IMPROVING BOND STRENGTH FOR CFRP-RC BEAMS INTERFACE

By

AMAD-ADEEN BAIUK

(BSc, MSc)

Submitted in fulfilment of the requirements for the degree of

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ABSTRACT

Carbon Fibre Reinforced Polymer (CFRP) composites were proved to be an efficient strengthening material for structural elements, including reinforced concrete (RC), due to their superior physical and mechanical properties compared with conventional strengthening materials such as steel plates. CFRP composites have a high strength to weight ratio, high resistance to environmental conditions and are easy to apply. Concrete structures strengthened with CFRP sheets show premature debonding failure before reaching full capacity. The brittle nature and poor toughness of the bonding agent (epoxy resin) are the main parameters that contribute to this premature failure.

The main aim of this research is to investigate the overall behaviour of retrofitted RC beams with respect to strength and ductility in order to overcome the encountered premature failure (debonding).

To achieve the research goal, the neat epoxy resin was modified with two types of liquid rubber modifiers epoxy; namely, carboxyl terminated butadiene-acrylonitrile (CTBN) and amine terminated butadiene-acrylonitrile (ATBN). Rubber modified epoxy has greater toughness compared to neat epoxy and hence provides more ductility for the retrofitted member.

An experimental investigation included testing 10 RC beams strengthened with multilayers of CFRP subjects to four-point bending up to failure. Test results showed that the retrofitted beams with a modified epoxy resin exhibited more ductile behaviour compared to the beams used a neat epoxy resin. In addition, the ductility of the retrofitted beams has been improved up to 66% and 42% when using ATBN-modified epoxy and CTBN-modified epoxy, respectively.
A mathematical model has been developed to predict the behaviour of beams retrofitted with multi-layered CFRP sheets that allowed for interlayer slip and non-linear material properties. The mathematical model has been verified with current test results and previous tests carried out by other researchers. The predicted results of test beams such as failure load, deflection, interface slip and differential strain were generally within 10% of the experimental results. Moreover, a parametric study on a virtual beam confirmed the influence of the shear stiffness of the bonding agent on the beam’s overall behaviour.
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LIST OF ABBREVIATIONS AND NOTATIONS

Abbreviations

A1- A10 Code of Test Beams
AF Aramid Fibre
AFRP Aramid Fibre Reinforced Polymer
ATBN Amine-Terminated Butadiene-Acrylonitrile
CF Carbon Fibre
CFRP Carbon Fibre Reinforced Polymer
CN-E Cyanoacrylate Adhesive
CTBN Carboxyl-Terminated Butadiene-Acrylonitrile
DMTA Dynamic Mechanical Thermal Analysis
FRP Fibre Reinforced Polymer
GF Glass Fibre
GFRP Glass Fibre Reinforced Polymer
NE Neat Epoxy
PS Polyester Resin
RC Reinforced Concrete
SG1- SG3 Strain Gauges

Notations

\( A_c \) Cross-sectional area of concrete in compression member, mm\(^2\)
\( A_{1,2,3} \) Area of CFRP sheet, mm\(^2\)
\( d_{1,2,3} \) Distance between centroids
\( E' \) Storage modulus of the resin, MPa
\( E_{1,2,3} \) Tensile modulus of elasticity of CFRP layers, MPa
$E_c$  Modulus of elasticity of concrete, MPa  
$E_f$  Tensile modulus of elasticity of CFRP, MPa  
$E_{cm}$  Scant modules of elasticity of concrete, MPa  
$f_{cd}$  Design value of concrete compressive strength  
$f_{ck}$  Characteristic compressive cylinder strength of concrete at 28 days  
$f_{cm}$  Mean value of concrete cylinder compressive strength  
$f_f$  Stress level in FRP reinforcement, MPa  
$F_c$  Axial forces in concrete, kN  
$F_{1,2,3}$  Axial forces in CFRP layers, kN  
$K_s$  Shear stiffness of the bonding agent, N/ mm$^2$  
$m$  Number of nodes  
$M_c$  Moment of concrete  
$P$  Applied load, kN  
$q$  Uniform shear flow of the bonding agent, N/mm  
$Q$  Longitudinal shear force of the bonding agent, N  
$T_g$  Glass transition temperatures, °C  
$U_{1,2}$  Interface slip between first CFRP layer and second CFRP layer  
$U_{2,3}$  Interface slip between second CFRP layer and third CFRP layer  
$U_{1,2,3}$  Longitudinal displacement in CFRP layers  
$U_{c,1}$  Interface slip between the concrete and first CFRP layer  
$U_{1,2,3}$  Longitudinal displacement in CFRP layers  
$U_{1i,2i,3i}$  Total longitudinal displacement of CFRP layers  
$U_{ci}$  Total horizontal displacement of the concrete component  
$V_c$  Shear force of concrete, kN  
$W_{1,2,3}$  Vertical displacement in CFRP layers  
$W_c$  Vertical displacement in concrete
<table>
<thead>
<tr>
<th>Symbol</th>
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<tr>
<td>$x$</td>
<td>Subscript denotes the differentiation with respect to $x$</td>
</tr>
<tr>
<td>$Z_c$</td>
<td>Distance from the concrete surface to the origin of axes</td>
</tr>
<tr>
<td>$\varepsilon_{1,2,3}$</td>
<td>Axial strain in CFRP layers</td>
</tr>
<tr>
<td>$\varepsilon_c$</td>
<td>Axial strain in concrete</td>
</tr>
<tr>
<td>$\varepsilon_{c1}$</td>
<td>Strain at ultimate stress</td>
</tr>
<tr>
<td>$\varepsilon_{c2}$</td>
<td>Strain at reaching the maximum strength</td>
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<td>$\varepsilon_{cu1}$</td>
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<td>$\sigma_c$</td>
<td>Compressive stress in the concrete, MPa</td>
</tr>
<tr>
<td>$\sigma_{ys}$</td>
<td>Yield strength of steel, MPa</td>
</tr>
<tr>
<td>$\sigma_{UTS}$</td>
<td>Ultimate tensile stress of steel, MPa</td>
</tr>
<tr>
<td>$\mu_d$</td>
<td>Ductility index</td>
</tr>
<tr>
<td>$\Delta_u$</td>
<td>Deflection at ultimate load, mm</td>
</tr>
<tr>
<td>$\Delta_x$</td>
<td>Distance between two adjacent nodes, mm</td>
</tr>
<tr>
<td>$\Delta_y$</td>
<td>Deflection at the yield load, mm</td>
</tr>
<tr>
<td>$\delta x$</td>
<td>Length of composite element, mm</td>
</tr>
<tr>
<td>$\rho$</td>
<td>Uniformly distributed load, kN/mm</td>
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</tbody>
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CHAPTER 1
INTRODUCTION

1.1. Overview

Reinforced concrete (RC) structures often face challenges in urban development due to the modifications and improvements required during their service life. The contributing factors that necessitate modifications include deterioration due to environmental effects, change of use and natural incidents such as earthquakes.

The costs of maintenance, rehabilitation and upgrading of concrete are among the most critical issues for structural members such as bridges and buildings. The high cost of rehabilitation materials and labourers, environmental impacts, and inconvenience to disturb daily life. For this reason, upgrading structural members make retrofitting methods an efficient technique. Structural engineers are in favour of using Fibre Reinforced Polymer (FRP) composite instead of steel plates to strengthen RC structures [1-10]. FRP has a higher strength to weight ratio, is easier to handle and install, has better resistance to environmental conditions, and is flexible enough to fit into complex designs.

The use of retrofitting methods depends on several factors such as the amount of additional strength required, the environmental conditions, and the complexity of the member’s shape. Currently, strengthening methods using an externally bonded Carbon Fibre Reinforced Polymer (CFRP) composite sheet is preferred, and is a useful technique for improving structural performance. It is an appropriate method applied to many types of RC structures including columns, beams, slabs, walls, chimneys, tunnels and silos.
1.2. Problem Statement

CFRP is recommended by many researchers as a suitable material for the external strengthening of RC structures. CFRP composites improve the ability of RC beams to sustain extra loads. However, the premature (debonding) failure of RC structures retrofitted by a CFRP composite is a major problem of this application. Debonding is a significant failure mechanism of concrete epoxy interfaces. The crack initiated at the high-stress zone then propagates along the weakest part of the concrete substrate or concrete-epoxy interfaces. The debonding failure could occur in many ways, such as plate end debonding, concrete cover delamination, plate end interfacial debonding and midspan interfacial debonding.

The general behaviour of a composite member of two different materials depends on the shear connection (bonding agent) in the interface and other supporting mechanics such as wrapping or bolted plate. The bonding agent is used to transfer stress from the retrofitted member to the CFRP layers by shear [11, 12]. The brittle nature and poor toughness of the epoxy resin restrict the bond efficiency in this application and tend to lead to premature failure [13].

In practice, most composite sections are designed based on the full interaction concept. When using a rigid bonding agent, the value of the interface slip is assumed to be equal to zero and infinitely rigid.

1.3. Aim of the Research

The objective of this research is to investigate the strength and ductility of RC beam externally retrofitted with CFRP composites and to overcome premature failure experimentally and theoretically.

In the experimental section, the study aimed to investigate the strength and ductility behaviour of the retrofitted beam by improving bond behaviour. The aim was achieved by modifying the epoxy resin available for use in this application in order to improve its ductility and toughness,
and consequently to improve the mechanical behaviour of the composite section and hence delay or prevent the premature failure of the composite members.

The epoxy was modified using two different types of reactive liquid polymers: Carboxyl-Terminated Butadiene-Acrylonitrile (CTBN) and Amine-Terminated Butadiene-Acrylonitrile (ATBN).

Theoretically, most of the available mathematical models assumed full interaction behaviour for the CFRP/RC beam composite section. In this respect, the interface slip was assumed to be equal to zero. Therefore, the adhesive was considered infinitely rigid. In previous work [14-16], researchers measured the interface slip between CFRP/RC beams, and confirmed the partial interaction behaviour, providing a basis for incorporating the differential strain in a reliable mathematical model. Few mathematical models in the literature [17, 18] consider partial interaction; however, models have assumed elastic material properties. In this work, a reliable mathematical model was developed to predict the overall behaviour and strength of the composite section along the beam span incorporating non-linear material behaviour.

Hence, the primary research goal was:

“To overcome the debonding failure and minimise the interfacial shear concentration at RC beams retrofitted with CFRP sheets and predict the partial interaction behaviour of such beams”.

1.4. Methodology

To understand the premature failure of the RC beam retrofitted by multilayer CFRP sheets, the research was divided into two main parts.

1.4.1. Experimental Investigation

In order to understand the premature failure occurs in RC beams retrofitted with multilayered CFRP composites. The experimental investigation included two stages.
1.4.1.1. Investigating modified epoxy resin

The application of the carbon fibre matrix required the use of a bonding agent to provide sufficient integrity in the composite cross-section. The bonding efficiency is influenced by many parameters, such as the types of hosting surface, a method of application, contact area and loading. Hence, defining its strength on the existing boundary conditions becomes essential for accurate analysis and modelling. The aim of this study was to investigate the mechanical properties of the bonding agent of the modified epoxy with respect to longitudinal shear stiffness. To investigate the bond on different types of hosting surfaces, the single-lap shear test was performed on 9 concrete prisms and 21 steel plates having carbon fibre matrix bonded to the surface.

The study is focused on several parameters such as modifying the bonding agent, the number of the CFRP sheet and applied load rate. Dynamic Mechanical Thermal Analysis (DMTA) was used to measure the viscoelastic properties of the modified epoxy.

1.4.1.2. Investigating the Behaviour of RC Beam Retrofitting by CFRP and Modified Epoxy

Structural engineers are in favour of a more ductile behaviour and a more delayed debonding failure for retrofitted structures. The aim of this study was to investigate the improvement of strength and ductility of the RC concrete beams. To achieve the aim, 10 RC beams were designed and cast at the Concrete Lab. The dimensions of these beams were 2300 mm in length, 150 mm in width and 250 mm in depth. One of these beams was kept as a control beam. Nine beams were strengthened with varying CFRP layers and using two modified epoxy resins. A four-point bending test was applied to all the beams up to failure, using a 500 kN bending machine.
1.4.2. Theoretical investigation

A mathematical model was developed to simulate the performance of RC beams strengthened with a multilayered CFRP matrix, to predict the general behaviour of composite sections that allow for the inter-layer slip and non-linear material properties. The objective of this model is to simulate and describe the partial interaction behaviour of composite beams and verify it against experimental results.

