and 6) showed a small microstrain values.
4.2.6. **S-B-M specimen**

4.2.6.1. **Load and deflection results.** This beam was strengthened in shear with CFRP sheets using bore-epoxy anchorage with five medium bores. It failed in shear with a crack started from the load point and going down until the fourth CFRP sheet then continues through the beam bottom until it reached the edge experiencing higher load and displacement compared to control beams. The ultimate load was 229.40 kN with deflection of 8.97 mm (see Figure 4-52) which is 96.14% higher than the control beam. The medium epoxy bores were able to help in delaying the debonding of CFRP sheets. At last, the diagonal crack caused CFRP sheets number 2, 3 and 4 to delaminate as in Figure 4-53.
Figure 4-52: Load-deflection relationship for beam S-B-M

Figure 4-53: Failure mode for beam S-B-M
4.2.6.2. Strain results. Figure 4-54 shows the load versus microstrain response curve for the concrete and steel reinforcement. The maximum strain reached by concrete and steel is 3875.7 and 2396.8 microstrain which is less than yielding values for concrete and steel.

![Figure 4-54: Load-strain relationship for steel and concrete in beam S-B-M](image)

Six strain gages were attached to the CFRP sheets to record the strain while applying the load, they were attached at the center of every CFRP sheet. CFRP sheet number 3 and 2 showed maximum microstrain of 202.7 and 3854.3 which represents 1.01 and 19.3 % of the ultimate strain of the CFRP sheet (20,000) (see Figure 4-55), this proves that they are the most stressed sheets because the diagonal crack passed under them leading FRP to delaminate, while the other CFRP sheets (number 1, 4, 5 and 6) showed a small microstrain values.
4.2.7. S-B-L specimen

4.2.7.1. Load and deflection results. This beam was strengthened in shear with CFRP sheets using bore-epoxy anchorage with five large bores. It failed in shear with a crack started from the load point and going down until the third CFRP sheet then continues through the beam bottom until it reached the edge experiencing higher load and displacement compared to control beams. The ultimate load was 258.66 kN with deflection of 9.28 mm (see Figure 4-56) which is 121.15 % higher than the control beam. The narrow epoxy bores were able to help in delaying the debonding of CFRP sheets. Ultimately, the diagonal crack caused the CFRP sheets number 2, 3 and 4 to delamination as in Figure 4-57.
Figure 4-56: Load-deflection relationship for beam S-B-L

(a) Untested beam

(b) Tested beam

Figure 4-57: Failure mode for beam S-B-L
**4.2.7.2. Strain results.** Figure 4-58 shows the load versus microstrain response curve for the concrete and steel reinforcement. The maximum strain reached by concrete and steel is 1098.8 and 2260.2 microstrain which is less than yielding values for concrete and steel.

![Figure 4-58: Load-strain relationship for steel and concrete in beam S-B-L](image)

Six strain gages were attached to the CFRP sheets to record the strain while applying the load, they were attached at the center of every CFRP sheet. CFRP sheet number 3 and 2 showed maximum microstrain of 5502.0 and 171.8 which represents 27.5 and 0.9 % of the ultimate strain of the CFRP sheet (20,000) (see Figure 4-59), this proves that they are the most stressed sheets because the diagonal crack passed under them leading FRP to delaminate, while the other CFRP sheets (number 1, 4, 5 and 6) showed a small microstrain values.

![Figure 4-59: Load-strain relationship for CFRP plates in beam S-B-L](image)
4.3. Summary of Results

Table 4-1 presents summary for all the experimental results; load, deflection at mid span and the percentage increase over the control beam.

