6-2016

Performance of Reinforced Concrete Beams Strengthened with Mechanically-Fastened Composite System under Corrosive Environment

Amna Zaid Suwaid Al Nassibi

Follow this and additional works at: https://scholarworks.uaeu.ac.ae/all_theses

Part of the Civil Engineering Commons

Recommended Citation
https://scholarworks.uaeu.ac.ae/all_theses/141

This Thesis is brought to you for free and open access by the Electronic Theses and Dissertations at Scholarworks@UAEU. It has been accepted for inclusion in Theses by an authorized administrator of Scholarworks@UAEU. For more information, please contact fadl.musa@uaeu.ac.ae.
United Arab Emirates University  
College of Engineering  
Department of Civil and Environmental Engineering  

Performance of Reinforced Concrete Beams Strengthened with Mechanically-Fastened Composite System under Corrosive Environment  

Amna Zaid Suwaid Al Nassibi  

This thesis is submitted in partial fulfillment of the requirements for the Master of Science in Civil Engineering  

Under the direction of  

Dr. Tamer El-Maaddawy (Principal Supervisor)  
Associate Professor  
Department of Civil and Environmental Engineering  
United Arab Emirates University  

Dr. Amr El-Dieb (Co-advisor)  
Professor  
Department of Civil and Environmental Engineering  
United Arab Emirates University  

June 2013
DECLARATION OF ORIGINAL WORK

I, Amna Zaid Suwaid Al Nassibi, the undersigned, a graduate student at the United Arab Emirates University (UAEU) and the author of the thesis titled “Performance of Reinforced Concrete Beams Strengthened with Mechanically-Fastened Composite System under Corrosive Environment”, hereby solemnly declare that this thesis is an original work done and prepared by me under the guidance of Dr. Tamer El-Maaddawy and Prof. Amr El-Dieb, in the College of Engineering at UAEU. This work has not been previously formed as the basis for the award of any degree, diploma or similar title at this or any other university. The materials borrowed from other sources and included in my thesis have been properly acknowledged.

Student’s Signature ……………………….. Date ………………………..
Copyright © 2013 by Amna Zaid Suwaid Al Nassibi
All Rights Reserved
Approved by

Advisory Committee

Dr. Tamer El-Maaddawy (Principal Supervisor)
Associate Professor
Department of Civil and Environmental Engineering
College of Engineering
United Arab Emirates University
Signature ......................... Date ..............................

Dr. Amr El-Dieb (Co-advisor)
Professor
Department of Civil and Environmental Engineering
College of Engineering
United Arab Emirates University
Signature ......................... Date ..............................
Thesis Examination Committee

Dr. Tamer El-Maaddawy (Principal Supervisor)
Associate Professor
Department of Civil and Environmental Engineering
College of Engineering
United Arab Emirates University
Signature …………………….. Date ………………………

Dr. Amr El-Dieb (Co-advisor)
Professor
Department of Civil and Environmental Engineering
College of Engineering
United Arab Emirates University
Signature …………………….. Date ………………………

Dr. Bilal El-Ariss (Internal Examiner)
Associate Professor
Department of Civil and Environmental Engineering
College of Engineering
United Arab Emirates University
Signature …………………….. Date ………………………

Dr. Kenneth W. Neale (External Examiner)
Professor Emeritus
Department of Civil Engineering
Université de Sherbrooke, Canada
Signature …………………….. Date ………………………
Accepted by

Master’s Program Director: Dr. Tamer El-Maaddawy
Signature ……………………… Date ……………………..

Dean of the College: Prof. Reyadh A. Almehaideb
Signature ……………………… Date ……………………..
ABSTRACT

This study examines the effect of corrosion exposure on the flexural response of reinforced concrete (RC) beams strengthened with a powder-actuated fastened (PAF) composite system. Twenty-one RC beams were constructed and tested to failure under four-point bending. The corroded beams were subjected to 30, 60, and 100 days of accelerated corrosion that corresponded to measured tensile steel mass losses of 6, 11, and 18%, respectively. Other test parameters included width of the fiber reinforced polymer (FRP) composite strip, fastener length, and number of fasteners rows.

For the uncorroded beams, the PAF-FRP system resulted in up to 24% increase in beam flexural capacity and 20% average reduction in the beam ductility index. The strengthening effectiveness reduced with increased level of corrosion. Nevertheless, the flexural capacity of the strengthened beams at all levels of corrosion damage was either higher or almost same as that of the control beam. Increasing the fastener length increased the gain in flexural capacity of the uncorroded beams but had no noticeable effect on the flexural strength gain of the corroded beams. Doubling the width of the FRP strip or number of fastener rows had insignificant effect on the flexural strength gain.

An analytical model that can predict the flexural capacity of corroded RC beams strengthened with PAF-FRP system has been introduced. The model accounts for the non-linear behavior of materials and strain incompatibility
between the PAF-FRP strip and concrete. The validity of the analytical model has been demonstrated by comparing its predictions with the experimental results.

**Keywords:** concrete, corrosion, composite, flexural, powder-actuated fasteners, strengthening, FRP
LIST OF PUBLICATIONS

Journal Publications


Conference Publications


ACKNOWLEDGEMENTS

I would like to thank God for giving me the faith and strength to successfully complete this work. I would also like to express my sincere thanks to my family who have provided me with all the support and strength to complete this work. I would like to express my deepest thanks to all individuals who helped me during this significant period of my life. At the first place, I would like to deliver my deepest respect and appreciation to my thesis supervisor Dr. Tamer El Maaddawy for his continuous support, inestimable guidance, and the valuable knowledge he provided me throughout the project. I would like to thank him for the friendly environment he has created for me and the brotherly advice I received from him. My gratitude is also extended to Prof. Amr El Dieb for his continuous support and encouragement.

I would like also to express my keen appreciation to the structural engineering laboratory specialist Eng. Tarek Salah, the concrete laboratory technician assistant Mr. Faisal Abdul-Wahab, and Eng. Hanan Al Saedi for their help throughout testing. Special recognition goes also to my dearest sisters and friends and all faculty members of the Civil and Environmental Engineering Department at the UAEU for their help and support.

I wish to express my gratitude to the Emirates Foundation for financing this research work under grant no. 2009/052. The support provided by the UAEU under grant no. 1572-07-01-10 is also acknowledged.
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>TITLE PAGE</td>
<td>i</td>
</tr>
<tr>
<td>DECLARATION OF ORIGINAL WORK</td>
<td>ii</td>
</tr>
<tr>
<td>COPYRIGHT PAGE</td>
<td>iii</td>
</tr>
<tr>
<td>SIGNATURE PAGE</td>
<td>iv</td>
</tr>
<tr>
<td>ABSTRACT</td>
<td>vii</td>
</tr>
<tr>
<td>LIST OF PUBLICATIONS</td>
<td>ix</td>
</tr>
<tr>
<td>ACKNOWLEDGEMENTS</td>
<td>x</td>
</tr>
<tr>
<td>TABLE OF CONTENTS</td>
<td>xi</td>
</tr>
<tr>
<td>LIST OF TABLES</td>
<td>xv</td>
</tr>
<tr>
<td>LIST OF FIGURES</td>
<td>xvi</td>
</tr>
<tr>
<td>LIST OF NOTATIONS</td>
<td>xviii</td>
</tr>
<tr>
<td>Chapter 1: INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>1.1 Introduction</td>
<td>1</td>
</tr>
<tr>
<td>1.2 Retrofitting by EB-FRP composite systems</td>
<td>2</td>
</tr>
<tr>
<td>1.2.1 Applications</td>
<td>2</td>
</tr>
<tr>
<td>1.2.2 Failure modes</td>
<td>4</td>
</tr>
<tr>
<td>1.3 Anchorage Systems for FRP</td>
<td>5</td>
</tr>
<tr>
<td>1.4 Retrofitting by using MF-FRP systems</td>
<td>7</td>
</tr>
</tbody>
</table>
1.4.1 Failure modes ................................................................. 8

1.5 Organization of the work ....................................................... 10

Chapter 2: LITERATURE REVIEW .................................................. 12

2.1 Introduction .............................................................................. 12

2.2 The PAF-FRP System ............................................................... 12

2.3 Previous Studies on the MF-FRP Flexural Strengthening System .......... 13

2.3 Research Objectives ................................................................. 36

2.3.1 Research Significance .......................................................... 37

Chapter 3: EXPERIMENTAL PROGRAM ........................................... 38

3.1 Introduction .............................................................................. 38

3.2 Test Program ............................................................................ 38

3.3 Specimens Details .................................................................... 41

3.4 Specimens Fabrication ............................................................... 41

3.5 Material properties ................................................................. 45

3.5.1 Concrete .............................................................................. 45

3.5.2 Steel Reinforcement .............................................................. 45

3.5.3 Fiber Reinforced Polymers (FRP) ........................................... 46

3.6 Strengthening Methodology ....................................................... 47

3.7 Accelerated Corrosion Process ................................................. 48

3.7.1 Cleaning of corroded steel coupons ....................................... 49
<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.8 Test Set-up and Instrumentation</td>
<td>51</td>
</tr>
<tr>
<td>Chapter 4: EXPERIMENTAL RESULTS</td>
<td>53</td>
</tr>
<tr>
<td>4.1 Introduction</td>
<td>53</td>
</tr>
<tr>
<td>4.2 Test Results</td>
<td>53</td>
</tr>
<tr>
<td>4.2.1 Corrosion Damage</td>
<td>53</td>
</tr>
<tr>
<td>4.2.2 Failure Mode</td>
<td>55</td>
</tr>
<tr>
<td>4.3 Structural Response</td>
<td>58</td>
</tr>
<tr>
<td>4.3.1 Load capacity</td>
<td>64</td>
</tr>
<tr>
<td>4.3.2 Ductility index</td>
<td>66</td>
</tr>
<tr>
<td>4.3.3 Longitudinal FRP Strain Profile</td>
<td>67</td>
</tr>
<tr>
<td>4.3.4 Effect of Corrosion on Longitudinal FRP Strain Profile and Fastener Loads</td>
<td>71</td>
</tr>
<tr>
<td>4.3.5 Strain Profile along Section Depth</td>
<td>75</td>
</tr>
<tr>
<td>Chapter 5: ANALYTICAL INVESTIGATION</td>
<td>77</td>
</tr>
<tr>
<td>5.1 Introduction</td>
<td>77</td>
</tr>
<tr>
<td>5.2 Material Constitutive Laws</td>
<td>77</td>
</tr>
<tr>
<td>5.2.1 Concrete</td>
<td>77</td>
</tr>
<tr>
<td>5.2.2 Steel Reinforcement</td>
<td>79</td>
</tr>
<tr>
<td>5.2.3 Fibre Reinforced Polymer</td>
<td>80</td>
</tr>
<tr>
<td>5.3 Model Development</td>
<td>81</td>
</tr>
<tr>
<td>5.3.1 Compatibility Requirements</td>
<td>81</td>
</tr>
</tbody>
</table>
5.3.2 Equilibrium Requirements .................................................................................. 85
5.3.3 Model Procedure ............................................................................................... 86
5.4 Comparative Analysis ......................................................................................... 87

Chapter 6: CONCLUSIONS & RECOMMENDATIONS .............................................. 89
6.1 Research Summary and Conclusions ................................................................... 89
6.2 Recommendations for Future Work ................................................................. 90

REFERENCES ............................................................................................................. 92
LIST OF TABLES

Table 3.1: Test matrix ..................................................................................................................40
Table 3.2: Mechanical properties of FRP used in strengthening (SAFSTRIP® 2008).....46
Table 4.1: Test results ..................................................................................................................58
Table 5.1: Comparison between experimental and analytical results.................................88
**LIST OF FIGURES**

Figure 1.1: The manual application EB-FRP systems (FHWA) ........................................4
Figure 1.2: Typical failure modes associated with EB-FRP (J.G. Teng and J.F. Chen 2007) ..................................................................................................................5
Figure 1.3: Different anchorage devices and systems; a) FRP Anchor Spikes (Sergio F. and Geoffrey N. 2013); b) Plate Anchorage; c) U-Anchor d) bolted angle system (Grelle and Sneed 2013) ......................................................................................................................7
Figure 1.4: Application of MF-FRP system (SAFSTRIP® 2008) .............................................8
Figure 1.5: Typical failure modes associated with MF-FRP (Nardone F. et al 2011) ........10
Figure 2.1: Anchorage System Details (El Maaddawy and Soudki 2008) ..................................26
Figure 2.2: Anchorage System Details (Wu et al. 2011) ...........................................................33
Figure 3.1: Test Specimen Details ...........................................................................................42
Figure 3.2: Fabrication of steel cages and casting form .............................................................43
Figure 3.3: Bonding the Electric strain gauges (S.G) to steel bars ...........................................43
Figure 3.4: Steel cages installed inside the forms ........................................................................44
Figure 3.5: Test Specimens after removing the forms .................................................................45
Figure 3.6: Equipment and materials used in strengthening: (a) pultruded pre-cured composite strips; (b) fasteners, washers, gun, and power cartridges ........................................47
Figure 3.7: FRP strips with bonded washers at location of fasteners .........................................48
Figure 3.8: Test specimens under accelerated corrosion exposure ..........................................49
Figure 3.9: Stages of chemical cleaning of corroded steel coupons .........................................50
Figure 3.10: Schematic view of the test setup ...........................................................................51
Figure 3.11: A test in progress ..................................................................................................52
Figure 4.1: Measuring the crack width for the corroded specimens ........................................54
Figure 4.2: Typical corrosion crack pattern for the corroded control beams ..........................54
Figure 4.3: Crushing of concrete in compression zone .............................................................56
Figure 4.4: Pull-out of fasteners and peeling of concrete cover ................................................56
Figure 4.5: Sustained bearing failure at end anchors .................................................................56
Figure 4.6: Rotation and pull-out of fasteners ..........................................................................57
Figure 4.7: Rupture of the tensile steel ....................................................................................57
Figure 4.8: Load-deflection curves of group [A] .......................................................................59
Figure 4.9: Load-deflection curves of group [B].................................61
Figure 4.10: Load-deflection curves of group [C].................................63
Figure 4.11: Load-deflection curves of specimens of group [D]...............64
Figure 4.12: Longitudinal FRP strain profile at peak load for group [A].......68
Figure 4.13: Longitudinal FRP strain profile at peak load for group [B].......69
Figure 4.14: Longitudinal FRP strain profile at peak load for group [C].......69
Figure 4.15: Longitudinal FRP strain profile at peak load for group [D]......70
Figure 4.16: Initial cracks occurred during the installation of fasteners........70
Figure 4.17: Effect of corrosion on longitudinal FRP strain profile at peak load (specimens with 50 mm wide FRP strip)..............................71
Figure 4.18: Locations of Regions A and B........................................73
Figure 4.19: Effect of corrosion on the load per fastener (specimens with 50 mm wide FRP strip).................................................................74
Figure 4.20: Strain distribution along the mid-span section depth at peak load for the uncorroded beams.........................................................76
Figure 5.1: Assumed stress-strain relationship of concrete.......................78
Figure 5.2: Idealized stress-strain relationship of steel reinforcement.........79
Figure 5.3: Idealized stress-strain relationship of CFRP laminates.............80
Figure 5.4: Strain and stress distributions along section depth..................82
Figure 5.5: FRP strain reduction factor versus tensile steel mass loss.........84
LIST OF NOTATIONS