1.5. Thesis Structure

The thesis has been divided into eight chapters.

The introduction in Chapter 1 of this thesis explored general information about the strengthening and rehabilitation of existing structural members using FRP composites. In addition, this chapter described the problem statement, the aim of the research, and the methodology used to achieve the research aims.

Chapter 2 is literature review provides information about CFRP materials used for concrete rehabilitation. Also, this chapter presents a critical review of the use of CFRP composites and steel plates for retrofitting existing concrete structures.

The first section of the experimental part of the thesis is presented in Chapter 3. In this chapter, the MBrace epoxy resin, which is used to bond CFRP to the RC member, was modified using two types of reactive liquid polymers in order to improve the ductility and the toughness of the neat epoxy. A DTMA test was applied to the neat and modified epoxies to measure the viscoelastic properties of the modified epoxy. In addition, the mechanical properties of neat and modified epoxies are tested on both concrete prism and steel plate surfaces under a single-lap shear test.

Chapters 4 and 5 describe the experimental programme and the test results for the RC beams. Ten RC beams (2300 mm x 250 mm x 150 mm) were tested under the same conditional load,
across a number of CFRP layers and different types of epoxies. The beam deflection, interface slip and strain behaviour along the load gradient until the failure is recorded and plotted.

Chapter 6 focuses on the theoretical part of the thesis. In this chapter, a mathematical model is presented for dealing with RC beams strengthened using multilayered CFRP matrix that allows for inter-layer slip and partial interaction. The model will consider inelastic material properties and the programme built using the MATLAB software.

Chapter 7 is devoted to verifying the model prediction with test results. Moreover, this chapter presents a study of the effect of changing the values of some key parameters.

Chapter 8 describes the outcomes for the experimental and theoretical parts of the thesis. Suggestions for future studies are provided at the end of this chapter.

1.6. Conclusion

The use of FRP composite materials in structural engineering saw significant growth over the past two decades. These materials have proven themselves useful for improving the strength of existing structures.

External strengthening is a useful technique for the strengthening of RC using FRPs, particularly the use of multilayered CFRP bonded to the tension face of the concrete. This method will improve the flexural strength of the structural member, along with its tensile strength, shear force and ductility. In addition, the structural member will become more durable against environmental effects.

Several issues need to be addressed when using the multi-layered CFRP as the external strengthening of RC element. These issues (such as the forces interacting through the concrete, the FRP composite and the adhesive interface) raise concerns about the safety of using CFRP in the infrastructural application. However, available design codes addressing these issues have not been finalised.
Debonding is one of the most common problems affecting the integrity of RC beams strengthened with CFRP sheets. To date, researchers have focused on finite element analyses of debonding. There is no doubt that critical to this field are experiments using solid models and observations of whether debonding has occurred. It is a challenge to investigate debonding using mathematical modelling. However, a model is more efficient than experimentation. A mathematical model will be used to describe the RC beams externally strengthened with varying number of CFRP layers bonded to the tension face and tested under a four-point bending setup until failure.

Test results and observations will be used to verify predicted behaviour through a mathematical model. A parametric study will be carried out to investigate the influence of various parameters of the overall behaviour of strengthening elements, using the developed mathematical model.
CHAPTER 2
LITERATURE REVIEW

2.1. Introduction

This chapter provides a comprehensive review of the FRP application on concrete structural rehabilitation; it focuses on RC beams externally attached CFRP composite. The literature review is divided into several sections that explore information related to FRP composites, applications of FRP, externally strengthening of concrete structures, types of failure modes and interface bond properties. In section 2.2, basic information about FRP composites is explored, including its mechanical behaviour. In section 2.3, the application of FRP composites in structural engineering is explained. Section 2.4 explores the external strengthening of concrete structures using both steel plates and FRP composite. The failure modes that occur with the use of FRP composites and the main parameters affecting the failure criteria are also explained in section 2.4. The interface bond agent and interfacial stress-strain distribution are described in section 2.5. Existing models proposed to overcome debonding failure that occurs at the interface of FRP/concrete are explored in section 2.6.

2.2. Fibre Reinforced Polymer (FRP)

FRP composites consist of high tensile strength fibres embedded in a matrix of polymer resin, as shown in Figures 2.1 and 2.2. The fibres are usually carbon, glass or aramid surrounded by a matrix such as epoxy or vinyl ester. FRP is available as laminates or sheets that can be fabricated using pultrusion or pre-impregnated fibre mats processes.
Under loading, the fibres have a linear elastic property until failure [21, 22]. The matrix’s role in the composite is to protect the surface of the fibres from mechanical abrasion and environmental conditions. In addition, the matrix allows efficient stress transfer between the composite constituents.

Currently, FRP composites are used in rehabilitating applications in two forms; namely, pre-cured strips or uncured sheets [7]. The pre-cured strips are fabricated from a unidirectional fibre with a typical thickness of 0.5-1.5 mm and width of 50-200 mm. The uncured sheets are fabricated from unidirectional and/or bidirectional fibres with a thickness of less than 1 mm. The uncured sheets are pre-impregnated or in situ impregnated to bond to the retrofitted member using a bonding agent [23].

### 2.2.1. Mechanical Properties of FRP Composite

FRP composite has better mechanical properties than conventional materials such as steel plates. Figure 2.3 shows the typical stress-strain relationship of different FRP composites compared with mild steel; the mechanical properties are listed in Table 2.1.

There is a vast difference between FRP material tensile strength compared with mild steel; for example, the tensile strength of CFRP is five times greater than that of mild steel. However, the ductility of mild steel is better than that of FRP composites.
The mechanical properties of FRP composite are affected by several factors, including the type of fibre and matrix properties, fibre content, fibre orientation and the fibre/matrix interface. The FRP composite is classified according to the fibre orientation form as unidirectional, bidirectional or multidirectional.

### 2.2.2. Characteristics of FRP Composite

The lightweight characteristic of FRP composite material makes it easy to handle and install. Unlike steel plates, FRP composite materials do not need a heavy lifting machine, bolts, drilling.
or any other mechanical anchor to fix to the host member. FRP composite materials simply need an adhesive agent (epoxy) to bond to concrete, as shown in Figure 2.4.

![Figure 2.4: The installation of FRP to the structure member [27]](image)

FRP composites are not limited to a particular length, whereas steel plate has a maximum length of six metres. FRP composite materials are durable, low-maintenance and easy to repair by adding a layer.

On the other hand, the production cost of FRP composite materials is a reason to limit its use. FRP composite material is more expensive than other conventional strengthening materials such as steel plate. Overall cost, including labour, wages, access and material cost should be taken into consideration. FRP composite has a high risk of deterioration due to removing or scratching the bonded plate. However, damaged areas can be easily repaired.

2.3. Applications of FRP Composites in Structural Engineering

FRP composite can be applied to a structural member in three ways:

A. Retrofitting the existing structures by bonding and wrapping the FRP composites to the existing reinforced concrete beams in order to:
I. Enhance the flexure strength as shown in Figure 2.5a, and increase the shear strength of beams and slabs by bonding the FRP sheets to the tension face and side face, respectively, as shown in Figure 2.5b.

![Figure 2.5: Retrofitting of the existing structures](image)

a) Enhancing flexure strength  
b) Increasing shear strength

II. Enhance flexure, shear and torsion strength of RC columns by wrapping the FRP fabrics and sheets around it to improve structural integrity and prevent buckling, as shown in Figure 2.6.

![Figure 2.6: Column wrapping to improve its integrity](image)

III. Improve the shear strength of the beam-column joints as illustrated in Figure 2.7.

![Figure 2.7: Shear strengthening of beam-column joint](image)
B. FRP composites materials are used to replace conventional materials to improve the efficiency of construction members; these materials include bars, cables and profiles as shown in Figure 2.8.

![Figure 2.8: FRP composite materials](image)

a) Glass- and carbon-reinforced FRP bars  
b) Carbon fibre cable used for bridges

C. FRP composites are used in architectural applications combined with conventional materials such as steel and concrete to create a hybrid structure such as siding/cladding, roofing, flooring and partitions as shown in Figure 2.9.

![Figure 2.9: Hybrid structures](image)

a) Multi-storey framed building  
b) FRP cooling tower

**2.4. Strengthening and Retrofitting of Concrete Structures**

Increasing capacity use and deterioration of existing concrete element are the primary reasons for strengthening existing concrete members. Steel plates and FRP laminates are the most common materials used to strengthen concrete structures [28]. Recently, externally bonded FRP sheets have been extensively used to strengthen existing concrete structures since they are
easy to install and repair. However, bonding techniques suffer from premature failure, which occurs before full capacity is reached. Various studies and design codes have been conducted to mitigate or prevent this bonding failure.

2.4.1. External Strengthening Using Steel Plates

The strengthening of concrete structure members using bonded steel plates was pioneered in France and South Africa at the 1960s [29-33]. Since 1975, many types of research and testing programmes have been conducted at several academic institutions to investigate and evaluate appropriate bonding agents. One of the earliest studies was the strengthening of Quinton Bridge in the United Kingdom (UK) [34-38]. The main observations were that deflection was significantly reduced, and that load capacity increased by 95% and stiffness increased by 35% [36, 39].

Swiss Federal Laboratories for Material Testing (EMPA) investigated existing structural strengthening using steel plate [40]. These investigations confirmed that the external strengthening of RC beams using bonded steel plates is efficient when using an end anchor at high-stress zones [41]. The flexure strength and ductility behaviour of the strengthened beams were improved when the steel plate was anchored.

High levels of stress induced on adhesive/concrete and adhesive/plate interfaces tends to initiate a crack at the plate end, and consequently, the structure member fails due to plate separation [42]. To avoid this type of failure, a more flexible adhesive is used and extends the external steel plate in the region where the tensile strain is concentrated [43]. In addition, thicker steel plates are used for strengthening the RC beams with anchorage steel strips, which increase the strength of the structure and avoid debonding failure. Also, the ductility and load capacity of the strengthened beam are improved [41].
The ductility of the RC beams was significantly increased when anchored with bolts at the ends of the steel plates. The ductility of the beams anchored with steel bolts was found to be inversely proportional to the thickness of the steel plates. The premature failure occurred by a diagonal shear crack initiated around the bolts [41, 44].

Longitudinal shear stress in the concrete/steel plate interface can be reduced using flexible shear connectors (studs) instead of the bonding agent [17, 45]. Shear connectors are usually appropriate for the design of shear force/slip relationship at the interface [11, 46].

The external strengthening of concrete structures using a steel plate improves flexure strength, controls the crack width and increases load carrying capacity. However, limitations of the strengthened member include:

- The surrounding environment influences the steel plates. In particular, the steel plates are susceptible to corrosion that affects the bond integrity leading to failure.
- Fitting the steel plates into complex concrete structures is difficult.
- The heavy weight of the steel plates alters the design load of the retrofitted concrete member.
- It is difficult to transport and to handle and requires special installation equipment.

2.4.2. RC Beams Strengthening Using FRP Materials:

Limitations on using steel plates hastened the need to search for alternative materials for the external strengthening of concrete structures. In the mid-1980s, the use of FRP composites was proven to be effective as an alternative to the use of steel plates for strengthening concrete structures [47-49]. The FRP composite is characterised by resistance to environmental effects such as corrosion and resistance to acids, alkaline and salt across a range of temperatures [50]. Compared with steel, FRP composites are lightweight, non-magnetic and non-conductive. Moreover, the FRP composites are characterised by high specific strength and high stiffness,
making FRP composites easy to handle, transport and fit into complex structures. The premature failure that occurs by debonding the FRP laminate from the host structure and production costs are the main issues for this application [51, 52].