Table 4-1: Summary of experimental results:

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen</th>
<th>Load (KN)</th>
<th>Deflection (mm)</th>
<th>Capacity percent Increase over C (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>C</td>
<td>116.96</td>
<td>5.24</td>
<td>--</td>
</tr>
<tr>
<td>2</td>
<td>P-EBR</td>
<td>162.90</td>
<td>7.10</td>
<td>39.28</td>
</tr>
<tr>
<td>3</td>
<td>PGT</td>
<td>241.68</td>
<td>8.77</td>
<td>106.63</td>
</tr>
<tr>
<td>4</td>
<td>PGM</td>
<td>248.30</td>
<td>7.79</td>
<td>112.29</td>
</tr>
<tr>
<td>5</td>
<td>PGD</td>
<td>176.30</td>
<td>7.72</td>
<td>50.74</td>
</tr>
<tr>
<td>6</td>
<td>PBN</td>
<td>178.1</td>
<td>6.185</td>
<td>52.27</td>
</tr>
<tr>
<td>7</td>
<td>PBM</td>
<td>182.01</td>
<td>6.5</td>
<td>55.62</td>
</tr>
<tr>
<td>8</td>
<td>PBL</td>
<td>196.06</td>
<td>10.83</td>
<td>67.63</td>
</tr>
<tr>
<td>9</td>
<td>S-EBR</td>
<td>190.04</td>
<td>5.85</td>
<td>62.48</td>
</tr>
<tr>
<td>10</td>
<td>SGT</td>
<td>240.60</td>
<td>7.35</td>
<td>105.71</td>
</tr>
<tr>
<td>11</td>
<td>SGM</td>
<td>281.89</td>
<td>10.93</td>
<td>141.01</td>
</tr>
<tr>
<td>12</td>
<td>SGD</td>
<td>214.56</td>
<td>8.85</td>
<td>83.45</td>
</tr>
<tr>
<td>13</td>
<td>SBN</td>
<td>214.70</td>
<td>5.50</td>
<td>83.57</td>
</tr>
<tr>
<td>14</td>
<td>SBM</td>
<td>229.40</td>
<td>8.97</td>
<td>96.14</td>
</tr>
<tr>
<td>15</td>
<td>SBL</td>
<td>258.66</td>
<td>9.28</td>
<td>121.15</td>
</tr>
</tbody>
</table>

4.4. Discussion of Results

4.4.1. C, S-EBR and P-EBR beams. Figure 4-60 shows load and deflection relationship for the control beam (C), the beam with EBR strengthening method using CFRP plates (P-EBR) and the beam with EBR strengthening method using CFRP sheets (S-EBR), the last one showed larger load capacity than P-EBR and C.
Figure 4-60: Load-deflection relationship for C, S-EBR and P-EBR beams

4.4.2. PGT, PGM and PGD beams. Figure 4-61 shows load and deflection relationship for the beams with groove-epoxy anchorage strengthening method using CFRP plates (PGT, PGM and PGD), the two medium grooves showed better response than the thin and dense groove with larger load capacity (see Figure 4-62), while the two thin grooves performed better than the dense groove. All the beams used groove-epoxy anchorage method was better than the conventional method (EBR).

Figure 4-61: Load-deflection relationship for PGT, PGM and PGD beams
4.4.3. **PBN, PBM and PBL beams.** Figure 4-63 shows load and deflection relationship for the beams with bore-epoxy anchorage strengthening method using CFRP plates (PBN, PBM and PBL), the large bores experienced the maximum load along with the maximum ductility among the three specimens which proves that the larger the hole the better response (see Figure 4-64), No significant difference was noticed in samples with narrow and medium bores. All the beams used bore-epoxy anchorage method was better than the conventional method (EBR).

![Figure 4-62: Comparison for load capacity for C, P-EBR, PGT, PGM and PGD](image1)

![Figure 4-63: Load-deflection relationship for PBN, PBM and PBL beams](image2)
4.4.4. **SGT, SGM and SGD beams.** Figure 4-65 shows load and deflection relationship for the beams with groove-epoxy anchorage strengthening method using CFRP sheets (SGT, SGM and SGD), the two medium grooves showed better response than the thin and dense groove with larger load capacity (see Figure 4-66), while the two thin grooves performed better than the dense groove. All the beams used groove-epoxy anchorage method was better than the conventional method (EBR).
4.4.5. **SBN, SBM and SBL beams.** Figure 4-67 shows load and deflection relationship for the beams with bore-epoxy anchorage strengthening method using CFRP sheets (SBN, SBM and SBL), the large bores experienced the maximum load along with the maximum ductility among the three specimens which proves that the larger the hole the better response (see Figure 4-68). All the beams used bore-epoxy anchorage method was better than the conventional method (EBR).
4.5. Effect of Bore Diameter in bore-Epoxy Anchorage Method