$A_i$ = area of concrete layer $i$ ($mm^2$)

$A_{si}$ = cross sectional area of steel bar $i$ ($mm^2$)

$A_f$ = cross sectional area of the FRP strip ($mm^2$)

$b$ = beam width ($mm$)

c = depth of the neutral axis measured from the extreme compression fiber ($mm$)

d = The depth of the tensile steel measured from the top face of the beam ($mm$)

d' = The depth of the compression steel measured from the top face of the beam ($mm$)

$d_f$ = distance between plastic centroid of concrete section and center of the FRP strip ($mm$)

d_i = distance measured from plastic centroid of concrete section ($mm$)

d_{ci}$ = distance between plastic centroid of concrete section and centroid of concrete layer ($mm$)

d_{si} = distance between plastic centroid of the concrete section and center of steel bar $i$ ($mm$)

$E_c$ = The Young's modulus of the concrete ($MPa$)

$E_f$ = Young’s modulus of the FRP ($MPa$)

$E_s$ = steel Young’s modulus ($MPa$)

$E_{sp}$ = steel strain hardening modulus ($MPa$)

$f_f$ = stress in FRP ($MPa$)
\( f_{fr} \) = tensile strength of FRP (MPa)

\( f_c \) = concrete compressive stress (MPa)

\( f_c' \) = concrete compressive strength (MPa)

\( f_{ci} \) = concrete stress at the center of the layer \( i \) (MPa)

\( f_s \) = steel stress (MPa)

\( f_{si} \) = stress in the steel bar \( i \) (MPa)

\( f_{su} \) = The steel ultimate strength (MPa)

\( f_y \) = steel yield stress (MPa)

\( h \) = beam depth (mm)

\( l_f \) = length of fastener (mm)

\( m_l \) = percentage steel mass loss in tensile steel

\( M_n \) = nominal moment strength (N-mm)

\( N \) = number of fasteners in a region between sections \( i \) and \( j \)

\( P_{f,avg} \) = average transferred load per fastener in a region between sections \( i \) and \( j \) (N)

\( w_f \) = width of the FRP strip (mm)

\( z \) = distance measured from neutral axis of the beam (mm)

\( \Delta_f \) = mid-span deflection at ultimate load (mm)

\( \Delta_{max} \) = beam deflection capacity (mm)

\( \Delta_y \) = mid-span deflection at the yielding load (mm)

\( \varepsilon_{cu} \) = The concrete strain corresponding to the concrete compressive strength

\( \varepsilon_{c} \) = The concrete strain for a given loading condition

\( \varepsilon_{cu} \) = concrete crushing strain taken as 0.003
\[ \varepsilon_f = \text{The CFRP strain for a given load condition} \]

\[ \varepsilon_{f,\text{exp}} = \text{measured FRP strain at the mid-span section at the onset of concrete crushing} \]

\[ \varepsilon_{f,\text{bonded}} = \text{calculated FRP strain in case of a fully-bonded FRP strip} \]

\[ \varepsilon_i = \text{FRP strain measured at a section } i \text{ nearest to the support} \]

\[ \varepsilon_j = \text{FRP strain measured at a section } j \text{ nearest to the mid-span} \]

\[ \varepsilon_{sy} = \text{The steel strain corresponding to the yield stress } f_y \]

\[ \varepsilon_{su} = \text{The steel strain corresponding to the steel ultimate strength } f_{su} \]

\[ \varepsilon_z = \text{strain at a distance } z \text{ measured from the neutral axis of the concrete section} \]

\[ \kappa = \text{deterministic FRP strain reduction factor} \]

\[ \mu_A = \text{beam ductility index} \]
CHAPTER 1: INTRODUCTION

1.1 Introduction

The use of composites in strengthening and retrofitting of reinforced concrete (RC) structural components deficient in flexural strength has gained a wide acceptance by the structural engineering community. The most popular composite system is the externally-bonded fiber reinforced polymer (EB-FRP) in which a FRP strip or sheet is bonded to the tension face of the flexural element using a structural adhesive. Although the system is effective in increasing the flexural capacity, it is often vulnerable to fail prematurely in a brittle manner due to delamination of the bonded FRP thus reducing the beam ductility (Bank 2006; ACI 440.2R 2008; Balaguru 2009). Accordingly, several anchorage devices have been developed to be used in combination with the EB-FRP system to avoid premature detachment/delamination of the bonded FRP and improve the performance of retrofitted RC beams (El Maaddawy and Soudki 2008; Galal and Mofidi 2009; Wu et al. 2011; Kalfat et al. 2013).

Moreover, popular and current methods of RC beam strengthening (through adhesive bonding of thin composite laminates to the beam surfaces) are time-consuming, since they take days per application to sandblast, clean and prepare the concrete surface to make it suitable for bonding the strip (Lamanna et al. 2001; Bank et al. 2002). As an alternative, a powder-actuated fastening (PAF) system may be used to attach composite strips to concrete and meet requirements for rapid strengthening especially in time critical situations as it eliminates the
need for concrete preparation and clamping the strip while the epoxy cures (Gambhir 2009). The powder-actuated fasteners (PAF) are used for the rapid strengthening of concrete beams with fiber reinforced polymer (FRP) composite materials (Lamanna et al. 2001).

1.2 Retrofitting by EB-FRP composite systems

Several types of FRP strengthening systems are available: wet lay-up systems; systems based on prefabricated elements; and, special systems such as: automated wrapping, prestressing, etc., though each of these however corresponds to several manufacturers’ specifications, different configurations, types of fibers, adhesives and the like (FIB 2001). The use of FRP for externally bonded reinforcement (EB-FRP) to rehabilitate and strengthen existing structures and materials of reinforced concrete (RC) elements such as beams and slabs is becoming a widely accepted practice (FIB 2001).

1.2.1 Applications

The common method of strengthening deficient RC members is by adhesively bonding of thin prefabricated sheets or strips of composite laminates known otherwise as FRP sheets/strips as shown in Figure 1.1, to the surfaces of RC beams or slabs to increase their capacity (Lamanna et al. 2001; Gambhir 2009).
The commonly available composite materials for strengthening RC structures are mainly in the form of:

a) thin unidirectional strips (with thickness in the order of 1 mm) made by pultrusion; and

b) flexible sheets or fabrics that are made of fibers in one or at least two different directions that are sometimes pre-impregnated with resin (FIB 2001).

Flexural strengthening of RC slabs, beams, walls and columns is achieved by bonding FRP strips to the tension face or a portion of a flexural member (Bank 2006). However, the performance of these strips depends on several variables such as: the bonding strength of the adhesives used; the state of stress at the interface of the concrete and the FRP strips; the failure modes of the concrete; methods of curing and the material preparations needed (Gambhir 2009). These and among a host of other considerations form the bases for design and safety concepts (FIB 2001).

The basic FRP strengthening technique, which is most widely applied, involves the manual application of wet lay-up systems as shown in Figure 1.1(a). The application involves the following steps:

1) preparation of concrete substrate with "Putty-Filler" resins;

2) application of "Primer" resins on concrete substrate;

3) application of "Saturating" resins;

4) application of fabric sheet;
5) application of another layer of "Saturating" resins;

6) application of "Protective Coatings" resins.

For bonding rigid FRP strips as shown in Figure 1.1 (b) the installation of precured FRP systems is generally similar to that of single-ply wet lay-up. Surface preparation of the concrete substrate shall provide an open roughened texture.

a) wet lay-up system

b) precured FRP system

Figure 1.1: The manual application EB-FRP systems (FHWA)

1.2.2 Failure modes

The debonding of an externally bonded FRP sheet/strip can be predicted by considering the different bond failure modes which can occur under any of the following occurrences: bond-critical failure modes (end debonding, intermediate crack debonding); cohesive and adhesive strengths of the concrete; ultimate strength for end debonding (concrete rip-off); ultimate strength for intermediate rip-off; and, interfacial stresses for the serviceability limit state (FIB 2006) as shown in Figure 1.2. Failure in case of RC beams may take place through:
1. concrete crushing before yielding of the reinforcing steel;
2. steel yielding followed by FRP rapture;
3. steel yielding followed by concrete crushing;
4. cover delamination; and;
5. FRP debonding

Figure 1.2: Typical failure modes associated with EB-FRP (J.G. Teng and J.F. Chen 2007)

1.3 Anchorage Systems for FRP

A mechanical anchorage is being utilized whenever possible, to offset bonding problems associated with inadequate development of chemical or adhesive bond between a RC member (e.g., slab or beam) and an FRP laminate, or problems associated with the soundness and tensile strength of the concrete substrate.

However, anchorage systems are typically fabricated from metals, which are susceptible to corrosion. Hence, the use of steel anchorage system may not be a durable option (Grelle and Sneed 2013). It is worth noting that many of the FRP strengthening systems available could be designed and manufactured without the
need for anchorage systems in case sufficient space exists for the FRP design strength to be developed along an “active bond length” over which, the majority of the bond stress is maintained (Grelle and Sneed 2013).

The performance of anchorages is critical in the design of FRP strengthening systems because the improved strength due to the anchorage still may not be high enough to offer the full tensile strength of FRP, and, failure modes are most often anchorage failure (Grelle and Sneed 2013). Under fatigue loading for instance, debonding problems may become significant or persistent until adequate bond anchorage is provided (Buyukozturk et al. 2004). According to Grelle and Sneed (2013), the purpose of anchorage systems is to allow the FRP strengthening system to reach higher design strength by accomplishing one or more of the following:

1. preventing or delaying a premature debonding failure by resisting tensile normal forces;
2. reducing the in-plane development length required to achieve a specified design by transferring load from the FRP to anchorage via shear; or,
3. transferring the force in the FRP laminate to another structural component where no development length is available (Grelle and Sneed 2013).

Research on systems to mechanically anchor EB- FRP strengthening systems has included anchor spikes, transverse wrapping, U-Anchors,
longitudinal chases, FRP strips, plate anchors, bolted angles, cylindrical hollow sections, ductile anchorage systems, and other miscellaneous systems as shown in Figure 1.3. Each of these anchorage systems has unique geometrical constraints, installation limitations, and force (stress) transfer characteristics.

Figure 1.3: Different anchorage devices and systems; a) FRP Anchor Spikes (Sergio F. and Geoffrey N. 2013); b) Plate Anchorage; c) U-Anchor d) bolted angle system (Grelle and Sneed 2013).

1.4 Retrofitting by using MF-FRP systems

The primary ways to make connections in FRP materials are to adhesively bond the pieces or to mechanically fasten them. While adhesively bonded joints minimize stress concentrations, the adhesives used can be severely weakened by environmental effects. Adhesively bonded joints also require significant surface
preparation and their integrity is difficult to verify by inspection. Mechanically fastened joints do not require surface preparation and are relatively easy to inspect. Mechanically-fastened FRP system has emerged as a promising alternative solution for rapid strengthening of RC structures (Lamanna et. al. 2001, Lamanna et al. 2004). The technique is suitable for emergency repairs where constructability and speed of installation are critical requirements. In fact, unlike EB-FRP, the MF-FRP system requires conventional hand-tools and workmanship, which enables the immediate use of the strengthened structure. In this system a composite strip is attached and anchored to the concrete surface using powder-actuated fasteners (PAF), threaded screws, or expansion bolts without bonding as shown in Figure 1.4.

![Figure 1.4: Application of MF-FRP system (SAFSTRIP® 2008)](image)

**1.4.1 Failure modes**

The behavior of MF-FRP connections is related to any of the components constituting the connection, which are the concrete substrate, the fastener and the FRP material. As a result, failure modes can involve the concrete (crushing), the (rotation/pullout or rupture), or the FRP (delamination or rupture).
There are typically five failure modes for mechanically fastened connections. The first failure mode is a net-tension failure, which is characterized by a fracture across the net section, perpendicular to the direction of loading. The second failure mode is cleavage failure, which is characterized by a crack parallel to the applied load that starts at the edge of the composite and propagates toward the bolt hole, leading to the initiation of other cracks across the net section due to the formation of in plane stresses. Cleavage failure is also called block shear. The third failure mode is bearing failure which is characterized by crushing of the material around the bolt contact area. Following bearing failure, a fourth failure mode called shear out failure is often observed. The pryout or spalling of the concrete depends on the local composition of the concrete surface around the fastener. Once pryout failure develops, the fastener rotates and the FRP laminate pulls it out of the concrete. Several factors promote the initiation of the concrete failure, such as a fastener hitting a hard aggregate during installation, low concrete strength, cracked concrete substrate conditions, short edge distance that may cause spalling, and poor fastener embedment depth (Lamanna 2001). The five failure modes of mechanically fastened connections in FRP are shown in Figure 1.5.
Figure 1.5: Typical failure modes associated with MF-FRP (Nardone F. et. al 2011)

Thorough investigations are needed to examine the durability of the mechanically-fastened composite system under corrosive environment before it can be employed routinely as a solution to strengthen reinforced concrete (RC) structural components.

1.5 Organization of the work

The present research work investigates, experimentally and analytically, the effect of corrosion exposure on flexural behavior of concrete beams strengthened with the mechanically-fastened fiber reinforced polymer (MF-FRP) system.

A literature review on flexural strengthening of RC beams with MF-FRP strengthening systems is presented in Chapter (2). The research objectives and significance concludes the chapter.

Chapter (3) provides detailed information on the experimental program, specimen dimensions, geometry, and fabrication. It also includes information on
materials properties and strengthening methodology. A full description of the test set-up, instrumentation, control, and loading procedure is presented in the same chapter.

Chapter (4) presents results of the experimental testing and observations. The results include failure modes, load capacity, and ductility index. Longitudinal FRP strain profile and load-deflection curves are presented. Discussions and comments relevant to the results are included.

In Chapter (5), an analytical model for predicting the flexural capacity of corroded RC beams strengthened with the PAF-FRP system is introduced. A comparison between experimental and analytical results is presented and discussed.

Chapter (6) summarizes the general conclusions of the work along with recommendations for future studies and developments on performance of RC beams strengthened with the MF-FRP composite system.
CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

This chapter includes a review of the available literature on reinforced concrete beams strengthened in flexure with the MF-FRP composite system. Different parameters which may influence the effectiveness and viability of the system is presented and discussed.

2.2 The PAF-FRP System

The use of mechanical fasteners instead of adhesive bonding as a method to attach FRP strips to concrete beams has been studied in recent years (Lamanna et al. 2001, Lamanna et al. 2004). Mechanical fastening has advantages that include very little surface preparation, rapid installation, lower costs and immediate use of the strengthened structure but, the method requires special FRP strips that cannot be used with conventional unidirectional carbon-epoxy FRP strips because they should have high bearing strength and longitudinal stiffness not ordinarily found in composites intended for adhesive bonding (Bank 2006).

In the PAF-FRP system, PAF are driven into the concrete substrate with the energy released by a black powder charge (Lamanna et al. 2001, Eligehausen et al. 2006). When the fastener is installed in the concrete, the surface of the fastener becomes deformed and generates friction with the surrounding material (Lamanna et al. 2001). The fastener achieves bullet-like velocities at the point of impact with the base material (Eligehausen et al. 2006). The heat generated in this
process causes sintering and creates a bond between the concrete and the fastener (Lamanna et al. 2001).