FRP composites are available in three main forms; namely, Glass-Fiber (GFRP), Aramid-Fibre (AFRP), and Carbon-Fiber (CFRP). Despite the high production cost of CFRP, it is more efficient in the external strengthening of concrete members [51, 52]. As mentioned in section 2.2.1, CFRP composite has a higher strength and stiffness and is more lightweight than GFRP and AFRP [24].

2.4.2.1. Failure modes

Structural engineers remain concerned about using FRP composites in structural applications due to the premature failure that occurs [53-57]. There are several types of the failure modes that can be classified, based on the degree of composite action, into two categories.

The first class of the failure modes occurs when the composite action is preserved until the maximum load is achieved. This occurs during concrete crushing before or during yielding of the steel reinforcement, rupture of FRP sheet, or by concrete shear crack initiated at the end of FRP plate/sheets shown in Figures 2.10 a,b and c.

The second class of failure modes is interfacial debonding, which occurs when the composite action is not preserved until the ultimate load. The interfacial debonding failure mode occurs in the form of concrete cover separation as shown Figure 2.10d, plate end interfacial debonding as in Figure 2.10e, and intermediate flexure crack as in Figure 2.10f [8, 53, 54].

Plate end debonding is reported as a dominant failure mode and occurs when FRP composite is bonded to the RC beam [58-61]. A plate end crack is initiated at the end of the FRP plate/sheet caused by high-stress zones and propagated towards the midspan of the beam. It is
considered to be plate end interfacial debonding or concrete cover delamination, as shown in Figure 2.11.

Figure 2.10: Failure modes of RC beams are strengthening with FRP composites [58]

The concrete cover delamination failure starts with a crack initiated at the end of the interface between the concrete and the reinforcement steel where the high interfacial stress zones exist.

Figure 2.11: Debonding and delamination of externally bonded FRP systems [62]
When the crack reaches the level of longitudinal steel reinforcement, the crack propagates horizontally leading to separation of the concrete cover as shown in Figure 2.12.

![Image of concrete cover separation](image)

Figure 2.12: Concrete cover separation [58]

The end plate interfacial debonding failure mode occurs at FPR/concrete interface as illustrated in Figure 2.13. It starts with a crack initiated at the plate end of FRP laminate due to high interfacial shear and normal stresses, and then the crack propagates along the FRP/concrete interface [58]. A thin layer of concrete accompanies with the FRP plate after debonding occurs.

![Image of crack propagation](image)

Figure 2.13: Typical failure mode of plate end interfacial debonding [55]

Midspan interfacial debonding failure mode occurs in the shear span zone as shown in Figure 2.14. It initiates in the zone of high moment-shear ratio and propagates toward the support [63].

![Image of crack propagation](image)

Figure 2.14: Typical mode of midspan interfacial debonding [55]

The failure mode changes from FRP rupture to ripping or plate interfacial debonding when the thickness of the FRP plate increases [60]. Rahimi and Hutchinson confirmed that the failure mode changed towards the end plate, and increased normal and shear stress when the plate
thickness is increased [64]. Toutanji, Zhao and Zhang tested several RC beams strengthened with externally bonded multi-layered CFRP sheets. The failure mode changed from CFRP rupture in the constant moment region to delamination of the CFRP layer in the concrete substrate as a result of increasing CFRP layers from three to six layers [65]. Hence, multi-layered CFRP sheets are an appropriate use for the external strengthening of RC beams. CFRP sheet is a thin layer and has the capability to control the failure mode when used to strengthen the concrete structure in the form of multi-layered sheets.

2.4.2.2. Premature Failure Mitigation

To mitigate and control premature failure, the end of the FRP plate/sheet is anchored. There are different ways in which the anchor applied to reaches the full composite action and prevents debonding failure [66]. Ross et al. [67], and Hutchinson and Rahimi [68] reported that the end anchorage positively affects strengthened concrete with FRP composites. Bonacci and Maalej [69] performed several tests on RC beams retrofitted with FRP plates, and showed that debonding of the FRP plate still occurred in half of the tested beams despite end anchorage being used.

Melo, Araujo and Nagato [70] reported that shear strength was improved by 106% when CFRP laminates are fully wrapped with CFRP fabrics as shown in Figure 2.15a. However, Al-Mahaidi and Kalfat [3] stated the fully wrapped anchorage has limited use. Fully wrapped anchorage needs to drill the concrete member to be applied to the complex, which affects the integrity of the retrofitted member.

Al-Amery and Al-Mahaidi [14] tested six RC beams with a multilayer of CFRP sheets and wrapped with CFRP straps in the form of a U-jacket, as in Figure 2.15b. They concluded that the flexure strength was improved by 95% compared with 15% obtained from strengthening the beams without using anchor straps.
Mechanical fastening (bolts or metal stirrups) is used to anchor the FRP laminate. The use of bolts to anchor the FRP laminate, as shown in Figure 2.16, reach levels of beam strengthening over 50% compared with those externally bonded with FRP systems [71]. The shear stress of the strengthened beams improved by over 150% compared with the use of FRP anchor. The brittle failure mode occurred by shear cracks initiated around the bolts [60].

Spadea et al. used the U-shape steel stirrups method to anchor the FRP laminate and showed a significant improvement in beam ductility [72]. Spadea et al. assert that the use of U-shape steel stirrups as an anchorage system is necessary to maintain the composite action between the FRP and the RC beams until failure [73].
El-Hacha et al. [74] prestressed the CFRP sheet using innovative mechanical anchorage through jacking with an anchor fixed on the tension face. This increases flexural stiffness and ultimate load compared with the control beams.

FRP fan anchorage system is a new method used to anchor FRP sheets, developed in Japan by the Shimizu Corporation [75]. It is applied by inserting the cutting strip of the CFRP straps into a pre-drilled hole and fanning the ends of the CFRP strap over the CFRP sheet as shown in Figure 2.17. The CFRP fan anchor improves the strength of the beam retrofitted with CFRP sheets [66, 76, 77]. However, the efficiency of this method depends on several parameters such as anchor depth, anchor size, bend radius and anchor spacing [6, 78].

![Figure 2.17: CFRP fan anchor [6, 76]](image)

The orientation of fibres of the CFRP sheet has a strong effect on strength improvement and failure mode. Norris et al. [79] tested several RC beams strengthened with CFRP sheets and concluded that the stiffness and the strength of the retrofitted beam improved when the CFRP fibres were perpendicularly oriented to the crack of the beam. Norris et al. used a combination of fibre orientations to avoid brittle failure mode.

Al-Mahaidi and Kalf [6] investigated the effect of using unidirectional and bidirectional CFRP fabrics as an anchorage system. The authors concluded that the ultimate load increased by 46% to 57% when the unidirectional CFRP fabric was oriented parallel to the direction of CFRP laminate. The ultimate load increased 128% when the bidirectional CFRP fabric was bonded at ±45° to the direction of CFRP laminate. However, the ultimate load
significantly increased up to 195% when the anchorage system used a combination of unidirectional and bidirectional CFRP fabrics.

The length of CFRP plate/sheet is another parameter changing the type of failure mode. Sebastian investigated the main difference between a failure initiated from a crack tip and an end peel/ripping failure [81]. The author concluded that end peel failure is more likely to occur when the edge of the CFRP plate is bonded further away from the beam support. Yang et al. suggest that there is a relationship between the area of laminate with the CFRP’s ripping failure [82]. Hutchinson and Rahim reported that the strength of the beam is increased when the CFRP laminate is bonded to the full beam length [68].

The stiffness of the CFRP plate significantly affects the mode of failure. A rigid CFRP plate tends to fail by the end peel mode [81, 83]. Yuan et al. [84] demonstrate that the ultimate load of the strengthened beam increases when using a rigid CFRP plate, and beam ductility is significant decreased. CFRP plates with greater stiffness are more likely to fail through delamination [62].

In addition, the strength of the retrofitted beam and the type of failure mode are strongly affected by the reinforcement ratio of the composite cross-section. The strengthening of light reinforcement beams fails through delamination of the CFRP laminate or by rupture of the CFRP sheet [67, 69, 85]. The heavy reinforcement beams failed by crushing of the concrete [67, 86]. Al-Ameri and Al-Mahaidi [14] concluded that increasing the reinforcement area of CFRP composite does not always improve the strengthened beam behaviour due to the interaction of flexural and shear stresses and the fact that the failure mode is shifted towards a brittle shear mode. Increasing the amount of FRP reinforcement by increasing the number of CFRP layers will significantly reduce beam ductility. However, the carrying load capacity increases with increasing FRP reinforcement [14, 65].
The shear stiffness of the tensile reinforcement is able to control the failure initiated from the shear crack. Triantafillou and Plevris stated that FRP laminates and steel reinforcement have the ability to resist the shear, particularly by the dowel action [87]. Hutchinson and Rahimi reported that the increasing of the shear capacity of composites beams is not predictable when using unidirectional composites [68]. Al-Ameri concluded that shear stiffness is a tool for expressing the degree of interaction between composite components [11]. The use of additional anchorage of CFRP straps tends to increase the shear stiffness of the composite component [88].

2.5. The Interface Adhesive Bond

The reliability of the bond used at the FRP-concrete interface is the core success of the external strengthening application. The surface preparation and the mechanical property of the epoxy are the main parameters affecting bond integrity. The quality and the reliability of the material have a significant effect on bond integrity. However, the surface preparation is a critical factor affecting the integrity of the bond [11, 12, 89-91]. The weaker substrate and the non-compact particles should be removed from the concrete surface [92]. Mechanical grinding, sandblasting and high-pressure water jet are commonly used for surface preparation of the concrete [93]. The aim of surface preparation is to obtain a roughened surface and expose the tip of the aggregate to provide a better bonding quality. A small paint brush is used to apply a uniform layer of epoxy on the prepared concrete surface while a metal roll is used to expel the air bubbles entrapped between the CFRP sheet and the concrete interface [14, 64, 94, 95]. The role of the epoxy resin is to transfer the stresses from the CFRP composite to the strengthened concrete member [13, 96-101].

The reliability of the strengthening method depends on bond integrity [11, 12]. Britteness and poor toughness characteristics of the epoxy resin are responsible for premature failure in
strengthening and rehabilitation due to a high cross-linked density [13, 98, 102]. Toughness is defined as the ability of epoxy to undergo plastic deformation in applied stress states. Greater toughenability can be achieved by reducing the crosslink density of the epoxy resin [103]. Butadiene-acrylonitrile based rubbers are the principle liquid elastomers used for the toughening of epoxies [104, 105]. Among them, CTBN and ATBN are used to modify the neat epoxy by introducing the rubbery phase to form the second phase particles that reduce the crosslink density of neat epoxy [106] tends to improve its ductility and toughness behaviour that in turn minimise the stress concentration at the end of the bonded CFRP sheet and improve the ductility of the retrofitted structure [89, 103, 105-110]. The loss of bond integrity between the FRP composites and the concrete substrate lead to premature failure [111].

Al-Tamimi [112] investigate the effect of the UAE harsh environments on the durability of bonding CFRP and the concrete under various load intensity. He concluded that the harsher the environment, the more the resulting bond deterioration. Also concluded that the higher sustained loaded specimens have more chance of failure. Hawileh et al. [113] experimentally investigate the bond behaviour of composite carbon (C), composite glass (G) sheets and their hybrid combinations (CG) under different temperature exposures. The authors concluded that the epoxy adhesives softened and the specimens failed primarily by partial loss of the epoxy adhesives followed by sheet splitting when the bond expose to the temperature range of 200–250 °C. Furthermore, the epoxy adhesives burned and the specimens failed by rupture of the fibres when the bond subjected to 300 °C [114]. Al-Tamimi et al. studied the effect of different harsh environmental exposures on the performance and sustainability of CFRP externally bonded to concrete prisms. The authors concluded that the bond in CFRP systems improved when exposed to an elevated temperature due to a greater polymer crosslinking and a creation of complex interactions in the polymer. Subhani et al. [115] studied the effect of the wet-dry cycle of the marine environment on CFRP-concrete interface for two different types of epoxy.
The authors concluded that the degradation rate in bond strength is less in modified epoxy adhered CFRP with concrete compared to the normal epoxy bonded to concrete beams.