The bore diameter has been varied in the bore-epoxy anchorage method with three values 10, 20 and 30 mm diameter. From Figure 4-69 we can notice that the more bore diameter, the more load capacity obtained. Also, it’s noticed that the bore diameter has the same effect in CFRP sheets and plates which means it can be used effectively with both materials.
4.6. Effect of Groove Width in Groove-Epoxy Anchorage Method

Figure 4-70 illustrates the effect of increasing the groove width on the load capacity. Groove width of 5, 10 and 40 mm was studied. The load capacity increased when the groove width increased from 5 to 10 mm in both CFRP sheets and plates, but it decreased when the groove width reached 40 mm.

![Figure 4-70: Effect of groove width in load capacity](image)

Figure 4-70: Effect of groove width in load capacity
Chapter 5. Theoretical Prediction

This chapter presents validation the experimental results of three specimens with estimating the ultimate load carrying capacity using four different equations developed by four design guidelines and codes. The codes are, Guide for the Design and Construction ofExternally Bonded FRP Systems for Strengthening Concrete Structures (ACI-440.2R-08) [11], Design and Construction ofBuilding Components with Fiber Reinforced Polymers (CAN/CSA-S806-02) [65], International federation of structural concrete (FIB 14) [66] and the concrete society in the UK (TR55) [67].

5.1. ACI-440.2R-08 Equation

ACI Committee 440-2R (2008) [11] presents guidelines for using FRP in flexure and shear application, these guidelines are based on a limit-states-design that sets limits on both serviceability and ultimate limit states. They recommend using additional strength reduction factors (ψf) on top of the nominal strength reduction factors (φ) to account for the unknowns about FRP systems. They treat FRP material as a linear elastic material when it fails. Therefore, the design modulus of elasticity is determined by Hooke’s Law:

\[ E_f = \frac{f_e}{\varepsilon_{fe}} \]

In FRP design the nominal shear capacity of a strengthened RC member multiplied by a strength –reduction factor should be greater than the required shear strength of the member.

\[ \phi V_n \geq V_u \]

The nominal shear strength of the FRP strengthened RC member is calculated by summing the individual shear strength contributions from the concrete, steel stirrup reinforcement, and FRP reinforcement. An additional strength reduction factor is applied to the strength contribution of the FRP reinforcement depending on the type of wrapping scheme applied. Figure 5-1 illustrates the details and notations for FRP systems.

\[ \phi V_n = \phi (V_c + V_s + \psi_f V_f) \]

Where:

\[ V_c = 2 \times \sqrt{f_c} \times b_w \times d \]
\[ V_s = \frac{A v \times F_y \times d}{S} \]

\[ V_f = \frac{A_f v \times f_{fe} \times d_f \times (\sin \alpha + \cos \alpha)}{S_f} \]

\[ A_f v = 2n t \times w_f \]

\[ F_{fe} = \varepsilon_{fe} \times E_f \]

The additional strength-reduction factor \( \psi_f \) is 0.95 for complete wrapping schemes and 0.85 for U-wrap and side wrap schemes.

The deboning failure mode (brittle failure mode) takes place at a strain below the FRP rupture strain, ACI specified a maximum attainable strain of 0.4% in the FRP wraps which have been adopted from Khalifa et al. [68] who proposed a maximum strain limit of 0.4% to maintain the shear integrity of the concrete and prevent loss of aggregate interlock. For completely wrapped elements:

\[ \varepsilon_{fe} = 0.004 \leq 0.75\varepsilon_{fu} \]

However, for U-wraps and side applications the ACI report introduces a bond-reduction coefficient \( K_v \) as these FRP applications are susceptible to the debonding failure mode. The bond reduction coefficient \( K_v \) that was experimentally derived is dependent upon several factors including concrete strength, type of wrapping scheme used, and stiffness of the FRP laminate.

\[ E_{fe} = K_v \times \varepsilon_{fu} \leq 0.004 \]