### 2.3 Previous Studies on the MF-FRP Flexural Strengthening System

Fastened-unbonded composite strengthening system was first developed by Bank and his research group at the University of Wisconsin-Madison, USA in 2001 (Lamanna et al. 2001; Bank et al. 2002). Further studies have been conducted in the same area after. In-depth review of available previous studies on flexural strengthening with MF-FRP system is given below.

**Lamanna et al. (2001)** investigated the feasibility of attaching pultruded FRP strips to concrete without bonding in the strengthening of small-sized reinforced concrete beams using a powder-actuated fastening (PAF) system. A total of 14 beams, divided into two concrete design strengths were tested; nine beams with strength (C-21) and five beams with strength (C-42). The beam’s cross section was 153 x 153 mm. The test specimens had a length of 1220 mm. Test variables included: concrete type, fastener type and length, the number of rows and strip width. The composites had tensile modulus that was in the range of 13.8 to 27.3 GPa; tensile strength between 204.5 and 560.7 MPa; and, thickness between 3.2 and 6.4 mm. All beams were tested in displacement control at a rate of 0.5 to 0.8 mm/min.

For all of the nine 21 MPa beams that utilized mechanical fasteners, failure occurred in the compression zone. Large flexural and shear cracks were observed in the beams at the point of crushing. On the other hand, one of the five
42 MPa beams sustained large initial cracks caused during the fastening of the strip, which resulted in spalling of large chunks of concrete. Additional trials using different fastener types (smaller in diameter) have yielded improved results. Although all of the remaining four specimens also sustained cracks, the degree of damage was less severe than the cracking caused by large diameter fasteners.

Comparing the overall results between the two beam types used, Lamanna et al. (2001) have observed that the cracking was consequential to the penetration of the fastener as impacted by factors like the fastener type and diameter along with the size of beams, edge and spacing distances. The study concluded that strengthening of RC beams with composite strips using a PAF system may be a feasible choice, especially when ‘speed’ is of a vital consideration. However, cracking is expected which may prevent the strengthened beam from reaching the increased ultimate moment that adhesively bonded strips are able to achieve. Therefore, a lower level of strengthening will be reached with smaller RC beams of minimum edge distances. However, the cracking could be minimized and a corresponding optimal strengthening can be achieved with larger beams having larger edge distances. It was recommended to conduct additional investigation and further studies before any design recommendations could be developed.

Bank (2004) presented an overview of the experimental and theoretical research on the fastened-unbonded composite method. Based on the experimental investigations conducted, it was concluded that this method is a viable technique to strengthen RC beams particularly where speed of installation and immediacy of use is imperative. It was also concluded that research is needed to determine the
long-term durability of the fastened-unbonded composite method. The environmental effects need to be studied. Issues of galvanic corrosion, degradation of composite strips at the holes, and effect of shear transfer length need further in depth investigations.

Lamanna et al. (2004) examined the application of the MF-FRP strengthening and determined the effectiveness of the PAF system on improving the load carrying capacity of large concrete T-beams. A total of nine RC T-beams were tested. The beams were 8.84 m long, reinforced with No. 9 reinforcing bars and tested on a span of 8.53 m. Test variables included: reinforcing steel layouts (three different layouts); the number of FRP strips attached; the strip length; fastener length; number of fasteners used and termination distance.

The concrete strength at the time of testing ranged from 32 to 40 MPa. The nominal yield strength of the steel used was 414 MPa, whereas the ultimate strength of strengthening strip (based on stress at rapture) was determined to be at 743.3 MPa. Heat-treated PAF with high strength zinc plated steel were used. Four point bending tests were performed, while the deflections of the web and of the flange at mid span were measured by linear variable differential transducers (LVDT).

One significant finding revealed that a concrete beam strengthened by FRP strip system could display several distinct failure characteristics or modes that include: concrete crushing, strip detachment and rupture. Of these, strip
detachment was acknowledged as the primary failure mode of all of the beams strengthened with mechanically attached composite strips.

The study concluded the following:

- A significant increase in moment capacity can be obtained with the use of relatively small amount of FRP strips.
- Strip-delamination due to end fastener failure began at the end of the strip and propagated inward toward the midspan;
- Strip delamination due to gross fastener failure began at the midspan and propagated toward the end of the strip, while the fasteners detach of the concrete one after another; and
- It was possible to strengthen very large reinforced concrete T beams with mechanically fastened composites.

A study by Lamanna et al. (2004b) was conducted with the primary purpose of determining if the MF-FRP method could be used to strengthen a larger-scale beam while maintaining at the same time a gradual failure mode, and to examine also the various parameters that affect the behavior of the strengthened beam. A total of 15 RC beams were tested – two were un-strengthened, 12 were strengthened with MF-FRP system, and one was strengthened with a bonded FRP strip. The specimens were 3.658 mm long with a cross section of 305 x 305 mm. The concrete strength at 28 days was 32.7 MPa. Primary steel reinforcement consisted of two No. 25 Grade 420 deformed bars. Two No.10 Grade 420 deformed top bars were used as compression steel. Shear reinforcement was
closed stirrups of No. 13 Grade 420 deformed bars placed 102 mm on center throughout the shear span of the beam and into one-third of the moment span. The FRP strips had three different moduli (15.2, 26.3 and 57 GPa) so that the effect of the strip modulus could be studied as each were labeled as standard, intermediate and hybrid. All strips had the same cross-sectional dimensions of 102 x 3.2 mm and were produced by the pultrusion process. The effect of omitting of fasteners in the constant moment span was studied.

It was concluded that FRP strengthening strips attached to reinforced concrete beams with PAF were as effective as the traditional method of bonding the strips to beams. Hence with the fastened method, it was possible to achieve a failure mode similar to that seen in a standard reinforced concrete beam. Finally, the time differential of 30 minutes to complete the mechanical fastening method versus 4 hours for the bonded method promoted encouragement for further research towards enhancing the MF-FRP method even more.

Rizzo et al. (2005a) presented a critical analysis of the parameters affecting the performance of the connection of concrete-fastener-FRP laminate to demonstrate how a designer can rationally choose the type of connection – depending on the restraints posted by the condition of the concrete substrate and the final purpose of the strengthening design. It was found that if the end distance in the longitudinal direction is properly chosen, the bearing stress at failure of the MF-FRP laminate can be assumed to be constant regardless of the size of the fastener and the distance between the hole center and the free edge in the direction normal to the applied load.
With the importance given to the bearing strength, the other parameters that characterized the connection should be given an attention, such as the: diameter, length and strength of the fasteners; clamping force, the presence and type of washer; presence and type of filler in the gaps between the FRP material, the fastener and fastener accessories and the concrete. Also, the fastening procedures (i.e. powder-actuated system; wedge bolts system and wedge anchor system) were also accentuated, since particular attention must be rendered to minimize the damage to the concrete as caused by the vibrations of the drill when installing FRPs. To examine the failure modes and the behavior of the connection between the FRP laminate and concrete member, Rizzo et al. (2005) forwarded some laboratory test evaluations pertaining to single-bolted shear tests and cited a few practical field applications in relation to the PAF system which was found very efficient for low compressive strength concrete, but for concrete with compressive strengths higher than 17.2 MPa, this fastening method resulted in concrete spalling and cratering which were not considered acceptable.

The PAF-FRP system was found to be more ideal for time critical projects being more rapid. Also, it allows the complete exploitation of the FRP strengthening right after its application because the load transfer mechanism relies just on the mechanical anchoring of the pins. The measures suggested to improve the system performance included: prevent major concrete damages starting at the drilling stages; use gap fillers to improve the performance of the connection in terms of strength and stiffness; and, use fasteners with a diameter of 9.525 mm because they appeared to be suitable for a wide range of concrete strength and
they allowed the reaching of the pseudo-ductile bearing failure in the MF-FRP laminate side.

**Lopez et al. (2005)** reported data on retrofitting an RC bridge using composites. The report pertained to the technology of composite strengthening of bridge structures using two installation techniques: (1) manual lay-up carbon FRP laminates, and (2) mechanically-fastened hybrid glass/carbon FRP laminates. The objective was to increase the flexural capacity of the structure with the MF-FRP system, after the advantages (fast and easy) had outweighed the disadvantage of ‘the damage caused to the concrete substrate’, since the fasteners were shot into the element.

The total bridge length was 6.4 m. the bridge deck was 178 mm thick and 7.3 m wide. The concrete compressive strength was 27.6 MPa. The steel yield strength was 276 MPa. The steel reinforcement composed of two grids – 19 mm and 25 mm diameter rebar in the top and bottom of the deck, respectively. The MF-FRP laminates system consisted of pre-cured FRP strips with high traverse bearing strength attached to the concrete surface using wedge anchors. The laminate used was glass and carbon pultruded strip with a thickness of 3.175 mm and width of 101.6 mm.

After having completed the installation of external reinforcement, some post installation analyses were made. Load rating was performed to obtain the safe load carrying capacity of the bridge after strengthening. Field validation was made to validate the behavior of the bridge prior to and after the strengthening.
An FEM analysis model was developed in order to interpret the experimental data and establish correlations. It was concluded that the FEM analysis was in good agreement with the experimental data, lending substantive support over test results showing that the strengthening systems increased the flexural capacities of the bridge.

Rizzo et al. (2005b) conducted a study on the application of MF-FRP composites to strengthen three rural bridges identified with flexural deficiencies. The bases for measuring efficiency were translated in terms of structural performances, cost, labor and time saving. Resin as gap-filler between the concrete, the strip and the fastener was introduced. Starting from the shortest, the total lengths of the bridges were: 7874 mm, 7925 mm and 9754 mm; and the widths of the decks were: 6325 mm; 6680 mm, 6756 mm, respectively.

The strengthening material used was a glass/carbon hybrid pultruded strip embedded in a vinyl ester resin with thickness of 3.175 mm and width of 101.6 mm. The fastening system consisted of 9.525 mm diameter concrete wedge anchors with a total length of 57 mm. Field validation and FEM analysis were made to validate the behavior of the bridge before and after strengthening. With the use of Linear Variable Differential Transducers (LVDTs), displacements in the longitudinal and transverse directions were measured.

Based on their analyses, Rizzo et al. (2005) have concluded that, MF-FRP was feasible and a convenient solution (i.e. rapid and economical) for the flexural
strengthening of bridges deck and girders. It was suggested that future inspections and load testing should be made in order to quantify the effects of corrosion.

**Ekenel et al. 2006** conducted an experimental investigation on seven RC beams (254 x 165 x 2000 mm). Two beams were used as a control, three beams were strengthened with externally-bonded composite fabrics, and two beams were strengthened with a pre-cured composite strip. Glass fiber anchor spikes were used with the externally-bonded composite fabrics, and mechanical fasteners were used in one of the beams strengthened with the pre-cured composite strip. Research results indicated that the use of anchor spikes in the externally-bonded composite system increased the ultimate strength. The use of anchor spikes resulted in about 40% increase in strength relative to a similar strengthened beam without anchor spikes. Mechanical fasteners can be used as an alternative to epoxy bonded pre-cured laminate system. Composite strengthening increased fatigue life by increasing stiffness and reducing crack propagation. The beam strengthened with fastened-unbonded composites showed more ductile behavior as to the beams with externally-bonded composite system.

**Tan and Saha (2007)** investigated the behavior of RC beams strengthened with MF-FRP system under cyclic loads to look into the effectiveness of the system and the effect of load range on deflection as well as the cracking characteristics of the strengthened beams. A total of eight beams were tested with a cross section dimension of 100 X 125 mm and a total length of 2000 mm. Two No. 10 bars were used for tensile reinforcement and two No. 6 bars were used as compressive reinforcement. No. 6 stirrups were placed at a spacing of 75 mm
throughout the entire length of the beam. Of the eight beams, two were unstrengthened to determine its inherent ultimate flexural strength and to compare the serviceability behavior with the strengthened beams when subjected to cyclic loading. The other six beams were strengthened with pultruded FRP strips using PAF and anchors. A single FRP strip was applied to five beams and a double FRP strips were applied to the sixth beam. The measured ultimate strength of the FRP strips was 844 MPa; the open-hole strength was 340 MPa; the bearing strength was 234 MPa, and the elastic modulus was 61.3 GPa.

Supported with a span of 1800 mm, the loads were applied at two points apart with a distance of 200 mm. The control beams were statically loaded to failure. For beams strengthened with a single FRP strip, the deflections were found to be larger for larger load ranges, especially for beams which failed during cyclic loading due to progressive pullout of fasteners. In general, the beams showed a steep reduction in stiffness in the initial 100 cycles but after which, the reduction subsided. It was observed that deterioration in the anchorage bond between the concrete and steel fasteners due to cyclic loading might be responsible for stiffness degradation in some FRP-strengthened beams. It was also found that load range had a direct impact on FRP strain. Meaning, the larger the load range, the higher was the FRP strain. On the aspect of crack width, macro cracks were noticed during the strengthening operation of the beams which had propensity to cause cracks to become wider during the cyclic loading.

Based on the test results, the conclusions drawn were: RC beams with a larger load range showed larger deflections and crack widths; the stiffness of the
strengthened beams was found to degrade quickly under cyclic loading; and further research should focus on limiting the cracking of beams during strengthening to assure better bond anchorage between mechanical fasteners and concrete under cyclic loading.

Bank and Arora (2007) presented an analytical model developed to predict the ultimate strength of RC members strengthened with the MF-FRP system. The framework was based on parameters such as strain compatibility, equilibrium and constitutive relations of the materials. Typical assumptions made in routine RC analysis were adopted in this model.

To verify the reliability of the model, four beams (one control and three strengthened) of 7.32 m long with a cross-section of 508 x 508 mm were tested. The concrete strength was 42 MPa. The beams were reinforced with three No. 22 Grade 420 main bars and No. 13 Grade 420 closed stirrups at 200 mm spacing along the full length size of the beam. Also, two No. 13 top bars were used. Of the three strengthened beams tested, two were strengthened with two FRP strips fastened using zinc plated (galvanized) hardened steel fasteners. The remaining strengthened beam was fastened using three FRP strips. In addition, either carbon or stainless steel end anchors were used at the strip ends. The pultruded FRP strips used were 102 mm wide and 3.2 mm thick.

Test results showed that the ultimate failure mode, in all of the tested beams was concrete crushing after yielding of the tensile beams. The strengthened beams demonstrated this failure mode at a significantly higher load than the
control beam. On the other hand, the detachment of the MF-FRP strengthening system in the strengthened beams occurred long after the concrete compression failure at large displacements. The researchers also presented an analytical model for prediction the flexural capacity of the beams strengthened with the MF-FRP system.

Bank and Arora (2007) hence concluded that consequential to the design of the analytical model, all beams were predicted to fail due to concrete compression with sustained bearing in the FRP strips. Moreover, the analytical predictions compared well with the experimental results. The model was able to predict the unique failure modes, in particular, the pseudo-plastic behavior of the FRP strip when it failed in a progressive fashion in sustained bearing. It was recommended to use FRP strips with a high longitudinal bearing strength and a high open-hole tensile strength in order to apply the PAF system and expansion anchors more effectively. Hence, when appropriately designed, the MF-FRP method can reach strengthening levels comparable to that of the EB-FRP system and yield a ductile response with the FRP strip remaining attached after concrete compression failure for very large displacements.