The strain compatibility can be achieved when concrete/steel reinforcement bond and CFRP/concrete interface bond are ideal. The performance of the member’s section is evaluated by the amount of strain transferred from the concrete to the CFRP composite. Interfacial slip occurring in CFRP/concrete disturbs the strain compatibility and thus affects the integrity of the strengthened structure.

The primary role of externally strengthened concrete beams with CFRP composites is to control the interaction of the composite component until failure [42, 116]. Al-Amery and Al-Mahaidi [14] reported that the full composite action can be achieved when the interface slip between the concrete and CFRP layers is controlled.

Many researchers suggest that strain compatibility is not achieved, particularly at the step before failure occurs [53, 117-119]. However, the strain compatibility of the concrete member can be achieved by increasing the depth of the concrete section [73, 87, 120, 121].

The amount and method of the anchorage system have a significant impact on the performance of the composite structure [14, 73]. Bakay concluded that the composite action of the RC beams externally strengthened with CFRP plates without any anchorage was halted at 85% of the ultimate load of the tested beam. The composite action maintained up to 98.6% of the maximum load and the amount of composite action has an effect on the failure [63]. Triantafillou and Plevris [87] asserted that the characteristic of the epoxy resin used has a significant impact on the degree of the composite action at the CFRP/concrete interface. The epoxy resin has a significant effect on the performance of the composite action [122]. However, stiffness, viscosity and flexibility are important factors of the epoxy resin to maintain the
composite action, since one of the roles of the epoxy resin is to transfer stress from the CFRP to the concrete structure [12, 42].

The stiffness of the epoxy resin is associated with premature failure in concrete structures retrofitted with FRP composites. In relation to ductility, use of a flexible epoxy resin helps to increase the bending moment of the retrofitted structure by reducing the stress concentration at the FRP/concrete interface [123]. Debonding failure is delayed when a flexible epoxy resin is used instead of a stiff adhesion [124]. Sebastian [81] stated that the utilisation of a flexible adhesive allows for shear deformation to occur, which consequently permits the CFRP plate to slip; more stress/strain gradient occurs, thus improving the performance of the retrofitted member. Also, Al-Ameri [11] reported that the overall behaviour of the composite structure is significantly affected by the type of shear connection between two components. For this reason, the shear connection (the bond) should be more ductile to sustain the plastic deformation. On the other hand, to overcome the vertical separation, the epoxy resin must have an adequate anchoring system.

Experiments conducted by Chen and Teng [125], Lu et al. [54], Ueda et al. [126] and Yuan et al. [84] concluded that several factors affect the composite structure throughout bond-slip criteria. The concrete compressive strength, the bond length, the axial stiffness of the FRP laminate, the FRP to concrete ratio, the adhesive compressive strength and the shear stiffness of the adhesive are the main factors.

The shear stiffness of the bonding agent ($K_s$), which can be defined as the membrane force, acts at the CFRP-concrete interface, is required to produce a unit length of deformation at the bonding layer [11]. For this reason, the use of an adhesion agent with a softer binder than commercial epoxy provides more flexibility to permit more deformation at the CFRP/concrete interface.
The failure mode of RC beams retrofitted with FRP is affected by the amount of interfacial stress transferred through the bond, as shown in Figure 2.18. These types of stresses have been examined during early testing of externally strengthened RC beams, but when failing has occurred due to the ripping mode. The plate separation occurring during most of the tests was a result of the combination of interfacial stress with peeling force at the plate ends. [90].

![Figure 2.18: Conceptual interfacial stresses of FRP laminate [62]](image)

The premature failure of strengthened RC beams is due to an accumulation of interfacial stress, and peel force that is a result of the high tensile force applied to the tension face of the strengthened beam [42]. Several parameters affecting interfacial stress have been reported by researchers such as:

- A higher compressive strength of the concrete increases the accumulation of the interfacial stress [127].
- A higher stiffness of the epoxy resin rises the interfacial stress with affecting the location of the peak point [8].
- The accumulation of interfacial stress increased as the CFRP plate thickness or the number of CFRP layers increases. However, CFRP plate/layers do not change the position of the interfacial stress peak [8, 64, 127, 128].
- Increased shear span to depth ratio will reduce the interfacial stresses irrespective of the elastic modulus of FRP plates [127].
The strain and stress distribution in the bonded FRP plates have been extensively studied. Hutchinson and Rahimi [68] concluded that there is no clear relationship between strain distribution and the overall behaviour of the strengthened beam. Fanning and Kelly emphasised that the strain gradient is responsible for the plate peel-off failure mode when reaching a particular value [95]. In addition, Maeda et al. [129] confirmed Fanning and Kelly’s research and concluded that there is no distinction in the strain gradient with varied plate stiffness and bond length. From the bond strength test (pull-out test), the strain distribution has a quadratic relationship with maximum values occurring near the loaded zone. The load increases with increased bond length until it reaches a critical length and remains constant until FRP failure [130]. The structure geometry and the surface preparation have a significant effect on the critical bond length [90]. However, the bond strength of the concrete and the FRP composite does not increase when the bond length reaches a particular value [54, 131, 132]. Nguyen et al. [61] concluded that strain development in CFRP laminates in flexure tests can be divided into three zones. The first zone is the de-stressed region at the FRP plate end, the second zone is the region where strain increases with linear behaviour, and the third zone is the composite zone. Al-Ameri [14] suggested that the longitudinal strain in the CFRP layers differs with the number of CFRP layers, which allows for the differential strain to exist within the total thickness of the CFRP matrix. Maeda et al. [123] concluded that the stress distribution is more uniform when a flexible bond is used, which improves the strength of the strengthened member.

2.6. Existing Modelling

Many models have been proposed to overcome the debonding issue at the FRP/concrete interface. Oehlers [133] proposed a model to investigate the debonding failure of FRP strengthened concrete beams. Oehlers obtained the flexural capacity of an externally bonded RC beam based on the classic ultimate strength analysis method and assumed the plate is a part
of the reinforcement. Smith and Teng [58] recommended a safe design equation that assumes the shear force is limited when the applied moment to the ultimate moment capacity ratio is less than 0.67. Teng and Yao [134, 135] proposed an experiential model predicting the flexural debonding of a plate end located in a pure bending region. The authors calculate the shear capacity of the concrete in this model based on three different codes: the British code (BS8110), the Australian code (AS600) and the American code (ACI318). Jansze [136] proposed a model to predict the shear force at the plate end. Ahmed and Van Gemert [137] modified Jansze’s model to be reliable using the entire shear span distance. Colotti et al. [138] proposed a model based on truss analogy to predict the shear strength for different types of failure modes. Ziraba et al. [139] proposed a debonding strength model for RC beams strengthened by steel plates to predict plate end interfacial debonding, which takes into account shear stiffness and normal stiffness of the adhesive layer properties. Wu and Niu [140] proposed a model based on the fracture mechanics principle, which is able to predict the debonding failure of FRP sheets bonded to RC beams in flexure strengthening that was initiated by intermediate flexural cracks.

A new model to predict the end debonding and intermediate crack debonding for RC beams retrofitted with FRP was proposed by Casas and Pascual [141]. Triantafillou [142] proposed a strength model using a semi-empirical approach to predict the shear contribution of FRP, through a known value of the effective strain in which the shear strength contribution of FRP depends on a failure mechanism. Khalifa et al. [143] modified Triantafillou’s model to overcome the FRP rupture mode by introducing a reduction factor with the effective strain of the FRP. Triantafillou and Antonopoulos [144] proposed a new model able to predict the effective strain of FRP failed across different modes.

To overcome the deficiency of the previous models, two shear strength models were proposed by Chen and Teng. The first model [145] is proposed for the FRP rupture failure mode while the second model [146] is proposed for the debonding failure mode. The two models were
developed based on a rational interpretation of the failure mechanisms instead of the regression of experimental data used in the previous models [142-144].

In 1985, Roberts [18] investigated the partial interaction of steel-concrete composite beams. Roberts [147, 148] also examined the interface shear stress concentration occurring at the end of externally bonded plates. In 1988, Xia [149] proved that slip will occur between two layers of a multilayered member under elastic impact. In 1990, Al-Ameri and Roberts [17] proposed a new formulation that governed the load-slip relationship for shear stud connectors in steel-concrete composite beams, based on partial interaction behaviour. In their proposed model, the authors incorporated non-linear material properties. The critical stress level on the external reinforcing plate bonded to the beam was estimated by Taljsten [150] to be derived from linear elastic theory. Based on Hart-Smith’s one-dimensional method, Albat and Romilly [151] obtained the adhesive shear stress and the adherent normal stress distribution for uniform thickness double-doubler joints, and double-sided reinforcements. The authors concluded that it was important to use composite adherents with a low shear modulus to correct the shear-lag in the adherent.

Recently studies done by several researcher [10, 152-157], confirms that the finite element method (FEM) is a powerful tool for analysis strain distributions in FRP-strengthened structures. In previous work done by Chen et al. [152, 153, 155], two elements were found to be critically important for the accurate simulation of debonding failures. The first is accurate modelling of the localised cracking behaviour of concrete and the second is accurate modelling of the interfacial bond–slip behaviour between concrete and external FRP reinforcements as well as the bond–slip behaviour between concrete. For the second element, most of the existing FE models did not define the unloading behaviour of the interfaces (between concrete and FRP and/or between concrete) within an FRP-strengthened member.
In 2006, Al-Ameri and Al-Mahaidi [88] proposed a theoretical model to represent the partial interaction behaviour of RC beams retrofitted with multilayer CFRP composites allowing for the interlaminar slip. The results of using elastic material properties showed a satisfactory level of correlation between key predicted parameters and experimental work around the elastic range of loading. Subhani et al. [115] proposed a model to predict the long-term durability of normal and modified epoxy under the influence of wet-dry cycle of the marine environment. In order to predict the bond strength under the marine environment, the reduction factor is introduced.

2.7. Conclusion

In the literature review, concrete structural members externally strengthened and retrofitted with steel plates or FRP composites have been explored. The use of CFRP composites to strengthen concrete structure has been proven to be more efficient than the use of steel plates. Externally strengthening with CFRP composites suffers from premature failure. Several parameters play a role in debonding failure, including brittleness and toughness of the bonding agent.

The epoxy resin is modified with reactive liquid polymer CTBN to overcome the issue. Improving the epoxy toughness will, in turn, improve the flexibility of the bonding agent and, therefore, will improve the overall mechanical behaviour of the strengthened member, such as strength and ductility, and delay premature failure. The use of multilayer CFRP sheets instead of CFRP laminate reduces interfacial stress, particularly at the end of CFRP sheet. The use of multilayer CFRP sheets permits inter-layer slip to occur. Reasonable inter-layer slip will minimise the interfacial stress and improve the overall ductility of the retrofitted structure.
An extensive review was carried out to investigate existing models dealing with the interface of FRP/concrete to mitigate debonding failure.

Based on the literature review and as stated in section 1.3, this research aims to improve the overall beam behaviour (more ductility), and hence delay debonding failure using ATBN and CTBN rubber modified epoxy. Moreover, the mathematical model proposed by Al-Ameri and Al-Mahaidi [88] will be developed to deal with the nonlinear material properties to predict the partial interaction behaviour of such beams.
CHAPTER 3
CFRP BOND PROPERTIES INVESTIGATION

3.1. Introduction

CFRP is currently considered an efficient material for strengthening and rehabilitation of existing and new infrastructure members [4, 14, 66, 158]. The reliability of this material depends on bond integrity [11, 12]. Brittleness and poor toughness characteristics of the epoxy resin are responsible for premature failure in strengthening and rehabilitation due to a high cross-linked density [13, 102]. Toughness is defined as the ability of epoxy to undergo plastic deformation in applied stress states. Greater toughenability can be achieved by reducing the crosslink density of the epoxy resin [103]. Butadiene-acrylonitrile based rubbers are the principle liquid elastomers used for the toughening of epoxies [104, 105]. Among them, CTBN and ATBN are used to modify the neat epoxy by introducing the rubbery phase to form the second phase particles that reduce the crosslink density of neat epoxy [106]. This chapter discusses how modifying MBrace epoxy resin using CTBN and ATBN liquid rubber improves its toughness characteristic by permitting interlayer slip to occur and hence provide more ductility for the retrofitted member. The modified epoxy is used to bond the CFRP sheet to different hosting surfaces such as concrete and steel. The modified bond agent has been tested under the single shear-lap test to examine the improvement of its toughness characteristic. DTMA was conducted to measure the dynamic mechanical properties of the modified epoxy, such as the storage modulus and the glass transition temperature across a range of temperatures.