Where:

\[ K_v = \frac{K_1 K_2 L_e}{468 \varepsilon_{fu}} \]

\[ L_e = \frac{2500}{(n t f E_f)^{0.58}} \]
\[ Kv = \left( \frac{f'c}{4000} \right)^{\frac{2}{3}} \]

\[ Kv = \frac{df - Le}{df} \text{ for } U - \text{wrap} \]

\[ Kv = \frac{df - 2Le}{df} \text{ for two sides} \]

Sample calculation for the P-EBR sample.

\[ V_c = 0.17 \sqrt{f'c' \text{ bwd}} = 0.17 \times \sqrt{39.0 \times 200 \times 259 \times 10^{-3}} = 57.14 \text{ kN} \]

\[ \varepsilon_{fu} = C_E \times \varepsilon_{fu'} = 0.95 \times \frac{3100}{170000} = 0.017 \]

\[ K_1 = \left( \frac{f'c}{27} \right)^{\frac{2}{3}} = \left( \frac{42.1}{27} \right)^{\frac{2}{3}} = 1.35 \]

\[ L_e = \frac{23300}{(n_f \times t_f \times E_f)^{0.58}} = \frac{23300}{(1 \times 1.4 \times 170000)^{0.58}} = 17.74 \]

\[ K_2 = \frac{d_{fv} - 2L_e}{d_{fv}} = \frac{259 - 2 \times 17.74}{259} = 0.932 \]

\[ K_v = \frac{K_1 K_2 L_e}{11900 \varepsilon_{fu}} = \frac{1.35 \times 0.032 \times 17.74}{11900 \times 0.017} = 0.108 \]

\[ \varepsilon_{fe} = \text{Min}(0.004, K_v \times \varepsilon_{fu}) = 0.00187 \]

\[ f_{fe} = \varepsilon_{fe} \times E_f = 0.00187 \times 170000 = 322.29 \]

\[ V_f = \frac{A_{fv} \times f_{fe} \times d_f \times (\sin\alpha + \cos\alpha)}{S_f} \]

\[ V_f = \frac{1.4 \times 50 \times 322.29 \times 259 \times (\sin\alpha + \cos\alpha)}{125} = 46.70 \text{ kN} \]

\[ V_n = V_c + V_f = 57.14 \times 2 + 46.70 = 160.90 \text{ kN} \]

Table 5-1 and Figure 5-2 illustrates the predicted ultimate load \((V_{pre})\) and experimental ultimate load \((v_{exp})\). The load capacity for the control beam (C) is calculated based on equation from ACI 318-11 design code. For specimens (P-EBR and S-EBR) the predicted load is calculated based on ACI 440-2R-08 equations. All the predicted values were less than the experimental ones. For P-EBR and S-EBR, the predicted values showed a difference of 1.6 and 26.20 % respectively, this means that ACI 440-2R-08 equations are conservative in estimating the shear load capacity.
Table 5-1: Predicted Load by ACI equations and experimental results

<table>
<thead>
<tr>
<th>Designation</th>
<th>Capacity ACI equation (kN)</th>
<th>Capacity Experimental (kN)</th>
<th>Difference %</th>
<th>$V_{\text{exp}}/V_{\text{pre}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>109.98</td>
<td>116.96</td>
<td>5.97</td>
<td>1.06</td>
</tr>
<tr>
<td>P-EBR</td>
<td>160.90</td>
<td>162.92</td>
<td>1.6</td>
<td>1.01</td>
</tr>
<tr>
<td>S-EBR</td>
<td>140.30</td>
<td>190.04</td>
<td>26.20</td>
<td>1.35</td>
</tr>
</tbody>
</table>

Figure 5-2: Comparison between the predicted Load (ACI) and experimental load

5.2. CAN/CSA-S806-02 Equation

CSA S806-02 proposed the equations below to determine the shear strength of FRP-reinforced concrete beams. The factored shear resistance shall be determined by the following equations:

$$V_r = V_c + V_s + V_f \leq V + 0.6 \times \lambda \sqrt{f'c} \times bw \times d$$

$$V_c = 0.2 \lambda \sqrt{f'c} \times bw \times d$$

$$V_s = \frac{\phi_s \times Av \times Fy \times d}{S}$$

$$V_f = \frac{\phi_f \times Af \times Ef \times \epsilon_f \times df}{S_f}$$

In the absence of more precise information, the value of $\epsilon_f$ may be conservatively assumed to be as follows:
1. for U-shaped wrap continuous around the bottom of the web: 4000µε
2. for side bonding to the web (and only in cases where sufficient development length cannot be provided 2000µ