Lee et al. (2007) conducted an investigation to determine the effects of simultaneous environmental exposure and sustained load on RC beams with mechanically fastened FRP strips. To examine degradation due to environmental effects and sustained load, 10 MF-FRP strengthened beams with a cross section of 200 x 150 mm and a length of 1520 mm (1370 mm between supports) were tested under 4-point flexural loading. Two No. 12 bars and two No. 10 bars were used
for the tension and compression areas in the beams, respectively. The number of fasteners to be used was based on the recommendations of Lamanna et. al (2001). The concrete strength was 34.5 MPa.

Main conclusions of this study are summarized hereafter;

- The use of anchors near the ends of the FRP strip significantly enhanced the ductility failure of the MF-FRP beams;
- Longer fasteners increased the flexural strengthening effect;
- Failure of the strengthened beams was largely governed by concrete crushing during gradual FRP delamination;
- The beams strengthened with MF-FRP system demonstrated progressive, ductile, FRP delamination under flexural loads;
- Flexural tests did not show any significant degradation as a result of sustained load or environmental effects over a period of six months;

El Maaddawy and Soudki (2008) examined the potential use of mechanically-anchored unbonded FRP (MA-UFRP) system for upgrading RC slabs deficient in flexural strength. No surface preparation and adhesive application or skilled labor were required to apply the system. A total of six slabs of 1800 mm long, 500 mm wide and 100 mm deep were used. One slab acted as a control. Two slabs were strengthened with EB-FRP with and without end anchorage. Three slabs were strengthened with MA-UFRP with different anchorage locations. The slabs were individually reinforced at tension side by three No. 10 deformed steel bars with a concrete clear cover of 20 mm,
corresponding to a steel reinforcement ratio of about 0.8%. The compressive strength of concrete was 28 MPa. The steel reinforcement was Grade 400 with nominal yield and ultimate strengths of 440 and 600 MPa, respectively. The FRP strip was carbon fiber reinforced polymer (CFRP) with a width of 50 mm and a thickness of 1.2 mm. The CFRP manufacturer’s data sheet indicated an elasticity modulus of 155 GPa and a rapture tensile strength of 3100 MPa with an ultimate elongation of 1.9%. The CFRP bonded to the tension face of the slab, corresponded to a CFRP reinforcement ratio of about 0.12%. The test variables included the strengthening system, condition of CFRP and number of anchors. The anchor system consisted of a steel plate of 100 x 130 x 10 mm having four holes of 15 mm diameter as shown in Figure 2.1. All slabs were tested under four-point bending with an effective span of 1500 mm and a shear span of 500 mm.

Figure 2.1: Anchorage System Details (El Maaddawy and Soudki 2008)
Test observations indicated that the control slab exhibited a conventional ductile flexural mode of failure in which the slab failed by yielding of steel reinforcement followed by crushing of concrete. For specimens strengthened with EB-FRP, the presence of end-anchorage prevented debonding of the CFRP strip and hence, the slab developed its full flexural capacity. All slabs strengthened with MA-UFRP failed by crushing of concrete which was preceded by yielding of steel reinforcement and excessive CFRP end slip.

El Maaddawy and Soudki (2008) concluded that the MA-UFRP strengthening system was effective in increasing the slab strength, although comparatively, the slab strength gain was less than that obtained with the use of EB-FRP strengthening system. Conversely, the mid-span deflection at ultimate load of the slabs strengthened with MA-UFRP system was on the average 56% higher than the slab strengthened with EB-FRP without end-anchorage, 5% higher than EB-FRP strengthened slab with end-anchorage and only 15% lower than that of the control beams. Finally, the stiffness and the yield load of slabs strengthened with MA-UFRP system were increased as the number of anchors along half of the slab span was also increased.

Martin and Lamanna (2008) investigated the effect of fastener number, spacing and fastener pattern on the behavior of six RC beams through the use of two specimen groups. RC beams used were 3657 mm long and had a cross section of 304.8 x 304.8 mm. The concrete strength was 48 MPa. All the reinforcing steel was a standard Grade 420. Longitudinal beam reinforcement consisted of two No. 25 bars in the tension zone and two No. 10 bars in the compression zone. Shear
reinforcement was made of No. 13 stirrups placed every 101.6 mm in order to ensure failure of the beam in flexure. The FRP plates were a proprietary mix of glass and carbon fibers in a vinylester resin measuring 101.6 mm wide and 3.175 mm thick, cut to length of 3,251 mm. The strength ranged from 638 to 871 MPa while the modulus ranged from 52,240 to 53,531 MPa. The concrete screws used to attach the FRP strip to the beam were 50.8 mm long and 12.7 mm in diameter. Each beam was tested in four-point bending over a 3350 mm span with a constant moment region of 1070 mm in a self-reacting test frame by a single 360 kN MTS actuator.

There were two series of tests conducted to determine the effect of the variations by screw spacing and the quantity used on the behavior of the strengthening system which was later identified and divided into pre-yield and post-yield phases. In general, tests indicated that a good strengthening system will exhibit an increase in ductility and improved capacity in the post-yield phase while achieving an improvement in stiffness and capacity in the pre-yield stage. The study concluded that MF-FRP systems strengthening resulted in 10 to 39% increase in the flexural capacity of RC beams when concrete screws were used in the MF-FRP system. Martin and Lamanna (2008) also found that the fastening spacing and pattern could ensure both an increase in strength and stiffness in the pre-yield phase as well as sufficiently augment post-yield ductility.

Elsayed et al. (2009) investigated the interfacial behavior between MF-FRP composites and concrete substrates through experimental tests and finite element modeling. A total of 23 concrete blocks, 150 X 150 X 400 mm each,
were constructed and tested under direct shear. Test parameters included fastener type, as the fasteners were driven or hammered into the concrete, and the bonding system. The concrete compressive strength was 42.1 MPa. The concrete tensile strength was 3.5 MPa. The strengthening regime consisted of fastening FRP composites to concrete blocks and bonding FRP strips to concrete blocks. Direct shear tests for MF-FRP strips with only one fastener and multiple fasteners were performed to investigate the behavior at the FRP-fastener-concrete connections and the bonded FRP as well. The FRP strip dimensions were 650 x 50 x 3.2 mm and the thickness was 3.2 mm. Test results indicated that the governing mode of failure in the case of the shot fasteners (PAF) was bearing failure associated with fastener pullout. In the case of the screwed fasteners, the failure was bearing in the FRP strip. Moreover, for the PAF, it was seen that instant cracks were induced during the fastening process. These cracks weakened the surrounding concrete, resulting in the eventual pullout of the fasteners off the concrete. Furthermore, the finite element results were consistent with the experimental data. The comparisons between the numerical predictions and the test results showed a very good agreement in terms of the ultimate carrying capacities, failure modes, and load-deflection relationships. It was concluded that:

- The pullout of the fasteners off the concrete mode of failure can be switched from FRP bearing to FRP rupture by increasing the number of fasteners.
• The screwed fasteners was found to be more efficient than the PAF due to the superior fastener installation, which did not damage either the concrete or the FRP strip.

• The predictive capability of the numerical tool can be exploited further for a better understanding of the role of other factors on the performance of MF-FRP strengthened concrete members, and thus contribute towards the optimization of FRP strengthening configurations.

**Lee et al. (2009)** examined the flexural behavior of RC beams strengthened with MF-FRP strips and evaluated the effectiveness of the MF-FRP strengthening method by estimating the shear capacity of the concrete-FRP joint. The ‘slip’ between the FRP and the concrete caused by factors such as bearing failure and nail rotation were among the regions of interest for this study. A total of 12 small size MF-FRP beams and two control beams were tested under flexural loading. The concrete blocks had a cross sectional dimension of 130 x 130 mm and a length of 260 mm. The beam were 200 mm deep, 150 mm wide, and 1500 mm long. The depth of concrete cover was 38 mm. The 28 day compressive strength of the concrete was at 28.8 MPa. Two No. 9 and two No. 13 bars were used as compressive and tensile reinforcements, respectively. No. 9 stirrups were used as shear reinforcement. The No. 13 steel bar had an elastic modulus of 190 GPa. The FRP strip used was 102 mm wide and 3.2 mm thick with an elastic modulus of 68.3 GPa. To attach the FRP strips, steel fasteners with a length of 32 mm were used. Anchors with a length of 54 mm and diameter of 6.5 mm were installed at the ends of the strip. The size of neoprene washers used was 3 mm in
thickness. The beams, including the control, were tested under 4-point flexural load with a shear span length of 63.4 cm and inner span of 15 cm.

Observations and strains measurements revealed a slip and strain incompatibility due to fastener-related phenomena. This was later taken in the analysis of flexural behavior of the MF-FRP beams. Based on the experimental and analytical investigations, it was concluded that: (a) the use of anchor near the ends of the FRP strip proved to be highly effective for strengthening as it significantly enhanced the ductility of the MF-FRP strengthened beams; (b) the experiments were able to demonstrate progressive slipping of the FRP strip under increasing flexural load. Factors such as rigid body rotation, localized bearing failure and fastener deformation tended to influence the FRP slip; (c) the force carrying capacity of the fasteners depended on the loading situation; (d) test results suggested that a slip accommodated by nail rotation decreased the strengthening effectiveness. The researchers were able to develop a method to estimate the nominal moment of the MF-FRP strengthened beams and the corresponding FRP strain using the experimental data.

Napoli et al. (2009) conducted an experimental investigation on the effects of fastener layout and laminate length on strength increase and failure mode. A total of six RC one-way slab specimens were tested under four-point bending. The slabs were 3658 mm long; with a cross section of 305 x 152 mm; a clear span of 3048 mm and shear span of 1219 mm. Flexural reinforcement composed of three No. 13 Grade 420 steel bars, corresponding to a longitudinal reinforcing ratio of 0.98%. The concrete strength on the test-day was 26.7 MPa.
The wedge bolts used to attach the FRP laminate were 44.5 mm long and 9.525 mm diameter. The test variables included: number of FRP laminates, width, thickness and length of the laminate, fastener layout and total number of bolts in the shear span.

Results of the beams strengthened with the MF-FRP system were compared to those of beams with the EB-FRP system. Results from the failure modes data showed that, the selection of the fastener layout was appropriate for preventing detrimental brittle failure of the connection as there was no premature shear-out failure in the laminate. Furthermore, it was found that the ultimate failure mode in the members strengthened with the MF-FRP system was characterized by crushing of the concrete after yielding of the steel. For the specimen strengthened with the EB-FRP system, a premature intermediate debonding of the FRP laminate was experienced. At increasing loads, the cracks opened and induced high interfacial shear stress causing debonding of the FRP laminate which propagated outwards along the shear span.

Based on the test results, Napoli et al. (2009) concluded that the flexural strength of RC one-way slabs strengthened with the MF-FRP method was comparable to that attained with the EB-FRP system. Other comparative and contributing factors in the performance of MF-FRP were attributable to the spacing of fasteners and length of the FRP laminate. An appropriate combination of the fastener spacing allowed an increase in the deformability of the system without significantly affecting the ultimate strength.
Wu et al. (2011) conducted an experimental study to develop an Improved Hybrid Bonded FRP (IHB-FRP) technique with the hypothesis that FRP debonding can be effectively prevented through the enhancement of interfacial friction. The anchorage system is shown in Figure 2.2. A total of 20 FRP strengthened RC beams were tested under three-point bending. The beams were divided into three Series; A, B and C. The beams used for Series A and B measured 2300 mm long, 300 mm wide and 150 mm high. For the Series C, the dimensions surpassed both former series by these measurements: 2960 mm long, 400 mm wide and 200 mm high. The compressive strength of the concrete was 66 MPa. All beams were individually reinforced with deformed steel bars in the tension zone and smooth steel bars in the compression zone. The tension steel reinforcement ratio were 0.482%, 0.725% and 0.124% for series A, B and C, respectively. The yield strength of deformed and smooth steel bars were 335 MPa and 235 MPa, respectively. The FRP strips that were externally bonded to the substrate of the beam for Series A and B were 1900 mm long and 50 mm wide. For Series C, the FRP length was 2530 mm and the width was 50 mm. The test variables included: steel reinforcement ratio, number of FRP plies and the fastener spacing.

![Anchorage System Details](image.png)

Figure 2.2: Anchorage System Details (Wu et al. 2011)
Test results showed that the IHB-FRP performed better than the EB-FRP system. With respect to failure modes, the beams strengthened using the latter method failed due to FRP detachment, resulting in the peeling-off a concrete layer measuring a thickness of 2-4 mm. This indicated that the interfacial bond strength of the EB-FRP was insufficient to ensure the full utilization even with a two-ply FRP strip. Conversely, the beams strengthened with the IHB-FRP method failed due to FRP rupture near the mid-span of the beam, but no debonding between the FRP and concrete occurred during the whole loading process. The mechanical fastener was kept in good condition until the beam did finally fail to sustain further load. Furthermore, the ultimate deflection of the IHB-FRP strengthened beam was larger than that of the EB-FRP strengthened beams. Wu et al. (2009) further concluded that FRP debonding can be effectively prevented with the IHB-FRP technique as developed from this study. For a given number of FRP plies and fastener spacing, the flexural strength gain decreased with the increase in the steel reinforcement ratio. The researchers proposed to adapt a simplified method for estimating the ultimate bending moment of the IHB-FRP strengthened beam.

Ebead (2011) examined the effectiveness of a hybrid technique that utilized a combination of externally bonded (EB) and mechanically fastened (MF) FRP strips for the flexural strengthening of RC beams. The objectives were to address the efficiency of using a combination of mechanical fasteners and epoxy for beam strengthening and, to complement previous research on the use of hybrid EB/MF steel and FRP plates for strengthening. A total of 21 beams were constructed with an average compressive strength of 35 MPa. Three beams were
un-strengthened and served as base reference/control, while 18 had steel reinforcements of multiple amounts and different FRP configurations. Stirrups consisted of 6 mm in diameter steel bars. The special hybrid carbon/glass FRP strips used were characterized by a high bearing strength and a high tensile strength with average modulus of elasticity and tensile strength of 72.02 GPa and 1003.4 MPa, respectively. The fasteners used were threaded-type with a shank length of 37 mm and shank diameter of 4.76 mm.

The beams that had FRP strips applied at full-length with epoxy bonding showed the highest post-cracking stiffness among all specimens. It was observed that strains in the FRP strips were lower than those of steel reinforcement due to slip of the FRP. The FRP ‘slip’ led to an ineffective use of the strengthening system, causing a relaxation effect and lower values of FRP strains.