3.2. Epoxy Resin Modification

The MBrace epoxy system was modified by using CTBN and ATBN as reactive polymer modifiers. The material properties of the neat epoxy and the modifier materials are detailed in the following section.
3.2.1. Epoxy Resin System

The epoxy resin system is widely used as a bonding agent to bond CFRP sheets in strengthening and rehabilitating structures. As per the manufacturer datasheet, the epoxy resin system has better physical and mechanical features than other commercial epoxy resins, such as being lightweight, durable, and having a higher strength to thickness ratio. Moreover, it increases the flexural strength and shear strength and increases the impact resistance of the rehabilitation system. It can be used for different substrates such as concrete, steel and masonry. The most important components of the MBrace system are described below.

Primer

The MBrace Primer is a low-viscosity polyamine cured epoxy consisting of two parts, namely a primer and a hardener, mixed to a ratio of 100:30 by weight. The MBrace Primer is applied to the prepared surface to penetrate the pore structure substrates and to provide a high-bond base coat.

Saturant

The MBrace saturant resin is a low-viscosity epoxy material based on a unique amine curing agent technology. It is used to encapsulate carbon, glass, and aramid fibre fabrics. Similar to the primer, it is a saturant resin and a hardener, mixed to a ratio of 100:30 by weight. The saturant resin consists of more than 60% of bisphenol A and less than 10% of amorphous silica. The hardener for concrete adhesive resins is based on amines consisting of more than 60% of isophoronediamine and less than 30% of benzyl alcohol and is designed to cure at ambient temperatures. When reinforced with carbon fibre fabrics, the saturant cures to provide a high-performance CFRP laminate that provides additional strength to concrete, masonry, steel and timber structural elements.
3.2.2. Reactive Liquid Polymers

Butadiene-acrylonitrile rubber is a liquid polymer frequently used to improve the toughness characteristic of epoxy resin [159]. It contains a relatively low molecular weight backbone of butadiene and acrylonitrile groups with reactive groups in the terminal position (X), as per the chemical formula is shown in Figure 3.1. It can be synthesised with a carboxylic group, such as CTBN, or amine group, such as ATBN (both at the chain ends). When a solution of rubber in epoxy is cured, rubber particles precipitate out as a second phase.

\[
X\left[\left(CH_2-CH=CH-CH_2\right)_n\left(CH_2-CN\right)_m\right]X
\]

Figure 3.1: Molecular formula of butadiene-acrylonitrile rubber [159]

The solubility and glass transition temperature of the reactive polymer are strongly affected by the acrylonitrile content, which varies from zero to 26%. In normal epoxy, to reduce the crosslink density of the epoxy resin and hence improve the toughness characteristic, the Hypro 1300X13 CTBN is used to react with the neat epoxy resin. At the same time, the Hypro 1300X16 ATBN is used to react with the neat epoxy resin and/or with other amine functional compounds [160].

3.2.3. The Modification Process

To investigate the optimum content of CTBN and ATBN needed to produce the best required characteristic of epoxy resin, three mixing ratios of the modifiers were examined as described below.
Using CTBN

Three different samples (20 g, 25 g, and 30 g of 1300x13 CTBN) were added to 100 g of the saturant resin to modify the neat epoxy as shown in Figure 3.2. A low-speed mixer (600 rpm) was used to mix the two parts. The mixing paddle was kept below the surface of the mixture to avoid air entrapment. Proper mixing took, at least, three minutes. The mixture was kept free of streaks or lumps. The mixture was heated in a vacuum furnace at 60˚C for 20 minutes to ensure homogeneity. Then, 30 g of the hardener of the original epoxy was added to the mixture of each modifier ratio and mixed at 600 rpm. The modified epoxy was left at least 24 hours to cure at room temperature.

![Figure 3.2: The process of modified epoxy resin](image)

Using ATBN

Three different mixing weight of ATBN (20 g, 25 g, and 30 g) were added to 30 g of the saturant to identify the ratio for the appropriate combination of ductility and toughness. The compound was mixed using a low-speed mixer (600 rpm) and the mixture was heated in the vacuum furnace for 20 minutes at 60˚C to obtain a homogenous mixture. Then, 100 g of the neat epoxy resin was added to each mixture and mixed at 600 rpm.

3.3. Mechanical Properties of the Modified Epoxies

Ductility is a material property used to assess how materials deform plastically before failure. Toughness is another material property used to determine the ability of a material to absorb the
energy of the plastic deformation before it fails via both strength and ductility [161-163].

Toughness is more useful when comparing the three resins in this application. The material that had high strength and ductility had a higher toughness property. The area under the curve of the stress-strain relationship usually represents the toughness value [161-163]. In the SI system, the unit of tensile toughness can be easily calculated by using area underneath the stress–strain (\(\sigma – \varepsilon\)) curve, which gives tensile toughness value, as given below [164]:

\[
\text{Toughness} = \text{Area underneath the stress–strain (} \sigma – \varepsilon \text{) curve} = \sigma \times \varepsilon = \text{MPa } \times \text{ %} = (\text{N.m}^{-2} \cdot \text{10}^{6}) \cdot (\text{m.m}^{-1} \cdot \text{10}^{-2}) = \text{N.m.m}^{-3} \cdot \text{10}^{4} = \text{J.m}^{-3} \cdot \text{10}^{4}
\]

The shear strength was calculated based on the longitudinal shear force applied on the interface (bond) between the carbon fibre matrix and hosting member. The strain was measured based on the slip occurrence at the interface between the two components using the Instron extensometer device with a 50 mm initial length, as shown in Figure 3.3.

![Figure 3.3: Instron extensometer](image)

The single-lap shear test was performed on the CFRP sheets bonded to steel plates and concrete prisms using the modified resin to investigate the effect of the additives on the mechanical properties of the original epoxy such as strength, ductility and toughness. The Instron 100 kN Universal Testing Machine was used to conduct the single-lap test, with a displacement control of 0.1 mm/min until failed. The bond between the carbon fibre matrix and the hosting member was subjected to a longitudinal shear force.
3.4. Test programme

Three different series of tests were carried out on different epoxies. The first series was CFRP bonded to a steel plate, the second series was CFRP bonded to the concrete prism, and the third series was a DMTA of the modified epoxy.

3.4.1. CFRP Bonded to Steel Plates

Although the research is primarily focused on the bond investigation for RC beams, the tests of the bond were carried out on a steel plate. The reason for this is that concrete is weaker than steel as a bond substrate and so is more likely to fail prematurely. The modified epoxy samples were performed on steel plates because the steel has a higher shear strength compared to concrete and this ensures that any failure will occur in the interface. The best result for ductility and toughness for each modified epoxy was then tested on the concrete prism and DMTA was used to confirm the test results obtained on the steel plates.

Sample Preparation

The surfaces of 21 steel plate samples were prepared using an electrical rotary disc to eliminate a rusted layer, roughen the surface and provide a good quality bond as shown in Figure 3.4. A CFRP sheet with a dimension of 180 mm x 80 mm x 2 mm was bonded to the steel plate and was manufactured by a local company. The mechanical properties of the CFRP sheet are listed in Table 3.1.

![Figure 3.4: Steel plate sample](image-url)
To provide uniform bonding and maintain the alignment direction of the CFRP laminate, a metal clamp was used to attach the CFRP laminate to the metal surface. After, all samples had been cured for 24 hours at room temperature, the single shear-lap test was carried out in the
School of Engineering Lab using a Universal Testing Machine of 3000 KN capacity, as shown in Figure 3.5.

Test Results

The test results of all samples are listed in Table 3.3.

<table>
<thead>
<tr>
<th>S/N</th>
<th>Sample</th>
<th>Max. load, kN</th>
<th>Max. shear stress, MPa</th>
<th>Max. strain, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>NE</td>
<td>84.30</td>
<td>13.17</td>
<td>0.87</td>
</tr>
<tr>
<td>2</td>
<td>NE</td>
<td>84.80</td>
<td>13.25</td>
<td>0.89</td>
</tr>
<tr>
<td>3</td>
<td>NE</td>
<td>84.20</td>
<td>13.16</td>
<td>0.86</td>
</tr>
<tr>
<td>4</td>
<td>20 CTBN</td>
<td>70.10</td>
<td>10.95</td>
<td>0.78</td>
</tr>
<tr>
<td>5</td>
<td>20 CTBN</td>
<td>69.60</td>
<td>10.88</td>
<td>0.77</td>
</tr>
<tr>
<td>6</td>
<td>20 CTBN</td>
<td>69.30</td>
<td>10.83</td>
<td>0.77</td>
</tr>
<tr>
<td>7</td>
<td>25 CTBN</td>
<td>69.60</td>
<td>10.88</td>
<td>0.77</td>
</tr>
<tr>
<td>8</td>
<td>25 CTBN</td>
<td>69.30</td>
<td>10.83</td>
<td>0.77</td>
</tr>
<tr>
<td>9</td>
<td>25 CTBN</td>
<td>70.10</td>
<td>10.95</td>
<td>0.78</td>
</tr>
<tr>
<td>10</td>
<td>30 CTBN</td>
<td>69.10</td>
<td>10.80</td>
<td>1.03</td>
</tr>
<tr>
<td>11</td>
<td>30 CTBN</td>
<td>67.70</td>
<td>10.58</td>
<td>1.02</td>
</tr>
<tr>
<td>12</td>
<td>30 CTBN</td>
<td>67.30</td>
<td>10.52</td>
<td>1.00</td>
</tr>
<tr>
<td>13</td>
<td>20 ATBN</td>
<td>69.20</td>
<td>10.81</td>
<td>2.09</td>
</tr>
<tr>
<td>14</td>
<td>20 ATBN</td>
<td>70.00</td>
<td>10.94</td>
<td>2.12</td>
</tr>
<tr>
<td>15</td>
<td>20 ATBN</td>
<td>68.70</td>
<td>10.73</td>
<td>2.05</td>
</tr>
<tr>
<td>16</td>
<td>25 ATBN</td>
<td>56.10</td>
<td>8.77</td>
<td>2.50</td>
</tr>
<tr>
<td>17</td>
<td>25 ATBN</td>
<td>56.60</td>
<td>8.84</td>
<td>2.53</td>
</tr>
<tr>
<td>18</td>
<td>25 ATBN</td>
<td>55.70</td>
<td>8.70</td>
<td>2.48</td>
</tr>
<tr>
<td>19</td>
<td>30 ATBN</td>
<td>52.10</td>
<td>8.14</td>
<td>1.03</td>
</tr>
<tr>
<td>20</td>
<td>30 ATBN</td>
<td>50.60</td>
<td>7.91</td>
<td>0.99</td>
</tr>
<tr>
<td>21</td>
<td>30 ATBN</td>
<td>50.90</td>
<td>7.95</td>
<td>1.00</td>
</tr>
</tbody>
</table>
Figure 3.6 shows the average stress-strain results of the neat epoxy. The average failure load was 84.4 kN with interface stress of 13.19 MPa. The average percentage of the strain was 0.87% at failure. The toughness recorded was 54.4 kJ.m\(^{-3}\). All of these data were used as a benchmark to assess the ductility and toughness of the modified epoxy resin. The neat epoxy samples were failed by the debonding of CFRP laminate from the steel plate.

![Figure 3.6: Shear stress vs. strain of the neat epoxy](image)

The mechanical properties of the average three samples of CTBN modified epoxy are listed in Table 3.4.