Sample calculation for the P-EBR sample.

\[ V_c = 0.2\beta \sqrt{f_{c'}} \text{ bwd} = 0.2 \times 1 \times \sqrt{39.0} \times 200 \times 259 \times 10^{-3} = 64.69 \text{ kN} \]

\[ V_f = \frac{A_f \times E_f \times d_f \times \varepsilon_f}{S_f} \]

\[ \varepsilon_f = 2000\mu \]

\[ V_f = \frac{0.85 \times 1.4 \times 50 \times 170 \times 2000 \times 10^{-6} \times 0.75 \times 300}{125} = 36.40 \text{ kN} \]

\[ V_n = V_c + V_f = 64.69 \times 2 + 36.40 = 165.78 \text{ kN} \]

Table 5-2 and Figure 5-3 illustrates the predicted ultimate load (Vpre) and experimental ultimate load (Vexp). The load capacity for the control beam (C) is calculated based on equation from CSA A23.3-04 design code. For specimens (P-EBR and S-EBR) the predicted load is calculated based on CAN/CSA-S806-02 equations. For P-EBR and S-EBR, the predicted values showed a difference of -5.16 and 2.07 % respectively, this means that CAN/CSA-S806-02 equations are more accurate in estimating the shear load capacity.

**Table 5-2 Predicted Load by CSA equations and experimental results**

<table>
<thead>
<tr>
<th>Designation</th>
<th>Capacity CSA-S806-02 equation (kN)</th>
<th>Capacity Experimental (kN)</th>
<th>Difference %</th>
<th>Vexp/ Vpre</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>129.40</td>
<td>116.96</td>
<td>-10.64</td>
<td>0.90</td>
</tr>
<tr>
<td>P-EBR</td>
<td>165.78</td>
<td>162.92</td>
<td>-1.73</td>
<td>0.95</td>
</tr>
<tr>
<td>S-EBR</td>
<td>186.11</td>
<td>190.04</td>
<td>2.07</td>
<td>1.02</td>
</tr>
</tbody>
</table>
5.3. **International Federation of Structural Concrete (FIB 14)**

International federation of structural concrete (FIB 14) explains the design and use of externally bonded fiber reinforced polymer reinforcement (FRP EBR) for reinforced concrete structures. The factored shear resistance shall be determined by the following equations:

\[
V_{fd} = 0.9 \times \varepsilon_{fde} \times E_{fu} \times \rho_f \times bw \times d(\cot \theta + \cot \alpha) \sin \alpha
\]

\[
\varepsilon_{fe} = \min \left[ 0.65 \left( \frac{f_{cm}^2}{E_{fu} \times \rho_f} \right)^{0.56} \times 10^{-3}, 0.17 \left( \frac{f_{cm}^2}{E_{fu} \times \rho_f} \right)^{0.3} \varepsilon_{fu} \right]
\]

\[
\varepsilon_{fke} = K \times \varepsilon_{fe}
\]

\[
\varepsilon_{fd,e} = \frac{\varepsilon_{fke}}{\gamma_f}
\]

\[
\rho_f = \left( 2 \times \frac{t_f}{bw} \right) \left( \frac{b_f}{s_f} \right)
\]

Where:

- \(V_{fd}\): The FRP contribution to shear capacity
- \(\varepsilon_{fde}\): design value of the effective FRP strain
- \(E_{fu}\): Elastic modulus of FRP in the principle fiber orientation
- \(\rho_f\): FRP reinforcement ratio
- \(bw\): Minimum width of cross section over the effective depth
bw: Width of the strips or sheets of the bonded reinforcement (mm)
sf: Spacing between the strips of the bonded reinforcement (mm)
tf: Thickness of bonded reinforcement (plates or sheets) (mm)
d: Effective depth of cross section (mm)
θ: Angle of diagonal crack with respect to the member axis, assumed equal to 45.
α: Angle between principal fiber orientation and longitudinal axis of member
K: reduction factor (k=0.8)
γf: Partial safety factor (if failure involves fracture)