The beams strengthened with hybrid EB/MF-FRP system showed more brittle behavior than those strengthened with the MF-FRP. The ductility of the EB/MF-FRP was concluded in this study as lower than those with the MF-FRP system. The specimens with full-length FRP, the dominating mode of failure was the flexural type initiated at high values of deformations associated with diagonal cracks at the strip end locations. On the other hand, the beams strengthened with partial-length FRP exhibited premature detachment due to the insufficient development length. Ebead (2011) suggested that superior enhancement can be achieved with the use of MF/EB system in terms of ultimate load-carrying capacity and stiffness of beams compared to those using only the MF-FRP without epoxy strip and fastener bonding.
2.3 Research Objectives

This research aims at investigating the effect of corrosion exposure on the performance of RC beams strengthened in flexure with the PAF-FRP system. The PAF have been shot into the concrete without pre-drilling. Although the use of predrilling holes prior to installation of fasteners would minimize the initial damage and cracking, it would compromise speed of installation. Hence, it was decided to shoot the fasteners without predrilling to take advantage of the rapid installation property of this strengthening system which is of a particular importance when immediate use of the structure is imperative. In addition, the inclusion of such deficiencies in terms of local damage and surface spalling during installation of the strengthening system would represent a critical situation that could be encountered in field conditions. This would provide realistic results and allow for a meaningful assessment of the system performance.

The main objectives for the current research are to:

- Examine the effect of different levels of corrosion damage on structural performance of RC beams strengthened with the PAF-FRP composite system.
- Investigate the impact of varying the width of the FRP strip and fastener layout on structural performance at different times of corrosion exposure.
- Introduce an analytical model that can predict the flexural capacity of corrosion-damaged RC beams strengthened with the PAF-FRP system.
2.3.1 Research Significance

Despite the numerous studies on the mechanically-fastened FRP system, no data is available in the literature on the system performance under corrosive environmental conditions. Corrosion cracking and damage of concrete cover could reduce anchorage of fasteners and hence compromise the system performance. The system performance under corrosion exposure should be investigated before the fastened FRP composite system can be employed routinely as a solution to strengthen RC flexural components.

The present research provides experimental evidences for performance evaluation of RC beams strengthened with the PAF-FRP composite system at different levels of corrosion damage. An analytical model for prediction of the flexural capacity of corrosion-damaged RC beams strengthened with the PAF-FRP composite system has been introduced. The research outcome is anticipated to help construction sectors in the UAE to extend the service life of existing RC structures and buildings. Research findings are anticipated to assist in developing guidelines for the application of the mechanically-fastened composite system in the corrosive environment of the UAE and the Gulf region.
CHAPTER 3: EXPERIMENTAL PROGRAM

3.1 Introduction

The effect of corrosion damage on structural performance of RC beams strengthened in flexure with the PAF-FRP composite system has been examined in this research work. The experimental program of the present work consisted of testing a total of twenty one reinforced concrete (RC) beams strengthened with the PAF-FRP composite system. The specimens were subjected to accelerated corrosion exposure to induce corrosion damage to the internal steel subsequent to strengthening.

Details of the experimental program are presented in this chapter. A description of test specimens, fabrication process, material properties, accelerated corrosion methodology, and FRP strengthening system is provided. Test set-up, instrumentation and procedure are also presented.

3.2 Test Program

The test matrix is shown in Table 3.1. A total of twenty-one RC beam specimens were constructed and tested. The specimens were divided into four main groups, [A], [B], [C], and [D] according to the time of corrosion exposure. Specimens of group [A] were not corroded whereas specimens of groups [B], [C], and [D] were subjected to accelerated corrosion for 30, 60, and 100 days, respectively. This corresponded to average tensile steel mass losses of 6, 11, and 18%, respectively. Steel mass losses were measured according to the ASTM G1-
99 (2001). Details of rust removal are given in section 3.7.1 To accelerate the corrosion, a salted mix was used along the testing span with a height of $h/3$ (where $h$ is the beam height) to cover the stainless steel tube bar shown in Figure 3.1. Other test parameters included the width of the FRP strip, 50 and 100 mm; fastener length, 32 and 52 mm; and number of fastener rows, one and two rows.

Group [A] consisted of six specimens that were not corroded. One specimen was not strengthened to act as control beam. Two beams were strengthened using a 50 mm wide FRP strip with 32 and 52 mm long fasteners at a single row. Two beams were strengthened using a 100 mm wide FRP strip fastened with one row of 32 and 52 mm long fasteners. One beam was strengthened using a 100 mm wide strip with two rows of 32 mm long fasteners.

Group [B] and [C] consisted of six specimens each. They were subjected to 30 and 60 days of accelerated corrosion, respectively which resulted in average tensile steel mass losses of 6 and 11%, respectively. One specimen from each group was not strengthened. The remaining specimens of each group were strengthened with the same PAF-FRP regimes as those of the specimens of group [A].

Group [D]

Group [D] consisted of three specimens that were subjected to 100 days of accelerated corrosion which resulted in average tensile steel mass loss of 18%. One specimen was not strengthened. The remaining two beams were strengthened using a single row of 52 mm long fasteners. One beam was strengthened using 50 mm wide FRP strip and one beam was strengthened with 100 mm wide FRP strip.
<table>
<thead>
<tr>
<th>Group</th>
<th>Time of corrosion exposure (days)</th>
<th>Tension steel mass loss (^a) (%)</th>
<th>FRP Strip width (mm)</th>
<th>Fastener length (mm)</th>
<th>No. of rows</th>
<th>Specimen designation (^b)</th>
</tr>
</thead>
<tbody>
<tr>
<td>[A]</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>C0-NF</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>32</td>
<td>1</td>
<td>C0-F50-32</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>32</td>
<td>1</td>
<td>C0-F100-32</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>32</td>
<td>2</td>
<td>C0-F100-32-2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>52</td>
<td>1</td>
<td>C0-F50-52</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>52</td>
<td>1</td>
<td>C0-F100-52</td>
<td></td>
<td></td>
</tr>
<tr>
<td>[B]</td>
<td>30</td>
<td>6</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>C1-NF</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>32</td>
<td>1</td>
<td>C1-F50-32</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>32</td>
<td>1</td>
<td>C1-F100-32</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>32</td>
<td>2</td>
<td>C1-F100-32-2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>52</td>
<td>1</td>
<td>C1-F50-52</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>52</td>
<td>1</td>
<td>C1-F100-52</td>
<td></td>
<td></td>
</tr>
<tr>
<td>[C]</td>
<td>60</td>
<td>11</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>C2-NF</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>32</td>
<td>1</td>
<td>C2-F50-32</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>32</td>
<td>1</td>
<td>C2-F100-32</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>32</td>
<td>2</td>
<td>C2-F100-32-2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>52</td>
<td>1</td>
<td>C2-F50-52</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>52</td>
<td>1</td>
<td>C2-F100-52</td>
<td></td>
<td></td>
</tr>
<tr>
<td>[D]</td>
<td>100</td>
<td>18</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>C3-NF</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>52</td>
<td>1</td>
<td>C3-F50-52</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>52</td>
<td>1</td>
<td>C3-F100-52</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\(^a\) Measured steel mass losses in corroded steel coupons according to the ASTM G1-99 (2001).

\(^b\) C0, C1, C2, and C3 refer to tension steel mass losses of 0, 6, 11, and 18%, respectively. NF refers to no FRP strengthening. F50 and F100 refer to FRP strip widths of 50 and 100 mm, respectively. 32 and 52, refer to fastener lengths of 32 and 52 mm, respectively. 2 refers to two rows of fasteners.
3.3 Specimens Details

A schematic of a typical test specimen showing details of reinforcement and layout of fasteners is shown in Figure 3.1. Test specimen was 1400 mm long RC beam with a rectangular cross section. The beam cross section is 150 mm wide and 210 mm deep. The salted concrete was placed up to $h/3$ of the section depth and the remaining was filled with a mix without salt. The beams were tested under four-point bending with an effective span of 1250 mm and a shear span of 550 mm. The tensile and compressive steel reinforcement consisted of 2 No. 12 and 2 No. 8 deformed steel bars, respectively. The tensile steel reinforcing bars had a 90° hook at each end to provide sufficient anchorage. The shear reinforcement consisted of No. 8 deformed steel stirrups spaced at a spacing of 75 mm. The beam was over designed for shear to ensure a flexural mode of failure will dominate.

3.4 Specimens Fabrication

The casting forms were fabricated using 18 mm thick plywood sheets. The stirrups corners were wrapped using electric tape to avoid being corroded through the accelerated corrosion process but it was not enough to protect them as the stirrups were also corroded. Steel bars were cut to desired lengths, fabricated and then fixed together to form the desired steel cage according to the specimen details as shown in Figure 3.2.

Electrical resistance strain gauges (S.G.) were bonded to the tensile and compression steel reinforcement in at the mid-span section. Bonding of strain gauges followed the steps shown in Figure 3.3.
Figure 3.1: Test Specimen Details
Figure 3.2: Fabrication of steel cages and casting form

(a) Steel cages

(b) Casting forms

Figure 3.3: Bonding the Electric strain gauges (S.G) to steel bars.

(a) Preparing the surface of bonding by smoothing and cleaning.

(b) Isolate wires of the gauge from being in contact with the steel re-bars.

(c) Bonding the gauge to the target area and spacing the wire apart to avoid touching.

(d) Protecting the wire using wax tape to avoid touching during handling.

(e) Protecting the gauge using wax or clay to avoid the damage during casting.

(f) Fixing the wire by “bridging” it to make sure it will not cut through handling.
To ensure that the desired concrete cover was maintained, mortar cement blocks (i.e. spacers) were connected to the steel cages in specific locations. Figure 3.4 shows the steel cages installed inside the forms prior to casting.

![Steel cages installed inside the forms](image)

**Figure 3.4:** Steel cages installed inside the forms

The unsalted concrete was placed first at each end of the form up to a height of \( h/3 \). The salted mix was then placed inside the forms up to a height of \( h/3 \). Finally, unsalted concrete was then placed to fill the remaining depth of the form. The concrete was compacted during casting using a hand held vibrator and then trowel finished after the completion of casting. All specimens were removed from the wooden formwork after three days and then kept moist using wet burlap for seven days. Figure 3.5 shows the specimens after being removed from the forms.
3.5 Material properties

3.5.1 Concrete

The mix that was used in this study to produce nominal concrete grade of 30 MPa contained ordinary Type I Portland cement. The coarse aggregate was natural crushed stone with medium and large sizes 10 mm and 19 mm respectively. The concrete mix proportions for specimens by weight were (cement: sand: medium aggregate: large aggregate: w/c; 1: 1.3: 0.9: 2.1: 0.52). During casting, concrete cylinders with a diameter of 150 mm and a length of 300 mm were sampled. The concrete compressive strength at the time of structural testing was 32 ± 1.7 MPa.

3.5.2 Steel Reinforcement

The bottom longitudinal steel reinforcement was No. 12 deformed bars with a measured yield strength of 538 MPa. The shear reinforcement and the top longitudinal steel reinforcement were No. 8 deformed bars with a measured yield strength of 512 MPa.
3.5.3 Fiber Reinforced Polymers (FRP)

Pultruded FRP strips commercially known as SAFSTRIP® (SAFSTRIP® 2008) were used for flexural strengthening. This type of FRP is a hybrid carbon and glass fiber composite with vinylester matrix. A typical FRP strip has a thickness of 3.2 mm, elastic modulus of 68.3 GPa, and tensile strength of 848 MPa (Lamanna et al. 2001; Lee et al. 2009). According to the manufacturer data sheet, a typical FRP strip has an average tensile modulus of 62.2 GPa, tensile strength of 852 MPa, clamped and unclamped bearing strengths of 351 and 214 MPa, respectively. It should be noted that higher bearing strength values of 270 and 540 MPa were recorded at the onset of bearing damage and bearing failure, receptively (Lee et al. 2009). Details of mechanical properties of materials used in strengthening are shown in Table 3.2.

Table 3.2: Mechanical properties of FRP used in strengthening (SAFSTRIP® 2008)

<table>
<thead>
<tr>
<th>Property</th>
<th>Average Value $^1$ (MPa)</th>
<th>ASTM Test Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile Strength $^*$</td>
<td>852</td>
<td>D-638</td>
</tr>
<tr>
<td>Tensile Modulus $^*$</td>
<td>62,190</td>
<td>D-638</td>
</tr>
<tr>
<td>Clamped Bearing Strength $^*$</td>
<td>351</td>
<td>D-5961</td>
</tr>
<tr>
<td>Unclamped Bearing Strength $^{**}$</td>
<td>214</td>
<td>D-5961</td>
</tr>
<tr>
<td>Open Hole Strength $^*$</td>
<td>652</td>
<td>D-5766</td>
</tr>
</tbody>
</table>

* 20 Sample coupons per test series
** 17 Sample coupons per test series

$^1$ Average value of test series
3.6 Strengthening Methodology

The FRP composite strips were attached to the beams using 4 mm diameter galvanized steel fasteners with a length of either 32 or 52 mm. A special powder-actuated gun was used to shoot the fasteners into the concrete and FRP strip. It is important to highlight that using two rows of fasteners decreased the edge distance from 75 to 50 mm as shown in Figure 3.1. The decreased edge distance caused V-shaped cracks along the corners of the beams when the fasteners were shot into the concrete. Similar behavior was reported in the literature by other researchers (Lamanna et al. 2001; Lamanna et al. 2004). Installation of fasteners with the longer length of 52 mm required a higher power charge which, in some locations, caused local damage and/or surface spalling. Stud expansion anchors with a diameter of 10 mm were installed at the ends of each FRP strip to provide end anchorage and prevent premature end-detachment of the FRP strip. Figures 3.6 and 3.7 show equipment and materials used in strengthening.

Figure 3.6: Equipment and materials used in strengthening: (a) pultruded pre-cured composite strips; (b) fasteners, washers, gun, and power cartridges
3.7 Accelerated Corrosion Process

The impressed current technique is widely used to accelerate corrosion of steel in concrete so that corrosion tests can be completed within a reasonable time frame (El Maaddawy and Soudki 2003). To depassify the tensile steel reinforcement and promote corrosion, 3% NaCl, by weight of cement, was added to the concrete mix used to cast the bottom half of the corroded beams. A constant current of 300 mA was impressed on the steel cage using an external power supply. An internal stainless steel tube with an external diameter of 6 mm and a wall thickness of 1 mm was placed longitudinally at a distance 80 mm from the bottom face of the beam to act as a cathode during the accelerated corrosion process as shown in Figure 3.1. The tensile steel bars and the stainless steel tube were extended out of the beam to facilitate making electrical connections for the accelerated corrosion process. The tensile steel bars were connected to the positive terminal of the power supply to act as an anode while the stainless steel tube was connected to the negative terminal to act as a cathode. Water mist was sprayed over the
specimens to facilitate corrosion reactions. Test specimens under accelerated corrosion exposure are shown in Figure 3.8.

(a) Test specimens under accelerated corrosion.  
(b) Power supply

Figure 3.8: Test specimens under accelerated corrosion exposure

3.7.1 Cleaning of corroded steel coupons

After testing the beams to failure, coupons of corroded steel bars were extracted from the beams to measure the steel loss at the end of each corrosion phase. To remove the corrosion products from the steel there are chemical, mechanical and electrolytic techniques described in the ASTM Standard G1-90. The most suitable method to remove corrosion products from rusted steel bars is chemical cleaning. The cleaning procedure goes through different steps:

- Immersion of the corroded steel in a chemical solution for a specified time
- Washing the steel bars to get rid of the chemicals then drying them.
- Measuring the weight of the steel bars using a calibrated scale.
- Comparing the weight of the corroded coupons to that of uncorroded coupons.