<table>
<thead>
<tr>
<th>CTBN /100 g neat epoxy</th>
<th>Max. load, kN</th>
<th>Max. shear stress, MPa</th>
<th>Max. strain, %</th>
<th>Toughness, kJ.m(^{-3})</th>
<th>Toughness improved/ neat epoxy</th>
</tr>
</thead>
<tbody>
<tr>
<td>20 g</td>
<td>69.7</td>
<td>10.89</td>
<td>0.77</td>
<td>65.3</td>
<td>20 %</td>
</tr>
<tr>
<td>25 g</td>
<td>53.7</td>
<td>8.39</td>
<td>0.84</td>
<td>56.9</td>
<td>4%</td>
</tr>
<tr>
<td>30 g</td>
<td>68.0</td>
<td>10.63</td>
<td>1.02</td>
<td>83.6</td>
<td>54%</td>
</tr>
</tbody>
</table>

In Figure 3.7, the average mechanical properties are plotted and compared with the values of the neat epoxy. Although the shear stress of all samples of CTBN-modified epoxy was decreased when compared with the neat epoxy, the sample of 20 g CTBN-modified epoxy had the higher shear stress (at failure load of 10.89 MPa) when compared with the samples of 30 g
and 25 g (8.39 MPa and 10.63 MPa, respectively). However, the sample of 30 g had larger plastic deformation than the samples of 20 g and 25 g. The sample of 30 g had higher toughness improvement than the samples of 20 g and 25 g when compared with the toughness of the neat epoxy. Therefore, the sample modified by 30 g CTBN-modified epoxy demonstrated better combined mechanical properties than the samples of 20 g and 25 g CTBN-modified epoxy.

![Figure 3.7: Shear stress vs. strain of the CTBN-modified epoxy](image)

The average test results of the three different samples of the ATBN-modified epoxy group are described in Table 3.5 and plotted in Figure 3.8. The sample of 20 g had the higher shear stress (before failure of 10.83 MPa) compared with the other two samples of 25 g and 30 g (8.87 MPa and 8.00 MPa, respectively). The shear stress of all ATBN-modified epoxies was decreased in comparison with the shear stress of the neat epoxy. However, the sample of 25 g had a higher ductility than the other two samples of 20 g and 30 g. The toughness of all ATBN-modified epoxy samples was improved in comparison with the neat epoxy. The sample of 25 g had higher toughness improvement than the other samples of 20 g and 25 g when compared with the toughness of the neat epoxy.
Table 3.5: Mechanical properties of ATBN-modified epoxy

<table>
<thead>
<tr>
<th>ATBN /100 g neat epoxy</th>
<th>Max. load, kN</th>
<th>Max. shear stress, MPa</th>
<th>Max. strain, %</th>
<th>Toughness, kJ.m(^{-3})</th>
<th>Toughness improved/ neat epoxy</th>
</tr>
</thead>
<tbody>
<tr>
<td>20 g</td>
<td>69.3</td>
<td>10.83</td>
<td>2.09</td>
<td>176.4</td>
<td>224 %</td>
</tr>
<tr>
<td>25 g</td>
<td>56.1</td>
<td>8.78</td>
<td>2.50</td>
<td>183.7</td>
<td>238 %</td>
</tr>
<tr>
<td>30 g</td>
<td>51.2</td>
<td>8.00</td>
<td>1.01</td>
<td>63.0</td>
<td>16 %</td>
</tr>
</tbody>
</table>

Therefore, the sample of 25 g exhibits a higher ductility and toughness than the other samples of 20 g and 30 g ATBN-modified epoxy.

Figure 3.8: Shear stress vs. strain of the ATBN-modified epoxy

To compare the mechanical properties of the three groups, the samples of best-combined mechanical properties from each group are listed in Table 3.6 and plotted in Figure 3.9.

Table 3.6: Best combined mechanical properties of the three groups

<table>
<thead>
<tr>
<th>Type of epoxy</th>
<th>Max. load, kN</th>
<th>Max. shear stress, MPa</th>
<th>Max. strain, %</th>
<th>Toughness, kJ.m(^{-3})</th>
<th>Toughness improved/ neat epoxy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Neat epoxy</td>
<td>84.4</td>
<td>13.19</td>
<td>0.78</td>
<td>54.4</td>
<td>---</td>
</tr>
<tr>
<td>30 g CTBN</td>
<td>68.0</td>
<td>10.63</td>
<td>1.02</td>
<td>83.6</td>
<td>54%</td>
</tr>
<tr>
<td>25 g ATBN</td>
<td>56.1</td>
<td>8.78</td>
<td>2.50</td>
<td>183.7</td>
<td>238 %</td>
</tr>
</tbody>
</table>

Although the neat epoxy can sustain a higher load than the other two modified epoxies, the sample of 25 g of ATBN modified epoxy has the highest combination of ductility (strain) and toughness compared to the sample of 30 g CTBN modified epoxy and the neat epoxy [165].
Failure Mode

All tested samples failed through the debonding of the CFRP from the steel plate at various loads and levels of elongation rates. All neat samples had a brittle failure pattern as shown in Figure 3.10a. The CFRP sheet delaminates from the steel surface suddenly without a prior sign of crack initiation. The samples of CTBN-modified epoxy failed in mixed mode between brittle to the ductile pattern. However, the ATBN-modified epoxy exhibited a ductile pattern particular to the sample of 25 g ATBN-modified epoxy, as shown in Figure 3.10b. The ductile crack initiated at the zone of high-stress concentration, then propagated with a shear crack.
toward the CFRP sheet, then delaminated the CFRP sheet from the steel surface. The failure pattern confirmed the toughness properties of the three groups.

3.4.2. CFRP Bonded on Concrete Prism

To confirm results obtained from the CFRP bonded to steel plates test, the neat epoxy, 25 g of ATBN-modified epoxy and 30 g of CTBN-modified epoxy were used to bond CFRP sheets to concrete prisms and tested under the single-lap test setup. A series of nine tests were conducted on CFRP bonded to concrete prisms. The details and design parameters of this series of the test specimens are given in Table 3.7. The concrete prisms were 145 mm x 125 mm x 20 mm and were cast and cured at Deakin Concrete Lab as shown in Figure 3.11.

Table 3.7: Details of test specimens

<table>
<thead>
<tr>
<th>S/N</th>
<th>Epoxy</th>
<th>Weight/mix</th>
<th>Modifier</th>
<th>Weight/mix</th>
<th>Hardener</th>
<th>Weight/mix</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Saturant Part (A)</td>
<td>100 g</td>
<td>-----</td>
<td>-----</td>
<td>Saturant Part (B)</td>
<td>30 g</td>
</tr>
<tr>
<td>2</td>
<td>Saturant Part (A)</td>
<td>100 g</td>
<td>-----</td>
<td>-----</td>
<td>Saturant Part (B)</td>
<td>30 g</td>
</tr>
<tr>
<td>3</td>
<td>Saturant Part (A)</td>
<td>100 g</td>
<td>-----</td>
<td>-----</td>
<td>Saturant Part (B)</td>
<td>30 g</td>
</tr>
<tr>
<td>4</td>
<td>Saturant Part (A)</td>
<td>100 g</td>
<td>CTBN</td>
<td>30 g</td>
<td>Saturant Part (B)</td>
<td>30 g</td>
</tr>
<tr>
<td>5</td>
<td>Saturant Part (A)</td>
<td>100 g</td>
<td>CTBN</td>
<td>30 g</td>
<td>Saturant Part (B)</td>
<td>30 g</td>
</tr>
<tr>
<td>6</td>
<td>Saturant Part (A)</td>
<td>100 g</td>
<td>ATBN</td>
<td>25 g</td>
<td>Saturant Part (B)</td>
<td>30 g</td>
</tr>
<tr>
<td>7</td>
<td>Saturant Part (A)</td>
<td>100 g</td>
<td>ATBN</td>
<td>25 g</td>
<td>Saturant Part (B)</td>
<td>30 g</td>
</tr>
<tr>
<td>8</td>
<td>Saturant Part (A)</td>
<td>100 g</td>
<td>ATBN</td>
<td>25 g</td>
<td>Saturant Part (B)</td>
<td>30 g</td>
</tr>
<tr>
<td>9</td>
<td>Saturant Part (A)</td>
<td>100 g</td>
<td>ATBN</td>
<td>25 g</td>
<td>Saturant Part (B)</td>
<td>30 g</td>
</tr>
</tbody>
</table>

Figure 3.11: CFRP bonded on concrete prism
IMPROVING BOND STRENGTH FOR CFRP-RC BEAMS INTERFACE

Surface Preparation

Surface preparation of the nine concrete prisms was carried out by using a sandblasting machine and water jet. The purpose of the surface preparation is to remove the thin loose layer of cement from the concrete surface. Then, the epoxy primer was applied to the prepared contact area of 80 mm x 75 mm. The specimens were left to cure at room temperature for 24 hours. The carbon fibre sheets were bonded to the concrete prisms using three different types of epoxies. Three samples were prepared from each of neat epoxy, 30 g of CTBN-modified epoxy and 25 g ATBN-modified epoxy. The applied resins cured for 24 hours at room temperature before being tested in the single shear-lap setup, as shown in Figure 3.12.

![Figure 3.12: Concrete Prism setup](image)

<table>
<thead>
<tr>
<th>S/N</th>
<th>Sample</th>
<th>Max. load, kN</th>
<th>Max. shear stress, MPa</th>
<th>Max. strain, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>NE</td>
<td>18.44</td>
<td>2.98</td>
<td>1.69</td>
</tr>
<tr>
<td>2</td>
<td>NE</td>
<td>17.86</td>
<td>2.89</td>
<td>1.72</td>
</tr>
<tr>
<td>3</td>
<td>NE</td>
<td>18.45</td>
<td>2.98</td>
<td>1.74</td>
</tr>
<tr>
<td>4</td>
<td>30 CTBN</td>
<td>16.91</td>
<td>2.81</td>
<td>1.86</td>
</tr>
<tr>
<td>5</td>
<td>30 CTBN</td>
<td>17.12</td>
<td>2.85</td>
<td>1.92</td>
</tr>
<tr>
<td>6</td>
<td>30 CTBN</td>
<td>17.03</td>
<td>2.83</td>
<td>1.82</td>
</tr>
<tr>
<td>7</td>
<td>25 ATBN</td>
<td>15.81</td>
<td>2.62</td>
<td>2.24</td>
</tr>
<tr>
<td>8</td>
<td>25 ATBN</td>
<td>16.00</td>
<td>2.65</td>
<td>2.30</td>
</tr>
<tr>
<td>9</td>
<td>25 ATBN</td>
<td>15.93</td>
<td>2.64</td>
<td>2.25</td>
</tr>
</tbody>
</table>
Test Results

The test results for all samples of this series are listed in Table 3.8. The average stress-strain relationship is shown in Figure 3.13, and the data are given in Table 3.9. The average strength indicated that the addition of CTBN and ATBN modifiers decreased the shear strength, but increased the ductile nature and toughness of modified resins in comparison with the neat resin.

Table 3.9: Best combined mechanical properties of all epoxies

<table>
<thead>
<tr>
<th>Type of epoxy</th>
<th>Max. load, kN</th>
<th>Max. shear stress, MPa</th>
<th>Max. strain, %</th>
<th>Toughness, kJ.m⁻³</th>
<th>Toughness improved/ neat epoxy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Neat epoxy</td>
<td>18.25</td>
<td>2.95</td>
<td>1.72</td>
<td>26.88</td>
<td>---</td>
</tr>
<tr>
<td>30 g CTBN</td>
<td>17.02</td>
<td>2.83</td>
<td>1.87</td>
<td>32.20</td>
<td>20 %</td>
</tr>
<tr>
<td>25 g ATBN</td>
<td>15.91</td>
<td>2.64</td>
<td>2.26</td>
<td>37.53</td>
<td>40 %</td>
</tr>
</tbody>
</table>

Figure 3.13: Shear stress vs. strain relationship of all epoxies

The ATBN-modified resin exhibited more ductility when compared with both the neat epoxy resin and the CTBN-modified resin, by 31.4% and 20.9% respectively. Further, the toughness of the resin modified by ATBN was higher than neat epoxy resin and the CTBN-modified resin, by 39.4 % and 16.5% respectively. However, the shear strength of neat epoxy was greater than the two modified resins by ATBN and CTBN, by 11.7% and 4.3% respectively. The resin modified by CTBN had a higher shear strength than the resin modified by ATBN, by 7.2%.
The gain of toughness will increase the efficiency of the retrofitting system by maintaining additional strength for a longer service life. Structural engineers are in favour of more ductile behaviour and delayed debonding failure for the retrofitted structures.