Sample calculation for the P-EBR sample.

\[ V_{fd} = 0.9 \varepsilon_{fd} E_{fu} \rho_f b_w d \]
\[ \rho_f = \left( \frac{2t_f}{b_w} \right) \left( \frac{b_f}{S_f} \right) = \left( \frac{2 \times 0.17}{200} \right) \left( \frac{50}{125} \right) = 6.8 \times 10^{-4} \]
\[ \varepsilon_{fe} = \min \left( 0.65 \left( \frac{f_{cm}}{E_{fu} \rho_f} \right)^{0.56} \times 10^{-3}, \quad 0.17065 \left( \frac{f_{cm}}{E_{fu} \rho_f} \right)^{0.3} \varepsilon_{fu} \right) \]
\[ \varepsilon_{fe} = 0.65 \left( \frac{f_{cm}}{E_{fu} \rho_f} \right)^{0.56} \times 10^{-3} = 0.0134 \]
\[ \varepsilon_{f_k,e} = K \varepsilon_{fe} = 0.8 \times 0.0134 = 0.0107 \]
\[ \varepsilon_{fd,e} = \frac{\varepsilon_{f_k,e}}{\gamma_f} = \frac{0.0107}{1.3} = 8.22 \times 10^{-3} \]
\[ V_{fd} = 0.9 \times 8.22 \times 10^{-3} \times 170 \times 6.8 \times 10^{-4} \times 200 \times 259 = 44.30 \text{ kN} \]
\[ V_n = V_c + V_f = 57.14 \times 2 + 44.30 = 158.58 \text{ kN} \]

Table 5-3 and Figure 5-4 illustrates the predicted ultimate load \( (V_{pre}) \) and experimental ultimate load \( (V_{exp}) \). For specimens (P-EBR and S-EBR) the predicted load is calculated based on FIB 14 equations. All the predicted values were less than the experimental ones. For P-EBR and S-EBR, the predicted values showed a difference of 2.66 and 13.12 % respectively, this means that FIB 14 equations are conservative in estimating the shear load capacity.
Table 5-3 Predicted Load by FIB equations and experimental results

<table>
<thead>
<tr>
<th>Designation</th>
<th>Capacity FIB14 equation (kN)</th>
<th>Capacity Experimental (kN)</th>
<th>Difference %</th>
<th>$V_{exp}/V_{pre}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>109.98</td>
<td>116.96</td>
<td>5.97</td>
<td>1.06</td>
</tr>
<tr>
<td>P-EBR</td>
<td>158.58</td>
<td>162.92</td>
<td>2.66</td>
<td>1.03</td>
</tr>
<tr>
<td>S-EBR</td>
<td>165.10</td>
<td>190.04</td>
<td>13.12</td>
<td>1.15</td>
</tr>
</tbody>
</table>

Figure 5-4: Comparison between the predicted Load (FIB) and experimental load

5.4. The Concrete Society in UK (TR55)

TR55 is the design guidance for strengthening concrete structures using fiber composite materials in UK. The basic principle in the fiber composite behavior that this report adopt is that there composite have a straight line stress-strain response to ultimate with no yielding. The report assumes that the shear strips of FRP effect in the RC beams as the steel stirrups do. Based on some studies, the FRP strain limit in the FRP strips should not exceed 0.4%. The following equations present the prediction of the contribution of FRP:

$$VR_f = \left( \frac{1}{\gamma_{mf}} \right) \times A_f \left( E_{fd} \times \epsilon_{fe} \right) \sin \beta \left( 1 + \cot \beta \right) \left( \frac{d_f}{s_f} \right)$$