This process was repeated many times until removal of all rust products. When repeating the process, care was taken to avoid removing the base metal of
the corroded steel bars. Six different chemical solutions were recommended by the ASTM Standard G1-90 to clean corroded steel bars; procedure C.3.5 in the ASTM G1-90 Standard was used in this study because;

- It can be done at the room temperature,
- It requires inexpensive chemical products and;
- It takes short time to be done comparing to the other procedures.

The solution was made out of 500 mL of concentrated hydrochloric acid (HCL) with 3.5 gm of hexamethyline tetramine. Reagent water was used to dilute the solution to 1000 mL then it was used to clean a group of steel coupons and was frequently replaced. Stages of chemical cleaning of corroded steel coupons are shown in Figure 3.9.

![Figure 3.9: Stages of chemical cleaning of corroded steel coupons](image)
3.8 Test Set-up and Instrumentation

Specimens of all groups were tested to failure under four-point bending with an effective span of 1250 mm and a constant moment region of 150 mm as shown schematically in Figure 3.10. The load was applied by means of a hydraulic actuator until failure. A spreader beam was used to transfer the load to the test specimen through two loading points which were 150 mm apart. The tests were conducted under incrementally increasing monotonic loading. The deflection at mid-span was monitored using a linear variable displacement transducer (LVDT). The compressive strains at mid-span were measured by using electrical strain gauge (S.G.) bonded to the top face of the specimens and to the top compression reinforcement steel. A strain gauge was also bonded to the tensile steel at the mid-span section. The longitudinal FRP strain was measured at three different locations as shown in Figure 3.1. Figure 3.11 shows a test in progress. A data logger was used to record the readings.

Figure 3.10: Schematic view of the test setup
Figure 3.11: A test in progress
CHAPTER 4: EXPERIMENTAL RESULTS

4.1 Introduction

The viability of flexural strengthening of reinforced concrete beams exposed to corrosion using the PAF-FRP system was investigated in this research work. This chapter presents the results of the experimental program for the different groups. The corrosion damage to the test specimens were observed and measured. The results include the mode of failure; the load capacity, ductility index and the longitudinal FRP strain profile. Special attention was paid to the selected parameters in order to gain a deeper understanding of the corrosion effect on the strengthening effectiveness. These parameters were the fastener spacing, FRP strip width, fastener length and number of fastener rows.

4.2 Test Results

4.2.1 Corrosion Damage

The corroded specimens exhibited similar visual corrosion-related distress in the form of extensive rust stains and longitudinal cover cracking parallel to the corroded steel reinforcing bars. The corrosion crack width was measured at the end of each time of corrosion exposure using a hand-held microscope as shown in Figure 4.1.
Corrosion exposure for 30, 60, and 100 days resulted in average corrosion crack widths of 0.4, 0.6, and 0.9 mm, respectively. The corresponded crack pattern for the corroded beams is shown in Figure 4.2. No delamination or spalling of concrete cover was observed. After the structural test to failure, steel coupons were extracted from the corroded bars, and then cleaned of rust according to the chemical cleaning procedure of the ASTM G1-03 (2011). After rust removal, the weight of the corroded coupons was measured then compared to that of uncorroded coupons. Average tensile steel mass losses of 6, 11, and 18% were recorded at 30, 60 and 100 days of accelerated corrosion exposure, respectively.

![Figure 4.1: Measuring the crack width for the corroded specimens.](image1)

![Figure 4.2: Typical corrosion crack pattern for the corroded control beams](image2)
4.2.2 Failure Mode

Typical failure modes of test specimens are shown in Figures 4.3 to 4.7. Failure of the unstrengthened beams, both corroded and uncorroded, occurred by yielding of tensile steel followed by crushing of concrete in compression zone as shown in Figure 4.3. The beams strengthened with the PAF-FRP system failed by yielding of tensile steel followed by concrete crushing concurrent with progressive pull-out of fasteners/peeling of concrete cover as shown in Figure 4.4. The strengthened beams could, however, undergo a significant deformation after concrete crushing with insignificant drop in load. Ultimately, the strengthened beams failed by sustained bearing failure of the FRP strip at the end anchors as shown in Figure 4.5 accompanied by peeling of concrete cover from the interior of the beam span. Bearing damage occurred only at the end anchors. No bearing damage was observed around the interior fasteners. It is believed that widening of flexural cracks in uncorroded beams caused rotation and pull-out of fasteners which disallowed bearing damage around the interior fasteners. For the corroded-strengthened beams, corrosion damage and cracking of concrete cover facilitated early pull-out of fasteners and peeling of concrete cover without a bearing damage around the fasteners as shown in Figure 4.6. The phenomenon of fastener’s rotation due to widening of flexural cracks with minor or no bearing failure of the FRP around the interior fasteners was also reported in the literature by Lee et al. (2009). Rupture of the tensile steel reinforcement was observed in some of the strengthened beams at failure as shown in Figure 4.7.
Figure 4.3: Crushing of concrete in compression zone

Figure 4.4: Pull-out of fasteners and peeling of concrete cover

Figure 4.5: Sustained bearing failure at end anchors
Figure 4.6: Rotation and pull-out of fasteners

Figure 4.7: Rupture of the tensile steel
4.3 Structural Response

The main test results are summarized in Table 4.1. The yield load for each beam was the load at which the load-deflection relationship for the particular beam was no longer linear. The ultimate (peak) load was the highest load the beam could reach.

Table 4.1: Test results

<table>
<thead>
<tr>
<th>Group</th>
<th>Specimen</th>
<th>$P_y$ (kN)</th>
<th>$P_u$ (kN)</th>
<th>$\Delta_y$ (mm)</th>
<th>$\Delta_{\text{max}}$ (mm)</th>
<th>$\Delta_f$ (mm)</th>
<th>$\mu_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>[A]</td>
<td>C0-NF</td>
<td>72</td>
<td>76</td>
<td>4.9</td>
<td>35</td>
<td>35</td>
<td>7.1</td>
</tr>
<tr>
<td></td>
<td>C0-F50-32</td>
<td>80</td>
<td>86</td>
<td>4.9</td>
<td>33</td>
<td>28</td>
<td>5.7</td>
</tr>
<tr>
<td></td>
<td>C0-F100-32</td>
<td>85</td>
<td>87</td>
<td>6.4</td>
<td>33</td>
<td>26</td>
<td>4.1</td>
</tr>
<tr>
<td></td>
<td>C0-F100-32-2</td>
<td>85</td>
<td>88</td>
<td>5.8</td>
<td>47</td>
<td>47</td>
<td>8.1</td>
</tr>
<tr>
<td></td>
<td>C0-F50-52</td>
<td>85</td>
<td>94</td>
<td>6.9</td>
<td>35</td>
<td>35</td>
<td>5.1</td>
</tr>
<tr>
<td></td>
<td>C0-F100-52</td>
<td>85</td>
<td>90</td>
<td>4.7</td>
<td>27</td>
<td>25</td>
<td>5.3</td>
</tr>
<tr>
<td>[B]</td>
<td>C1-NF</td>
<td>70</td>
<td>75</td>
<td>2.9</td>
<td>25</td>
<td>25</td>
<td>8.6</td>
</tr>
<tr>
<td></td>
<td>C1-F50-32</td>
<td>80</td>
<td>90</td>
<td>3.9</td>
<td>33</td>
<td>21</td>
<td>5.4</td>
</tr>
<tr>
<td></td>
<td>C1-F100-32</td>
<td>80</td>
<td>87</td>
<td>2.9</td>
<td>27</td>
<td>27</td>
<td>9.3</td>
</tr>
<tr>
<td></td>
<td>C1-F100-32-2</td>
<td>80</td>
<td>88</td>
<td>3.7</td>
<td>28</td>
<td>28</td>
<td>7.6</td>
</tr>
<tr>
<td></td>
<td>C1-F50-52</td>
<td>80</td>
<td>90</td>
<td>3.7</td>
<td>39</td>
<td>27</td>
<td>7.3</td>
</tr>
<tr>
<td></td>
<td>C1-F100-52</td>
<td>70</td>
<td>81</td>
<td>3.4</td>
<td>33</td>
<td>33</td>
<td>9.7</td>
</tr>
<tr>
<td>[C]</td>
<td>C2-NF</td>
<td>66</td>
<td>71</td>
<td>2.7</td>
<td>25</td>
<td>25</td>
<td>9.3</td>
</tr>
<tr>
<td></td>
<td>C2-F50-32</td>
<td>70</td>
<td>80</td>
<td>2.9</td>
<td>29</td>
<td>29</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>C2-F100-32</td>
<td>75</td>
<td>81</td>
<td>3.1</td>
<td>40</td>
<td>21</td>
<td>6.8</td>
</tr>
<tr>
<td></td>
<td>C2-F100-32-2</td>
<td>80</td>
<td>84</td>
<td>3.1</td>
<td>40</td>
<td>12</td>
<td>3.9</td>
</tr>
<tr>
<td></td>
<td>C2-F50-52</td>
<td>70</td>
<td>76</td>
<td>3.6</td>
<td>32</td>
<td>23</td>
<td>6.4</td>
</tr>
<tr>
<td></td>
<td>C2-F100-52</td>
<td>70</td>
<td>78</td>
<td>3.5</td>
<td>38</td>
<td>31</td>
<td>8.9</td>
</tr>
<tr>
<td>[D]</td>
<td>C3-NF</td>
<td>60</td>
<td>68</td>
<td>2.5</td>
<td>33</td>
<td>33</td>
<td>13.2</td>
</tr>
<tr>
<td></td>
<td>C3-F50-52</td>
<td>65</td>
<td>74</td>
<td>2.2</td>
<td>48</td>
<td>12</td>
<td>5.5</td>
</tr>
<tr>
<td></td>
<td>C3-F100-52</td>
<td>70</td>
<td>83</td>
<td>3.9</td>
<td>43</td>
<td>15</td>
<td>3.8</td>
</tr>
</tbody>
</table>
The load-deflection curves of specimens of groups [A], [B], [C] and [D] are depicted in Figures 4.8 to 4.11. The load-deflection curve of the control specimen C0-NF is included in all figures for the purpose of comparison.

Figure 4.8: Load-deflection curves of group [A]
Group [A]

For the uncorroded specimens, the PAF-FRP strengthening system with an FRP width of 50 mm and a fastener length of 32 mm increased the yield and ultimate loads by 11 and 13%, respectively relative to those of the control-uncorroded beam C0-NF. For the same FRP strip width of 50 mm, higher gains in yield and ultimate loads of 18 and 24%, respectively were recorded when the fastener length was increased from 32 to 52 mm. For the same fastener length, doubling the width of the FRP strip resulted in a minor or no additional increase in flexural capacity. Increasing the number of fasteners’ rows from one to two resulted in insignificant increase in the flexural strength gain.

Group [B]

Results of specimen C1-NF indicate that minor corrosion of 6% reduction in the mass of the tensile steel reinforcement slightly reduced the beam flexural capacity but significantly reduced the beam ductility relative to those of the control-uncorroded beam C0-NF. The yield and ultimate loads of the corroded-strengthened specimens were on average 11 and 16% higher than those of the corroded-unstrengthened beam C1-NF, respectively. This indicated that up to this level of corrosion, the strengthening system was effective. Specimen C1-F100-32-2 with two rows of fasteners had same load-deflection response as that of its counterpart C1-F100-32 with a single row of fasteners. Increasing the fastener length or FRP strip width did not improve the gain in flexural capacity of the corroded beams of this group because of the initial cracks occurred during strengthening.
a) Specimens with FRP strip width = 50 mm

b) Specimens with FRP strip width = 100 mm

Figure 4.9: Load-deflection curves of group [B]
Group [C]

Moderate corrosion damage of 11% tensile steel mass loss reduced the yield and ultimate loads by 8 and 7%, respectively. The PAF-FRP strengthening system increased the yield load and ultimate loads of the corroded specimens of this group by about 11 and 14%, on average, relative to those of specimen C2-NF, respectively. Despite corrosion damage and cracking, the ultimate loads of the corroded-strengthened beams were still higher than those of the control-uncorroded beam C0-NF. Increasing the fastener length did not result in a further increase in the flexural strength gain. Doubling the width of the FRP strip had insignificant effect on the gain in flexural capacity. The flexural capacity of specimen C2-F100-32-2 with two rows of fasteners was slightly higher than that of its counterpart C2-F100-32 with a single row of fasteners.

Group [D]

The yield and ultimate loads of specimen C3-NF with the highest corrosion level of 18% tensile steel mass loss were 17 and 11% lower than those of the control specimen C0-NF, respectively. The flexural strengths of strengthened specimens C3-F50-52 and C3-F100-52 were about 9 and 22% higher than that of specimen C3-NF that was corroded but not strengthened. Despite the high corrosion damage and cracking, the flexural capacity of the strengthened specimen C3-F50-52 was almost the same as that of the control specimen C0-NF. The flexural capacity of specimen C3-F100-52 was even higher than that of the control specimen C0-NF.
Figure 4.10: Load-deflection curves of group [C]

a) Specimens with FRP strip width = 50 mm

b) Specimens with FRP strip width = 100 mm
4.3.1 Load capacity

For the uncorroded specimens, the PAF-FRP strengthening system with a FRP of 50 mm width and a fastener length of 52 mm increased the yield and ultimate loads by 18 and 24%, respectively. The gain in yield and ultimate loads decreased with increased level of corrosion. For the same FRP strip width of 50 mm and a fastener length of 52 mm the gain in yield load was only 14, 6, and 8%; while the gain in the flexural capacity was 20, 7, and 9% at corrosion levels of 6, 11, and 18% tensile steel mass loss, respectively.

The reduced gain in the yield and ultimate loads in corroded specimens can be ascribed to damage and cracking of concrete cover caused by corrosion which reduced anchorage of fasteners into the concrete and thus compromised the effectiveness of the PAF-FRP strengthening system. Reducing the fastener length from 52 to 32 mm reduced the flexural strength gain of the uncorroded specimens.
For the corrosion-damaged specimens, reducing the fastener length did not result in a further reduction in the flexural strength gain. It seems likely that the fasteners in corroded specimens were loose to show an effect for the fastener length.

At higher levels of corrosion and due to damage and cracking, the fasteners are anticipated to further lose their anchorage with the concrete which would promote early pull-out of fasteners, peeling of concrete cover and FRP, and hence increasing the fastener length would not result in an additional flexural strength gain.

For both corroded and uncorroded specimens with a single row of fasteners, doubling the width of the FRP strip resulted in minor or no additional flexural strength gain. This confirms previous findings reported in the literature by Lamanna (2001), that the use of a single row of fasteners for a FRP strip width of 100 mm was not sufficient to hold the FRP strip firmly and transfer the load effectively to the concrete because of the shear lag effect. Increasing the number of fastener rows from one to two rows insignificantly increased the flexural strength gain. This is because of the initial cracks and damage occurred during installation of fasteners that compromised anchorage of fasteners and hence the contribution of the FRP to the beam flexural capacity.

It should be noted that despite corrosion damage and cracking, the flexural capacity of all strengthened specimens, but C3-F50-52, were higher than that of
the control uncorroded specimen C0-NF. The flexural capacity of specimen C3-F50-52 was almost same as that of the control uncorroded specimen.