**Failure Mode**

The failure mode of all samples occurred in the same manner by debonding the CFRP sheet from the concrete surface without any concrete left on the CFRP surface. However, the failure pattern differs from one resin to another. The ATBN-modified resin has a more ductile pattern mode than the CTBN and the neat epoxy, which has a brittle pattern as shown in Figure 3.14. The behaviour of the failure mode confirmed the toughness properties of the three epoxies illustrated in Figure 3.13.

![Crack pattern of brittle failure](image1)

![Crack pattern of ductile failure](image2)

**Figure 3.14: Type of failure mode**

3.4.3. **Dynamic Mechanical Thermal Analysis (DMTA)**

To measure various dynamic mechanical properties of the neat and modified epoxies, such as the storage modulus $E'$ and the glass transition temperature $T_g$, DMTA was performed on the three different epoxies. DMTA measures the material response to a small deformation applied in a cyclic manner.

**Specimen Preparation and Test Setup**

The homogeneous mixtures of the neat epoxy, 25 g ATBN modified epoxy and 30 g CTBN modified epoxy are poured into the steel mould, shown in Figure 3.15, and treated with a
release agent. The air bubbles formed during the mixing is degassed in the vacuum oven for 30 minutes at 60°C. Then the samples cured at room temperature for 24 hours.

Three samples of each resin type, with dimensions of 60 mm x 13 mm x 3 mm shown in Figure 3.16, were tested according to the ASTM-D7028 standard. The TA Q800 DMTA instrument illustrated in Figure 3.17, with dual cantilever mode, was used with the test specifics that listed in Table 3.10. On the DMTA system, all samples mounted on the device as a cantilever beam between the two clamps. The end of the cantilever beam subjected to a sinusoidal force with a constant frequency of 1 Hz. The ramp temperature elevated at the rate of 5°C/min at -20°C to 170°C.

<table>
<thead>
<tr>
<th>Strain</th>
<th>Start temp</th>
<th>Soak time</th>
<th>Final temperature</th>
<th>Ramp rate</th>
<th>Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.01%</td>
<td>-20 °C</td>
<td>5 min</td>
<td>170 °C</td>
<td>5 °C/min</td>
<td>1 Hz</td>
</tr>
</tbody>
</table>
The liquid nitrogen is connected to DMTA setup to test the sample under the required varying temperatures. The operating temperature ranges from -20°C to 170°C. The test is performed after completing the calibration process and fitted the specimen into the cantilever clamp.

The Viscoelastic Measurements result

DMTA is a powerful technique for evaluating the properties of resins through their storage modulus. The glass transition temperatures \( T_g \) and the viscoelastic measurement (the storage modulus \( E' \)) of neat and modified epoxies at room temperature (25°C) were measured and are summarised in Table 3.11.

<table>
<thead>
<tr>
<th>Epoxy</th>
<th>( T_g ) (°C)</th>
<th>( E' ) (MPa) @25°C</th>
</tr>
</thead>
<tbody>
<tr>
<td>MBrace Epoxy (NE)</td>
<td>79.9</td>
<td>2825</td>
</tr>
<tr>
<td>MBrace Epoxy Modified with CTBN</td>
<td>76.8</td>
<td>1750</td>
</tr>
<tr>
<td>MBrace Epoxy with ATBN</td>
<td>73.6</td>
<td>1560</td>
</tr>
</tbody>
</table>

The storage modulus versus temperature for the resins is plotted in Figure 3.18. In general, the curve consists of three phases for all types of resins. The stages are the glassy phase (-20 – ~50 °C), viscoelastic phase (50–79°C), and rubbery phase (79–170°C). In the first stage, where the specimen has a higher range of stiffness due to macromolecular chains, higher storage modulus values were recorded for the neat epoxy, CTBN-modified epoxy, and ATBN-modified epoxy, between 3500–2350 MPa, 1150–2350 MPa, and 2000–1150 MPa respectively. In the second stage, the molecule chain began to move freely, and the storage modulus of the three different resins rapidly decreased to 300 MPa due to the large deformation of the material. At the third stage, the entire molecular chain caused a slippage movement, and the gelatinous flow led to an irreversible deformation. Hence, the modulus rapidly fell to zero [97, 166].
The storage modulus of the neat epoxy at 25°C was greater than that of the epoxies modified with CTBN and ATBN by 60% and 80%, respectively [167]. The presence of the soft rubber particles reduced the intermediate cross-link density of the neat epoxy, and it consequently reduced the modulus of the rigid epoxy thus making the rigid epoxy more flexible. The storage modulus of all epoxies decreased significantly, starting from 45°C, and reached the lowest values at 70°C just before $T_g$ as shown in Figure 3.19.

In contrast, the glass transition temperature of the modified resin slightly decreased compared with the neat epoxy. As described in Table 3.11, the $T_g$ of the CTBN-modified resin decreased by 3°C, and the $T_g$ of the ATBN-modified resin decreased by around 6°C. These results are
expected, since the interaction between rubber liquid particles and epoxy resin tends to reduce in degree of cross-link density in the resin mixture and thus improve its flexibility [168, 169]. The DTMA test affirmed that the modified epoxy becomes softer than the neat epoxy. However, both the storage modulus and the glass transition temperature of the modified epoxy were decreased because of the reduced cross-link density of the neat epoxy. This outcome has confirmed the results obtained by the single-lap tests that were carried out on a steel plate and concrete prism.

3.5. Conclusion

In this chapter, an experimental investigation was conducted on normal epoxy modified by two types of reactive liquid polymers (CTBN and ATBN). This modification was used to improve mechanical properties in terms of ductility and toughness of the final epoxy matrix. This methodology prevents or delays the premature failure (debonding) of the retrofitting application of the CFRP sheets. The results obtained confirm the improvement of ductility and toughness of the modified resin compared with the neat epoxy resin. The main observations obtained are:

1. The results obtained show that the ductility and toughness improved when the neat epoxy was modified by both ATBN and CTBN liquid rubber.
2. The highest combination of ductility and toughness was obtained when the weight mix ratio was 25 g of ATBN-modified epoxy and 30 g of CTBN-modified epoxy.
3. The modified epoxy by 25 g ATBN had more ductility and toughness than the modified epoxy by 30 g CTBN when tested under the single-lap shear test on both steel and concrete hosting surfaces.
4. The failure mode of all the samples tested under the shear-lap test, when the hosting surface is steel or concrete, occurred through the interface between the CFRP sheet and the host surface, confirming a cohesion type of failure.
5. The storage modulus of the neat epoxy decreased up to 80% when modified by ATBN and 60% when modified by CTBN, indicating that the modified epoxy exhibits more softening. In accordance with these results, the 25 g ATBN and 30 g CTBN will be used to bond the CRFP sheets on the RC beams.
CHAPTER 4
TEST OF RC BEAMS RETROFITTED WITH CFRP AND MODIFIED EPOXY

4.1. Introduction

This chapter will describe the test programme for the RC beams retrofitted with CFRP and modified epoxy. Details of test parameters, test procedures, test setup and materials used to fabricate the test specimens are discussed. The fabrication of test specimens, including those strengthened with CFRP and various epoxy resins, are detailed. The strengthened beams were tested under four-point loading up to failure. The test results and observations, including the ultimate load, maximum central deflection, and interface slip for the tested beams, are reported and investigated in Chapter 5.

4.2. Test Variables

The primary aim of this research is to improve the strength and ductility of retrofitted RC beams and hence, to overcome premature failure (debonding). To achieve the research aim, the influence of modified epoxy on the beam behaviour is investigated. Ten RC beams retrofitted with multilayers of CFRP sheets using two types of modified epoxy resins were tested, and the behaviour assessed.

The test programme consisted of nine RC beams and was divided into three series strengthened with varying CFRP layers bonded with different modified epoxy resin. In addition, one RC beam was treated as a control.

After finalising the strengthening process, essential instruments were prepared and installed at the planned positions to record the test data; for example, strain gauges to measure the strain on the concrete surface and the outer CFRP layer. Also, laser optical displacement was used to record the central beam deflection and end interface slip.
All the beams in the last stage were tested under a four-point loading test up to failure. The data obtained included maximum load, maximum deflection, strain variation on the beam surface and strain variation on the outer CFRP layer, as well as the interface slip before the debonding occurred.

### 4.3. Test Programme

Three series of beams are planned to investigate the effect of modified epoxy on the beam behaviour such as the beam ductility, maximum load and interface slip retrofitted with varying number of CFRP layers.

All tested beams were re-coded. However, during the research, the beam labeled upon the number of CFRP layer and type of epoxy used. This code appears to confuse the reader. For that, all the beams were re-coded to simple code from A1 to A10 as shown in the details of the beam series listed in Table 4.1. This code will use onward in the thesis.

<table>
<thead>
<tr>
<th>Series</th>
<th>Current Beam code</th>
<th>Old Beam code</th>
<th>CFRP layers</th>
<th>Type of the epoxy resin</th>
</tr>
</thead>
<tbody>
<tr>
<td>CB</td>
<td>A1</td>
<td>ABCB</td>
<td>****</td>
<td>****</td>
</tr>
<tr>
<td>I</td>
<td>A2</td>
<td>AB1CF25AT</td>
<td>1 layer</td>
<td>ATBN-modified epoxy</td>
</tr>
<tr>
<td></td>
<td>A3</td>
<td>AB1CFNE</td>
<td></td>
<td>Neat epoxy</td>
</tr>
<tr>
<td></td>
<td>A4</td>
<td>AB1CF30CT3</td>
<td></td>
<td>CTBN-modified epoxy</td>
</tr>
<tr>
<td></td>
<td>A5</td>
<td>AB1CF25AT3</td>
<td></td>
<td>ATBN-modified epoxy</td>
</tr>
<tr>
<td>II</td>
<td>A6</td>
<td>AB2CF30CT</td>
<td>2 layers</td>
<td>CTBN-modified epoxy</td>
</tr>
<tr>
<td></td>
<td>A7</td>
<td>AB2CF25AT</td>
<td></td>
<td>ATBN-modified epoxy</td>
</tr>
<tr>
<td>III</td>
<td>A8</td>
<td>AB3CFNE</td>
<td>3 layers</td>
<td>Neat epoxy</td>
</tr>
<tr>
<td></td>
<td>A9</td>
<td>AB3CF30CT</td>
<td></td>
<td>CTBN-modified epoxy</td>
</tr>
<tr>
<td></td>
<td>A10</td>
<td>AB3CF25AT</td>
<td></td>
<td>ATBN-modified epoxy</td>
</tr>
</tbody>
</table>

The control beam, A1, was used as a reference beam to benchmark the test results. The first series consisted of four beams strengthened with one CFRP layer and different epoxy types. The first RC beam of this series, A2 used ATBN-modified epoxy to bond the CFRP sheet. The second beam, A3 used neat epoxy resin (MBrace epoxy) to attach the CFRP sheet. The third beam, A4 used the CTBN-modified epoxy while the remaining A5 used the ATBN-modified epoxy to bond the CFRP sheet.
The second series contained two RC beams retrofitted with two CFRP layers. The first RC beam, A6, used the CTBN-modified epoxy, whereas the second beam A7 used the ATBN-modified epoxy.

The third series of RC beams consisted of three RC beams strengthened with three CFRP layers. The first RC beam A8, used the neat epoxy; the second RC beam A9, used the CTBN-modified epoxy; and the third RC beam A10, used the ATBN-modified epoxy to bond the three CFRP layers.

4.3.1. Testing Frame

The testing frame was model HFT-500, as shown in Figure 4.1, with a maximum operating load of 500 KN. It consists of a loading frame and hydraulic jack control system. The loading frame is fixed to the strong floor at the Concrete Lab to provide a complete loading cycle. The software package supports the digital display of test load, peak value, piston displacement, and specimen deformation. The technical specifications of the test frame are listed in Table 4.2.