$$\epsilon_{fe} = \epsilon_{fu} \left[ 0.5622 \left( E_{fd} \times \rho_f \right)^2 - 10^{-3}, 1.2188 E_{fd} \times \rho_f + 0.778 \right]$$
\[ \varepsilon_{fe} = \frac{0.0042 \left(0.835 \left(f_{cu}\right)^2 \text{wfe}\right)}{(E_{fd} \times \rho f)^{0.58} \times df} \]

\[ \text{Afs} = 2t_f \times \text{wfe} \]

\[ \text{wfe} = df - 2Le \]

\[ \text{Le} = \frac{461.3}{(t_f \times E_{fd})^{0.58}} \]

\[ \varepsilon_{fu} = \left(\frac{\varepsilon_{fk}}{\gamma_{mf}}\right) \]

\[ \rho = \left(2 \times \frac{t_f}{bw}\right) \left(\frac{bf}{sf}\right) \]

Where:

VRf: The FRP contribution to shear capacity
Afs: Area of FRP shear reinforcement
Efd: design Elastic modulus of FRP (GPa)
\(\rho f\): FRP reinforcement ratio
bw: Minimum width of cross section over the effective depth
bw: Width of the strips or sheets of the bonded reinforcement (mm)
wf: Width of the strips or sheets of the bonded reinforcement (mm)
tf: Thickness of bonded reinforcement (plates or sheets) (mm)
d: Effective depth of cross section (mm)
fcu: cube strength of concrete (MPa)
\(\beta\): Angle between principal fiber orientation and longitudinal axis of member
\(\gamma_{mf}\): Partial safety factor for FRP

Sample calculation for the P-EBR sample:

\[ \rho_f = \left(2 \times \frac{t_f}{bw}\right) \left(\frac{bf}{sf}\right) = \left(2 \times \frac{1.4}{200}\right) \left(\frac{50}{125}\right) = 5.6 \times 10^{-3} \]

\[ \text{Le} = \frac{461.3}{(t_f \times E_{fd})^{0.58}} = \frac{461.3}{(1.4 \times 170)^{0.58}} = 65.56 \]

\[ \text{wfe} = df - 2Le = 300 - 2 \times 65.56 = 168.88 \]

\[ \text{Afs} = 2t_f \times \text{wfe} = 2 \times 1.4 \times 168.88 = 472.86 \]

\[ \varepsilon_{fu} = \left(\frac{\varepsilon_{fk}}{\gamma_{mf}}\right) = \left(\frac{0.02}{3.5}\right) = 5.714 \times 10^{-3} \]

\[ \varepsilon_{fe} = 5.714 \times 10^{-3}[0.5622(170 \times 5.6 \times 10^{-3})^2 - 1.2188 \times 170 \times 5.6 \times 10^{-3} + 0.778] = 7.269 \times 10^{-4} \]
\[
\varepsilon_{fe} = \frac{0.0042 \left( 0.835 \left( \frac{41.2}{2} \right)^2 \times 168.88 \right)}{(170 \times 5.6 \times 10^{-3})^{0.58} \times 300} = 0.0243
\]

\[
V_{Rf} = \left( \frac{1}{3.5} \right) \times 472.86 \times (170 \times 7.269 \times 10^{-4}) \times \frac{300}{125} = 40.92 \text{kN}
\]

\[
V_n = V_c + V_{Rf} = 57.14 \times 2 + 40.92 = 155.20 \text{kN}
\]

Table 5-4 and Figure 5-5 illustrates the predicted ultimate load \((V_{pre})\) and experimental ultimate load \((v_{exp})\). For specimens (P-EBR and S-EBR) the predicted load is calculated based on TR55 equations. For P-EBR sample, the predicted value showed a significant difference of -27.74 which means the equation is overestimating the load value and for S-EBR sample the difference is 21.85 % which is too conservative, this means that TR55 equations are not accurate in estimating the shear load capacity.