4.3.2 Ductility index

The beam deflection capacity, $\Delta_{max}$, is defined as the maximum deflection attained by the beam prior to failure. Large deflection before failure would provide ample warning of structural distress. The beam ductility is the capacity of the beam to sustain large deformation before failure without a significant drop in load capacity. The beam ductility index is typically defined as the ratio of the beam deflection at ultimate load to the deflection at yield load (Mukhopadhyaya et al. 1998). For some of the strengthened specimens, the ultimate load was attained at a deflection much lower than the beam deflection capacity. The residual load capacity of these beams and the corresponding deflection capacity were, however, still considerable. On the basis of this observation, the ductility index of the beams of the present study $\mu_\Delta$ is defined as:

$$\mu_\Delta = \frac{\Delta_f}{\Delta_y}$$  \hspace{1cm} (4.1)

where;

$\mu_\Delta$ = ductility index

$\Delta_f$ = mid-span deflection at ultimate load

$\Delta_y$ = mid-span deflection at the yielding load

For the beams without a softening branch, $\Delta_f$ is the mid-span deflection at ultimate load but for the beams with a softening branch, $\Delta_f$ is the mid-span
deflection corresponding to the 95% of the maximum load measured at the softening branch of the load-deflection curve. For all beams, $\Delta y$ is the mid-span deflection at the yielding load. From Table 4.1, it can be seen that the ductility index of the uncorroded-strengthened specimens was on average 20% lower than that of the control- uncorroded beam C0-NF. The ductility index of specimen C0-F100-32-2 was, however, higher than that of the control specimen. The average ductility index of the strengthened specimens at a corrosion level of 6% tensile steel mass loss was about 3%-37% higher than that of the control- uncorroded beam C0-NF. For the strengthened beams with a corrosion level of 11% tensile steel mass loss, the average ductility index was similar to that of the control-uncorroded beam. At the highest corrosion level of 18%, the ductility index of the strengthened beams was on average 34% lower than that of the control uncorroded beam C0-NF.

4.3.3 Longitudinal FRP Strain Profile

The FRP strain distribution along half of the beam span at the peak load for specimens of groups [A], [B], [C], and [D] are depicted in Figures 4.12 to 4.15. The FRP strain under the load point was assumed equal to that measured at the mid-span section. The proximity of the load point and the mid-span section in addition to the absence of fasteners in the constant moment region justify this assumption. Previous results reported in the literature by Lee et al. (2009) further confirm the validity of this assumption. The distribution and magnitude of the FRP strain were dependent on the fastener length and width of the FRP strip. Among the uncorroded specimens of group [A], specimen C0- F50-52, with a
FRP strip width of 50 mm and fastener length of 52 mm, exhibited the highest FRP strain at peak load while specimen C0-F100-32, with a FRP strip width of 100 mm and fastener length of 32 mm, exhibited the lowest strain. A higher FRP strain at the mid-span section would reduce the stress in tensile steel thus increasing the yield and ultimate loads of the beam. The FRP strain profile of the specimens with the smaller FRP strip width of 50 mm, expect C2-F50-52 and C3-F50-52, varied in proportion to the applied moment. In all other specimens with the greater FRP strip width of 100 mm, the FRP strain profile was almost uniform throughout half of the beam span indicating a poor bonding condition between the FRP strip and the concrete.

Figure 4.12: Longitudinal FRP strain profile at peak load for group [A]
Figure 4.13: Longitudinal FRP strain profile at peak load for group [B]

Figure 4.14: Longitudinal FRP strain profile at peak load for group [C]
The initial cracks and damage occurred during installation of fasteners in the specimens with two rows of fasteners due to the reduced edge distance compromised anchorage of fasteners, and hence the contribution of the FRP strip to the beam flexural capacity. Increasing the edge distance, use of pre-drilled holes, or different types of fasteners would result in better performance. The damage caused by the installation of fasteners in the specimens with two rows of fasteners is shown in Figure 4.16.
4.3.4 Effect of Corrosion on Longitudinal FRP Strain Profile and Fastener Loads

The effect of corrosion on the longitudinal FRP strain profile in the specimens with a FRP strip width of 50 mm is shown in Figure 4.17. The FRP strain profile was significantly affected by level of corrosion damage and cracking. For the specimens with no and minor corrosion of 6% tensile steel mass loss, the FRP strain profile followed the same shape as that of the bending moment. At the moderate and high levels of corrosion of 11 and 18% tensile steel mass losses, respectively, the FRP strain profile was almost uniform indicating inability of the fasteners to transfer the load effectively to the concrete. The FRP strain at the mid-span section reduced with increased level of corrosion damage and cracking.

Figure 4.17: Effect of corrosion on longitudinal FRP strain profile at peak load (specimens with 50 mm wide FRP strip)
The longitudinal FRP strain profile shown in Figure 4.17 has been used to determine the load transferred per fastener in Regions A and B nearest to the support and the mid-span section, respectively as shown in Figure 4.18.

The average transferred load per fastener in a region between sections i and j, $P_{f,avg}$, can be determined as follows:

$$P_{f,avg} = \frac{(\varepsilon_j - \varepsilon_i)E_f A_f}{N}$$  \hspace{1cm} (4.2)

where;

$\varepsilon_i$ = FRP strain measured at a section i nearest to the support

$\varepsilon_j$ = FRP strain measured at a section j nearest to the mid-span

$A_f$ = cross sectional area of the FRP strip

$E_f$ = Young’s modulus of the FRP

$N$ = number of fasteners in a region between sections i and j

If the calculated load per fastener, $P_{f,avg}$, has a negative sign then it shall be taken as zero. In such a case, a complete pull-out of fasteners and delamination has taken place and no forces are transferred to the concrete through the fasteners.

The effect of corrosion on the load per fastener for the specimens with a FRP strip width of 50 mm is shown in Figure 4.19.
From this figure it can been seen that at zero and minor corrosion of 6% tensile steel mass loss, the load per fastener in Region A was lower than the fastener’s load in Region B. On the contrary, at the moderate and high levels of corrosion of 11 and 18%, respectively, the fastener’s load in Region A was higher than the fastener’s load in Region B. In fact at the highest corrosion level of 18%, the fastener’s load in Region B nearest to the mid-span was nil indicating a complete pull-out/delamination of fasteners and peeling of concrete cover in this region at the peak load. The variation of the fastener’s load can be ascribed to early pull-out/delamination of fasteners at certain regions of the concrete soffit which was propagated at a later stage into a global delamination and peeling of concrete cover.
Figure 4.19: Effect of corrosion on the load per fastener (specimens with 50 mm wide FRP strip)

Results of the fastener loads shown in Figure 4.19 indicate that at zero and 6% corrosion, local pull-out/delamination of fasteners occurred first in Region A nearest to the support whereas at 11 and 18% corrosion, local pull-out/delamination of fasteners occurred first in Region B nearest to the mid-span section. From Figure 4.19, it can also be noted that the average load per fastener along the entire length of both regions decreased with increased corrosion level. This further explains how corrosion-related damage and cracking of concrete cover reduced anchorage of fasteners into the concrete and hence reduced the effectiveness of the PAF-FRP system. For the same specimens strengthened with a FRP strip width of 50 mm, the load transferred by the end anchor at peak load has also been calculated based on the FRP strain data measured by SG(1) located
at the vicinity of the FRP strip as shown in Figure 3.1. End anchor forces of 8.1, 9, 4.5, and 5.1 kN that corresponded to bearing stresses of 252, 280, 141, and 160 MPa were recorded at 0, 6, 11, and 18% corrosion, respectively. These values of bearing stresses are smaller than or almost equal to the FRP bearing strength at the onset of bearing damage, 270 MPa, reported by Lee et al. (2009). This indicates that bearing failure of the FRP strip was initiated at onset of concrete crushing (peak load) in the specimens with zero and minor corrosion of 6% whereas it was initiated after concrete crushing in the specimens with the moderate and high levels of corrosion of 11 and 18%, respectively.

4.3.5 Strain Profile along Section Depth

Figure 4.20 depicts the strain profile along the depth of the mid-span section at the onset of concrete crushing for specimens of group [A]. Strains in steel reinforcement in specimens of other groups could not be obtained due to malfunction of strain gages caused by corrosion. From Figure 4.20, it can be seen that the strain in the bonded compressive and tensile steel reinforcement varied linearly with concrete compressive strain at the extreme compression fiber. Although the FRP strip was located below the tensile steel, the strain in the FRP was less than the strain in the tensile steel. The reduced strain in the FRP can be ascribed to rotation and pull-out fasteners that reduced their anchorage into the concrete, and hence increased slip of the FRP strip. Incompatibility of strain between the PAF-FRP strip and concrete would affect the flexural capacity and hence must be taken into consideration during modelling and analysis.
Figure 4.20: Strain distribution along the mid-span section depth at peak load for the uncorroded beams
CHAPTER 5: ANALYTICAL INVESTIGATION

5.1 Introduction

The effect of corrosion exposure on the performance of RC beams strengthened in flexure with the PAF-FRP system was investigated. In this chapter, an analytical model that can predict the flexural capacity of corroded RC beams strengthened with the PAF-FRP system is introduced. The analytical model is based on realistic materials’ laws, and takes into consideration the effect of reduced area of steel due to corrosion and effect of strain incompatibility between the PAF-FRP strip and concrete. Properties of the concrete, steel and FRP described in Chapter 3 were used as input data in the analysis. A comparison between the model predictions and the experimental results is presented and discussed.

5.2 Material Constitutive Laws

5.2.1 Concrete

The assumed stress–strain relationship of concrete in compression illustrated in Figure 5.1 is described by a parabolic relationship (Hognestad et al. 1955; Collins and Mitchell 1987). The assumed parabolic stress-strain relationship is given by:

\[
f(c) = f_c \left[ \frac{2\varepsilon_c}{\varepsilon_{co}} - \left( \frac{\varepsilon_c}{\varepsilon_{co}} \right)^2 \right]
\]  

(5.1)
where:

\[ f'_c = \text{The concrete compressive strength (MPa)}, \]
\[ \varepsilon_{cu} = \text{The concrete strain corresponding to the concrete compressive strength} \]
\[ \varepsilon_c = \text{The concrete strain for a given loading condition}, \]
\[ f_c = \text{The concrete stress for a given concrete strain (MPa), and} \]
\[ E_c = \text{The Young's modulus of the concrete (MPa)}. \]

The ACI Standard 318-05 relates the Young's modulus of concrete to the square root of the concrete compressive strength as given in Equation 5.3. The concrete strain at crushing, \( \varepsilon_{cu} \), is taken as 0.003. The tensile stresses in the concrete after cracking were neglected.

![Assumed stress-strain relationship of concrete](image_url)

Figure 5.1: Assumed stress-strain relationship of concrete
5.2.2 Steel Reinforcement

The stress-strain relationship of the steel reinforcement is idealized to be linear elastic-plastic with a post-yield strain hardening of 1% (El Maaddawy et al. 2005) as shown in Figure 5.2.

\[
f_s = \begin{cases} \varepsilon_s E_s, & \text{Pre - yield stage} \\ f_y + E_{sp} (\varepsilon_s - \varepsilon_{sy}), & \text{Post - yield stage} \end{cases}
\]

Figure 5.2: Idealized stress-strain relationship of steel reinforcement

where:

- \( \varepsilon_s \) = The steel strain for a given load condition,
- \( f_s \) = The steel stress corresponding to \( \varepsilon_s \),
- \( f_y \) = The steel yielding stress,
- \( \varepsilon_{sy} \) = The steel strain corresponding to the yield stress \( f_y \),
- \( E_s \) = The modulus of the steel reinforcement before yielding (pre-yield stage),
\( E_{sp} \) = The modulus of the steel reinforcement after yielding (post-yield stage),

\( f_{su} \) = The steel ultimate strength, and

\( \varepsilon_{su} \) = The steel strain corresponding to the steel ultimate strength \( f_{su} \).

### 5.2.3 Fibre Reinforced Polymer

The stress-strain relationship of the FRP composite strip is idealized to be linear-elastic up to failure as shown in Figure 5.3.

\[
f_f = \varepsilon_f E_f \leq f_{fr}
\]  

(5.5)

![Idealized stress-strain relationship of CFRP laminates](image)

Figure 5.3: Idealized stress-strain relationship of CFRP laminates

where:

\( f_f \) = stress in FRP

\( \varepsilon_f \) = The CFRP strain for a given load condition,

\( E_f \) = Young’s modulus of the FRP

\( f_{fr} \) = tensile strength of FRP
5.3 Model Development

5.3.1 Compatibility Requirements

Experimental test results indicated that for the specimens strengthened with PAF-FRP system the strain in the compression and tensile steel reinforcement varied linearly with concrete compressive strain at the extreme compression fiber, but the strain in the PAF-FRP strip did not. Accordingly, a deterministic strain reduction factor, $\kappa$, has been introduced to account for the reduced strain in the FRP caused by rotation and/or pullout of fasteners that increased slip of FRP. Strain and stress distributions along section depth are shown in Figure 5.4. The strain $\varepsilon_z$ at any distance $z$ from the neutral axis of the concrete section is given by:

$$
\varepsilon_z = \kappa \frac{\varepsilon_{cu} z}{c}
$$

(5.6)

where:

$\kappa$ = strain reduction factor

$\varepsilon_{cu}$ = the strain at the top face of the beam

$c$ = The depth of the neutral axis measured from the top face of the beam
The strains in the compression steel, tensile steel, and FRP strip are then given by:

\[ \varepsilon_c' = \frac{\varepsilon_c (c - d')}{c} \]  
(5.7)

\[ \varepsilon_s = \frac{\varepsilon_c (d - c)}{c} \]  
(5.8)

\[ \varepsilon_f = \kappa \varepsilon_c (h - c) \]  
(5.9)

where:

- \( \varepsilon_c' \) = The strain in the compression steel reinforcement,
- \( \varepsilon_s \) = The strain in the tensile steel reinforcement,
- \( \varepsilon_f \) = The strain in the longitudinal CFRP laminate,
- \( c \) = The depth of the neutral axis measured from the top face of the beam,
- \( d' \) = The depth of the compression steel measured from the top face of the beam,
- \( d \) = The depth of the tensile steel measured from the top face of the beam,
- \( h \) = The height of the concrete cross section.
The concrete crushing strain $\varepsilon_{cu}$ was taken as 0.003 (ACI 318-05). For bonded steel and FRP reinforcement $\kappa = 1$, but for a PAF-FRP strip $\kappa < 1$. The FRP strains measured experimentally at the mid-span system at peak load and those calculated for an assumed fully-bonded FRP strip along with the corresponding FRP strain reduction factor $\kappa$ are given in Table 5.1. Since no bearing failure occurred around the interior fasteners, the FRP strain reduction factor was dependent on the fastener length, number of rows, and level of corrosion in tensile steel reinforcement. For the specimens strengthened with a 100 mm wide FRP strip fastened with a single and double-row of fasteners, the FRP strain reduction factors were on average 0.1 and 0.15, respectively. For the specimens with a FRP strip width of 50 mm, the FRP strain reduction factor reduced with increased level of corrosion as shown in Figure 5.5. Based on a linear regression analysis for the data plotted in Figure 5.5, Equation 5.10 has been developed. The average values of the FRP strain reduction factor calculated for the beams with a 100 mm wide FRP strip are also given in Equation 5.10. It is important to point out that this equation is valid only for FRP strips mechanically fastened by powder-actuated fasteners (PAF) shot into the concrete without predrilling. The use of predrilled holes or other types of fasteners such as threaded screws or expansion bolts would improve the contribution of the FRP strip to the flexural capacity. In such cases, Equation 5.10 would underestimate the strain and stress in the FRP and hence the beam flexural capacity. Further research is certainly needed to further develop Equation 5.10 for different concrete strengths,
section geometries, fasteners spacing, and edge distances when PAF are used to attach FRP to concrete without predrilling.