<table>
<thead>
<tr>
<th>Designation</th>
<th>Capacity TR55 equation (kN)</th>
<th>Capacity Experimental (kN)</th>
<th>Difference %</th>
<th>(V_{exp}/V_{pre})</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>109.98</td>
<td>116.96</td>
<td>5.97</td>
<td>1.06</td>
</tr>
<tr>
<td>P-EBR</td>
<td>155.20</td>
<td>162.92</td>
<td>4.74</td>
<td>1.05</td>
</tr>
<tr>
<td>S-EBR</td>
<td>150.80</td>
<td>190.04</td>
<td>20.65</td>
<td>1.26</td>
</tr>
</tbody>
</table>

Figure 5-5: Comparison between the predicted Load (TR5) and experimental load

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Chapter 6. Summary and Conclusion

In this research, the problem of early debonding of the CFRP sheets and plates has been addressed when used as EBR in shear strengthening application of RC beams. A total of fifteen beams have been prepared, strengthened and tested under four point bending, they were divided into two groups, The first group is strengthened with CFRP plates, it includes one control beam, one beam strengthened using EBR conventional method, three beams strengthened with groove-epoxy anchorage method and three beams with bore-epoxy anchorage method. The second group were strengthened using CFRP sheets. This group has one beam strengthened using EBR conventional method, three beams strengthened with groove-epoxy anchorage method and three beams with bore-epoxy anchorage method. In groove-epoxy anchorage, the groove width has been varied while in bore-epoxy anchorage, the bore diameter has been varied. It can be concluded from this investigation that:

1. Using EBR conventional method showed an increase in the shear capacity along with an increase in the maximum deflection over the control beam with 39.28 % load increase when CFRP plates are used and 62.48 % load increase when CFRP sheets are used.

2. When CFRP plates are used, Groove-epoxy anchors increased the shear capacity up to 112.29 % over the control beam and 52.42 % over the EBR strengthened beam. While when CFRP sheets were used, an increase of 141.01 % over the control beam and 48.36 % over the EBR strengthened beam were observed.

3. In Groove-epoxy anchorage method, the medium grooves showed the best performance among the other grooves; in addition, the two grooves showed better performance than one dense groove.

4. When CFRP plates are used, bore-epoxy anchors increased the shear capacity up to 67.63 % over the control beam and 20.30 % over the EBR strengthened beam. While when CFRP sheets were used, an increase of 121.15 % over the control beam and 36.20 % over the EBR strengthened beam were observed.

5. In bore-epoxy anchorage method, the large bores showed the best performance compared to other bores which confirmed that an increase in the diameter resulted in an increase in the shear capacity.
6. Both Groove-epoxy anchorage and bore-epoxy anchorage methods delayed the debonding of CFRP sheets and plates and also changed the failure into delamination of the CFRP as oppose to conventional EBR method that almost always results in early debonding failure.

7. When comparing Groove-epoxy anchorage with bore-epoxy anchorage, we notice that the former was more effective in delaying the debonding and increasing the load carrying capacity.

8. None of these anchorage methods was able to eliminate the debonding and delamination failure but they increased the utilization of the ultimate strain in the CFRPs.

9. The shear strength predictions calculated using the different design guidelines showed that CAN/CSA-S806-02 equation is the most accurate one when compared with the other codes.

For future studies, many parameters can be investigated such as the groove depth in Groove-epoxy anchorage method and the bore depth in bore-epoxy anchorage method, which were kept constant in this study. Conducting more experimental work investigating these anchorage methods can be used in improving the existing equations by the different codes to capture the effect of these epoxy anchorage methods. Moreover, Finite Element (FE) models can be developed to study the performance of the grooves and bores and give a better understanding of the anchorage processes.
References


[57] D. Mostofinejad and A. Tabatabaei Kashani, "Experimental study on effect of EBR and EBROG methods on debonding of FRP sheets used for shear..."


Vita

Khalid Mohamed was born in 1993, in Sudan. He received his primary and secondary education in Singa, Sudan. He received his B.Sc. degree in Civil Engineering from University of Khartoum in 2014. From 2014 to 2016, he worked as a Teaching Assistant at Civil Engineering department at the University of Khartoum.

In February 2016, he joined the Civil Engineering master's program in the American University of Sharjah. He was working as a graduate teaching assistant during his master's study, he co-authored one paper under title of “Using bore-epoxy anchorage to delay debonding of CFRP plates strengthened concrete beams” which was presented in advanced materials, design and manufacturing international conference in Dubai 2018. His research interests are in strengthening of reinforced concrete members with FRP materials.