\[
\kappa = \begin{cases} 
0.35 - 0.017 m_l & w_f = 50 \text{ mm} \& l_f = 52 \text{ mm} \\
0.26 - 0.002 m_l & w_f = 50 \text{ mm} \& l_f = 32 \text{ mm} \\
0.10 & w_f = 100 \text{ mm} \& \text{single row} \\
0.15 & w_f = 100 \text{ mm} \& \text{double row}
\end{cases}
\] (5.10)

where:

\(l_f\) = the length of the used fastener

\(w_f\) = the width of the used FRP strip

\(m_l\) = percentage steel mass loss in tensile steel

![Figure 5.5: FRP strain reduction factor versus tensile steel mass loss](image-url)

Figure 5.5: FRP strain reduction factor versus tensile steel mass loss
5.3.2 Equilibrium Requirements

Equilibrium conditions are imposed in terms of axial force and bending moment. In order to calculate the compression force in concrete, the cross-section is discretized into finite layers. The compression force in concrete is calculated by numerical integration of forces in each layer. The steel reinforcing bars and FRP strip are represented by discrete elements.

\[
\sum_{i=1}^{n} f_{ci}A_i + \sum A_{si}f_{si} + \sum A_f f_f = 0
\]  
(5.11)

\[
\sum_{i=1}^{n} f_{ci}A_id_{ci} + \sum A_{si}f_{si}d_{si} + \sum A_f f_fd_f = M_a
\]  
(5.12)

where:

\[
A_i = \text{area of concrete layer } i
\]

\[
A_{si} = \text{cross sectional area of steel bar } i
\]

\[
A_f = \text{cross sectional area of the FRP strip}
\]

\[
d_f = \text{distance between plastic centroid of concrete section and center of the FRP strip}
\]

\[
d_{ci} = \text{distance between plastic centroid of concrete section and centroid of concrete layer } i
\]

\[
d_{si} = \text{distance between plastic centroid of the concrete section and center of steel bar } i
\]

\[
f_f = \text{stress in FRP}
\]

\[
f_{ci} = \text{concrete stress at the center of the layer } i
\]
\[ f_{si} = \text{stress in the steel bar } i \]

\[ M_n = \text{nominal moment strength} \]

In these equations, compressive stresses are taken as positive and tensile stresses are taken as negative. The distance \( d_{ci}, d_s, \) or \( df \) is taken as positive if the corresponding concrete layer, steel bar, or FRP strip, respectively is located above the plastic centroid of the concrete section, otherwise the distance will be taken as negative as shown in Figure 5.4. For a given strain distribution along the section depth at the onset of concrete crushing, the sectional forces were integrated numerically and the flexural load capacity of the beam was predicted using an iterative process. It should be noted that the deterministic FRP strain reduction factor, \( \kappa \), was calculated based on the FRP strain data measured at the onset of concrete crushing and hence can be used only for prediction of the flexural load capacity. A more comprehensive regression analysis of the FRP strain data at various levels of loads for each specimen is needed to further develop Equation 5.10 and predict the entire load-deflection curve.

5.3.3 Model Procedure

The model procedure used to predict the load carrying capacity can be summarized as follows:

- Assume depth of the neutral axis \( c \).
- Calculate the strain in each layer of concrete, steel bars, and FRP strip using Equation 5.6.
• Calculate the stress in each layer of concrete, steel bars, and FRP strip using the materials constitutive laws.
• Calculate the forces in concrete, steel and FRP.
• Iterate the assumed neutral axis depth until equilibrium of forces (Equation 5.11) is satisfied.
• Calculate the moment capacity using Equation 5.12, then the corresponding load carrying capacity.

5.4 Comparative Analysis

All test specimens were analysed according to the model’s procedure. Section geometry and materials properties reported earlier were used as input data in the analysis. A comparison between the experimental and predicted load carrying capacities is given in Table 5.1. The predicted load capacities of the unstrengthened beams were within 6% error band. Higher variations between the experimental and predicted load capacities were recorded for the beams strengthened with the PAF-FRP system. The predicted loads of the strengthened beams were within 14% error band which can still be considered an acceptable margin of error. The variation between the experimental and predicted load capacities can be attributed to a variation in materials properties and/or steel mass losses caused by corrosion. Given the complexity of the problem, it can be stated that the proposed simplified analytical model can give reasonable predictions for the load carrying capacity of corroded RC beams strengthened with FRP strips mechanically-fastened with PAF shot into the concrete without predrilling.
Table 5.1: Comparison between experimental and analytical results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Measured FRP strain at peak load; $\epsilon_{f,\text{exp}}$ (µε)</th>
<th>Calculated FRP strain for a fully bonded condition; $\epsilon_{f,\text{bonded}}$ (µε)</th>
<th>Ratio $\kappa = \left( \frac{\epsilon_{f,\text{exp}}}{\epsilon_{f,\text{bonded}}} \right)$</th>
<th>Experimental load capacity (kN)</th>
<th>Predicted load capacity (kN)</th>
<th>Error in predicted load capacity (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C0-NF</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>76</td>
<td>76.2</td>
<td>0</td>
</tr>
<tr>
<td>C0-F50-32</td>
<td>2110</td>
<td>8984</td>
<td>0.23</td>
<td>86</td>
<td>97.6</td>
<td>+14</td>
</tr>
<tr>
<td>C0-F100-32</td>
<td>680</td>
<td>6833</td>
<td>0.10</td>
<td>87</td>
<td>93.4</td>
<td>+7</td>
</tr>
<tr>
<td>C0-F100-32-2</td>
<td>787</td>
<td>6833</td>
<td>0.12</td>
<td>88</td>
<td>100.2</td>
<td>+14</td>
</tr>
<tr>
<td>C0-F50-52</td>
<td>3314</td>
<td>8984</td>
<td>0.37</td>
<td>94</td>
<td>103.0</td>
<td>+10</td>
</tr>
<tr>
<td>C0-F100-52</td>
<td>803</td>
<td>6833</td>
<td>0.12</td>
<td>90</td>
<td>93.4</td>
<td>+4</td>
</tr>
<tr>
<td>C1-NF</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>75</td>
<td>72.2</td>
<td>-4</td>
</tr>
<tr>
<td>C1-F50-32</td>
<td>2687</td>
<td>9250</td>
<td>0.29</td>
<td>90</td>
<td>93.6</td>
<td>+4</td>
</tr>
<tr>
<td>C1-F100-32</td>
<td>1049</td>
<td>6996</td>
<td>0.15</td>
<td>87</td>
<td>90.1</td>
<td>+4</td>
</tr>
<tr>
<td>C1-F100-32-2</td>
<td>1114</td>
<td>6996</td>
<td>0.16</td>
<td>88</td>
<td>97.2</td>
<td>+10</td>
</tr>
<tr>
<td>C1-F50-52</td>
<td>2335</td>
<td>9250</td>
<td>0.25</td>
<td>90</td>
<td>93.6</td>
<td>+4</td>
</tr>
<tr>
<td>C1-F100-52</td>
<td>295</td>
<td>6996</td>
<td>0.04</td>
<td>81</td>
<td>90.1</td>
<td>+11</td>
</tr>
<tr>
<td>C2-NF</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>71</td>
<td>68.8</td>
<td>-3</td>
</tr>
<tr>
<td>C2-F50-32</td>
<td>1992</td>
<td>9477</td>
<td>0.21</td>
<td>80</td>
<td>90.3</td>
<td>+13</td>
</tr>
<tr>
<td>C2-F100-32</td>
<td>616</td>
<td>7134</td>
<td>0.09</td>
<td>81</td>
<td>87.4</td>
<td>+8</td>
</tr>
<tr>
<td>C2-F100-32-2</td>
<td>1049</td>
<td>7134</td>
<td>0.15</td>
<td>84</td>
<td>94.7</td>
<td>+13</td>
</tr>
<tr>
<td>C2-F50-52</td>
<td>1228</td>
<td>9477</td>
<td>0.13</td>
<td>76</td>
<td>84.5</td>
<td>+11</td>
</tr>
<tr>
<td>C2-F100-52</td>
<td>695</td>
<td>7134</td>
<td>0.10</td>
<td>78</td>
<td>87.4</td>
<td>+12</td>
</tr>
<tr>
<td>C3-NF</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>68</td>
<td>64</td>
<td>-6</td>
</tr>
<tr>
<td>C3-F50-52</td>
<td>728</td>
<td>9802</td>
<td>0.07</td>
<td>74</td>
<td>69.1</td>
<td>-7</td>
</tr>
<tr>
<td>C3-F100-52</td>
<td>948</td>
<td>7330</td>
<td>0.13</td>
<td>83</td>
<td>83.7</td>
<td>+1</td>
</tr>
</tbody>
</table>

*Error (%) = 100 x (predicted load-experimental load) / (experimental load)
CHAPTER 6: CONCLUSIONS &
RECOMMENDATIONS

6.1 Research Summary and Conclusions

For uncorroded RC beams, the PAF-FRP strengthening system without predrilling resulted in up to 24% increase in flexural capacity and 20% average reduction in the beam ductility. The strengthening effectiveness decreased with increased level of corrosion. At the highest corrosion level of 18% tensile steel mass loss, an average flexural strength gain of 15% was recorded but the ductility index was on average 34% lower than that of the control uncorroded beam.

Despite corrosion damage and cracking of concrete cover, the flexural capacity of all corroded beams strengthened with the PAF-FRP system was either higher or almost same as that of the control undamaged beam. Increasing the fastener length from 32 to 52 mm increased the gain in flexural capacity of the uncorroded beams but had no effect on the flexural strength gain of the corroded beams. Increasing the width of the FRP strip from 50 to 100 mm resulted in minor or no additional gain in flexural capacity due to the inability of a single row of fasteners to effectively transfer the load to the FRP strip with the greater width (shear lag effect). The use of two rows of fasteners instead of one row resulted in insignificant increase in the flexural strength gain due to the reduced edge distance that caused initial damage and cracking of concrete cover during installation of fasteners.
The results presented in this thesis provided insights into the flexural response of corroded RC beams strengthened with the PAF-FRP system. Further research is needed to expand existing database and support development of standards and design guideline for performance prediction of RC beams strengthened with mechanically-fastened FRP system.

### 6.2 Recommendations for Future Work

The following are recommendations for future studies in this area:

- Study the feasibility of retrofitting large-scale corrosion-damaged RC beams with mechanically-fastened FRP using different fastening systems such as powder-actuated fasteners with predrilled holes, expansion bolts, and threaded screws.

- Examine the performance of mechanically-fastened FRP systems with and without end anchors at degrees of corrosion higher than those of the present study.

- Investigate the fatigue performance of corroded RC beams strengthened with different mechanically-fastened FRP systems.

- Develop the analytical model proposed in this research to encompass a wider range of fastening methods, fastener types and patterns in large-scale RC beams. An analytical model capable of predicting the entire load-deflection curves of corroded RC beams strengthened with mechanically-fastened composites will be beneficial to researchers and practicing engineers.
• The handful of data points of the present study used to develop a formula for the FRP strain reduction factor, κ, need to be expanded to bring the analytical approach into wider use.
REFERENCES


FIB (2006). Retrofitting of Concrete Structures by Externally Bonded FRPs: With Emphasis on Seismic Applications. Federation Internationale Du Beton (International Federation for Structural Concrete), Lausanne, Switzerland.


ملخص الرسالة

تتناول هذه الدراسة تأثير التعرض للتآكل بفعل الصدأ على قدرة الجسور الخرسانية المسلحة المقاومة بالمواد المركبة المثبتة ميكانيكيا (PAF-FRP) لمقاومة عزم الانحناء. تم إعداد واحد وعشرون (21) عينة جسر خرساني مسلح واختبارها حتى حمل الانهيار. تعرضت العينات للصدأ في خلال 30، 60 و100 يوم والذي أدى إلى تآكل في مساحة حديد السماكة بنسبة 6، 11، و18 %، على التوالي. وجد أن الاختبارات تم إجراءها بناءً على عدة متغيرات مثل: عرض شريحة المواد المركبة، طول المسامير، عدد الصفوف المسامير. بالنسبة للجسور الخرسانية التي لم تتعرض للصدأ، أدى نظام PAF-FRP إلى ما يصل إلى 24% زيادة في قدرة مقاومة عزم الانحناء ونحو انخفاض بنسبة 20% في مؤشر لونية الجسور. قلت فعالية نظام التقوية بزيادة نسبة الصدأ والتآكل. ومع ذلك، فإن قدرة مقاومة عزم الانحناء لكل الجسور الخرسانية المقاومة التي تم تعرضها للتآكل والصدأ كانت إما أعلى أو مساوية لقدر مقاومة عزم الانحناء لعينة الجسر الخرساني الذي لم تتعرض للصدأ. زيادة طول المسامير أدى إلى زيادة قدرة مقاومة عزم الانحناء للعينات التي لم تتعرض للصدأ ولكنها لم تشكل أي تأثير على قدرة مقاومة عزم الانحناء للعينات التي تعرضت للصدأ. مضاعفة عرض شريحة المواد المركبة أو عدد الصفوف المسامير لم يكن لها أثر على معدل زيادة قدرة مقاومة عزم الانحناء للعينات.

وقد تم إعداد نموذج تحليلي يمكنه التنبؤ بقدر مقاومة عزم الانحناء للجسور الخرسانية التي تعاني من الصدأ ومازالت المواد المركبة المثبتة ميكانيكيا. يأخذ النموذج التحليلي في عين الاعتبار سلوك المواد غير الخطئي وقد ثبتت صلاحية النموذج التحليلي من خلال مقارنة النتائج النظرية مع النتائج التجريبية.

الكلمات المفتاحية: خرسانة، صدأ، مركب، عزم الانحناء، مسامير مثبتة ميكانيكيا، تقوية، ألياف مقاومة بالبوليمر
جامعة الإمارات العربية المتحدة
كلية الهندسة
قسم الهندسة المدنية والبيئية

سلوك الجسور الخرسانية المسلحة المقاومة بالمواد المركبة المثبتة ميكانيكيًا تحت تأثير عوامل بيئية مسببة للصدأ

أمانة زيد سويد النصيبي

تم تقديم هذه الرسالة لتلبية متطلبات الحصول على ماجستير في الهندسة المدنية والبيئية تحت إشراف

د. تامر المعداوي (المشرف الأساسي)
أستاذ مشارك
قسم الهندسة المدنية والبيئية
جامعة الإمارات العربية المتحدة

د. عمرو الديب (المشرف المساعد)
أستاذ
قسم الهندسة المدنية والبيئية
جامعة الإمارات العربية المتحدة

يونيو 2013