<table>
<thead>
<tr>
<th>Table 4.2: Technical specification for HFT-500 machine</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max load</td>
</tr>
<tr>
<td>Load accuracy</td>
</tr>
<tr>
<td>Displacement accuracy</td>
</tr>
<tr>
<td>Load frame dimension</td>
</tr>
</tbody>
</table>

Figure 4.1: The testing frame and supporting arrangement
4.3.2. Test Procedure

A four-point bending test arrangement was used for all the beams. The advantage of the four-point loading over three-point loading is that the latter provides an extended region for the momentum loading (Bending Moment) rather than one specific point, as is recommended for non-homogenous materials such as composites, concrete and wood.

The beam test runs initially under load control and proceeds under deflection control after the elastic point. Up to 30 kN (about one-third of the design load), the beam test ran under the load control at a rate of 0.05 kN/sec. Then, the test ran under the deflection control at a rate of 0.01 mm/sec. The benefit of using load control during the service load is that it limits cracking by imposing the displacement controls on a deflection of the beam once the cracks start. In addition, this arrangement helps to capture the full extension of the beam deflection curve.

The first crack load, load at CFRP deboning and the beam failure load were recorded. Additionally, the strain profiles, interface slip and mode of failure of all tested beams were recorded.

4.4. Fabrication of Test Specimens

4.4.1. RC Beams

Ten RC beams were designed to fail in flexure. The cross-sectional area of the concrete beams was 150 mm in width and 250 mm in depth, as shown in Figure 4.2. The total length of all beams was 2300 mm, while the clear span was 2100 mm, as illustrated in Figure 4.3. The typical beam is designed to sustain a minimum load of 100 kN before failure. The tension reinforcement was provided as three bars of N12 and the shear reinforcement stirrups of N10 at spacing 125 mm. The properties of the specimens taken from steel bars and stirrups were verified with standard tension tests; the average test result of three specimens from each type are reported in Table 4.3. The concrete cover was kept at 15 mm all over. CFRP layers of 1800 mm in length and 100 mm in width were used for all beams.
The beams were cast at the Deakin Concrete Lab from two batches. The formwork was fabricated from hard timber to accommodate five beams at the same time. Figure 4.4a shows steel reinforcement bars assembled and placed in the formwork. The maximum size of the coarse aggregate was 12 mm, and the specific gravity was 2.78. The aggregate was washed with water before use to eliminate the mud and impurity content. The tap water used for mixing the concrete was free from oils, acids, alkalines and other organic impurities. The fresh concrete was poured into the formwork while the vibrator was used to ensure the concrete filled all the gaps and expelled the entrapped air bubbles, as shown in Figure 4.4b. After casting, the
concrete surface was finished using a wet sponge to produce a smooth surface, as shown in Figure 4.4c. Immediately after finishing the casting, the formwork was covered by a wet fabric sheet to protect the concrete moisture from rapid loss and keep the concrete surface free from dust contamination. The next day, the formwork was released and all beams were labelled and covered as shown in Figure 4.4d. The fabric sheet was sprinkled with water every day for 28 days to maintain wet conditions.

From each concrete batch, nine standard cylindrical specimens (200 mm x φ 100 mm) were prepared. The nine standard cylinders were tested after 28 days to obtain the concrete compressive strength, concrete tensile strength, and the concrete elastic modulus (E-value). All the cylinders were labelled and cured in the same environmental conditions as that of the beams.
To achieve bond integrity, a high-pressure water jet machine as shown in Figure 4.5 was used for removing loose particles from the concrete surface where the CFRP was attached. Fine sand of 2−5 mm grain size was used with the water to accelerate the process, as shown in Figure 4.6.

![Figure 4.5: SPITWATER machine](image1)

![Figure 4.6: Preparation of concrete surface](image2)

### 4.4.2. Bonding Materials

The bonding materials used to bond the CFRP sheet to the bottom face of the RC beam are described below.

**MBrace primer:** This is a low-viscosity polyamine cured epoxy. It was applied to the prepared concrete surfaces to penetrate the pore structure of the cementitious substrates and to provide a high-bond base coat for the MBrace system. It is a two-component polyamine cured epoxy, namely, resin (part A) and hardener (part B). Part B was added to part A to achieve a ratio of 30:100 by weight.

**MBrace saturant resin (Neat Epoxy):** This is a low-viscosity epoxy material based on a unique amine curing agent technology. The MBrace saturant resin was used to encapsulate the CFRP sheet. It has two parts, namely, resin (part A) and hardener (part B). The hardener was added to the resin to achieve a ratio of 30:100 by weight. As per the manufacturer’s datasheet, the resin consists of >60% of bisphenol A, and <10% of amorphous silica. Neat epoxy was cured at ambient temperatures for at least 48 hours. When reinforced with carbon fibre fabrics,
the MBrace saturant is cured to provide a high-performance CFRP laminate that provides additional strength to the structural element.

**CTBN-modified Epoxy:** This is a neat epoxy modified with 30 g of CTBN liquid rubber modifier. As discussed in Chapter 3, a sample containing 30 g of CTBN produces the best combination of ductility and toughness. It is prepared by adding 30 g of CTBN to 100 g of the neat epoxy and mixing using a low-speed mixer. Then 30 g of hardener is added, which is then mixed again to obtain a homogenous blend.

**ATBN-modified Epoxy:** This is a neat epoxy modified with 25 g of ATBN liquid rubber modifier. As discussed in Chapter 3, a sample containing 25 g of ATBN produces the best combination of ductility and toughness. It is prepared by adding 25 g to 30 g of the hardener, mixing well, then adding 100 g of the neat epoxy to the mixture. A low-speed mixer is used to obtain a homogeneous blend.

The approximate working time for all types of resins should not exceed 10 minutes at room temperature. Therefore, only the amount needed for one beam was prepared and applied within each working time.

### 4.4.3. Strengthening Procedure

The CFRP sheet (MBrace 230/4900) is a reinforcement material impregnated with the epoxy resin to yield a high-performance composite. It is a unidirectional tow available from the manufacturer in a roll of 300 cm in width and 100 m in length, as shown in Figure 4.7. The CFRP sheet is lightweight, with a high strength to weight ratio. The manufacturer recommends its use for:

- Enhancement of the flexural and shear strengthening of the structural member.
- Axial confinement of columns to increase compressive strength.
The mechanical properties as per the manufacturer’s datasheet are reproduced in Table 4.4.

<table>
<thead>
<tr>
<th>Description</th>
<th>MBrace CFRP 230/4900</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fibre reinforcement</td>
<td>Carbon-high tensile</td>
</tr>
<tr>
<td>Fibre density (minimum)</td>
<td>1.76 g/cm³</td>
</tr>
<tr>
<td>Fibre modulus</td>
<td>230 GPa</td>
</tr>
<tr>
<td>Fibre thickness</td>
<td>0.17 mm</td>
</tr>
<tr>
<td>Ultimate tensile strength</td>
<td>4900 MPa</td>
</tr>
</tbody>
</table>

The CFRP sheets used for strengthening the beams were 1800 mm in length and 100 mm in width. Cutting off the CFRP sheet was carried out using electrical scissors, as shown in Figure 4.8. The same strengthening system was applied to all test beams.

The primer consumed per beam was 250–300 g and to prepare this amount, 60 g of primer hardener was added to 200 g of primer resin. A low-speed mixer mixed the two components for at least three minutes. Immediately after mixing, a thin film from the primer was applied using a small paintbrush, as shown in Figure 4.9a. The primer was left to cure at room temperature for at least 24 hours, as per the manufacturer’s data sheet as shown in Figure 4.9b. Next, the resin was applied to the prepared concrete surface. The type of resin epoxy used was consistent with the test programme parameters. The preparation of neat epoxy, CTBN-modified epoxy and ATBN-modified epoxy was explained in Chapter 3 (section 3.2). Before applying the epoxy resin, all surfaces should be cleaned from dust using pressurised air. The estimated
The amount of the resin consumed for each layer was 300 g. The wet lay-up method was used to apply the CFRP sheets, as shown in Figure 4.10a.

The wet lay-up procedure was started by carefully placing the CF sheet on the wet epoxy resin, then applying the overcoat epoxy resin. The metal roll was used to expel the air bubbles trapped underneath the CF sheet, as shown in Figure 4.10b. This process was repeated when the beam was strengthened with more than one CF sheet. The beam was left to cure at room temperature for at least 24 hours, as shown in Figure 4.10c. At this stage, the RC beams were ready for the tests.
4.5. Instrumentation

The instruments used to monitor and record the beam test results were the load cell, data logger, laser optical displacement measurement and electrical resistance strain gauges. The instrumentation details and locations are shown in Figure 4.11 and explained below:

![Figure 4.11: Instrumentation positions on the tested beam](image)

4.5.1. Laser Optical Displacement Measurement

Two-laser optical displacement measurement models (optoNCDT 1302-100) were used during the testing of beams. The first was used to measure the actual deflection at the beam midspan, as shown in Figure 4.12. The second was used to measure the interface slip between the concrete surface and the CFRP layer at the beam end, as shown in Figures 4.13 and 4.11.

4.5.2. Strain Gauges

Electrical strain gauges were used to record the longitudinal strain of the concrete and CFRP surface along the span. The foil type PL-11-1L strain gauge as shown in Figure 4.14 measured the longitudinal strain along the concrete surface, while the BFLA-5-5 strain gauge as shown in Figure 4.15 measured the longitudinal strain on the corresponding external CFRP layer. The specifications for both strain gauge types are listed in Table 4.5.
Table 4.5: Physical and electrical properties of strain gauges

<table>
<thead>
<tr>
<th>Strain gauge type</th>
<th>Gauge length</th>
<th>Gauge factor</th>
<th>Gauge resistance</th>
<th>Thermal Expansion Coeff.</th>
</tr>
</thead>
<tbody>
<tr>
<td>PL-60-11-1L</td>
<td>60 mm</td>
<td>2.07±1%</td>
<td>120±0.5 Ω</td>
<td>11.8x10^-6/˚C</td>
</tr>
<tr>
<td>BFLA-5-5</td>
<td>5 mm</td>
<td>2.09±1%</td>
<td>120±0.3 Ω</td>
<td>5.0x10^-6/˚C</td>
</tr>
</tbody>
</table>

Figure 4.12: Laser optical displacement at the centre of the beam to measure the absolute central deflection

Figure 4.13: Laser optical displacement measure the relative interface slip at the beam end.

Figure 4.14: PL-60-11-1L strain gauges (SG1)

Figure 4.15: BFLA-5-5 strain gauges (SG2&SG3)

The pre-selected position of the strain gauges on the concrete surface (SG1) and on the corresponding external CFRP layer (SG2) at 350 mm from the support is to measure the differential strain at the region of maximum interface slip. The second strain gauge was located on external CFRP layer (SG3) fixed at 875 mm from the support, in order to measure the longitudinal strain difference between SG2 and SG3 gauges.
4.5.2.1. Installation of Strain Gauges on the Concrete Surface

The Polyester Resin (PS adhesive), Figure 4.16 was used on the concrete surface before installing the strain gauges as a pre-coat, to level the concrete surface in the pre-selected position at 375 mm from the support, as shown in Figure 4.18a. The pre-coat epoxy was left to cure for at least 24 hours. To ensure the strain gauge was working properly before it was fixed, the resistance of the strain gauge was checked using an Avometer device.

![Figure 4.16: The PS adhesive](image1)
![Figure 4.17: CN-E adhesive](image2)

A thin film of the Cyanoacrylate adhesive (CN-E) was then applied, as illustrated in Figure 4.17, and the (PL-60-11-1L) strain gauges were immediately fixed in the correct position with light pressure applied to expel air bubbles, as shown in Figure 4.18b. A thin layer of chemical-proof putty was applied to the strain gauge as in Figure 4.18c to protect it from environmental contamination and chemical reactions.

![Figure 4.18: The strain gauge installation on the concrete surface (SG1)](image3)
4.5.2.2. Installation of Strain Gauges on the CFRP Sheet

Two electrical resistance strain gauges, BFLA-5-5, of length 5 mm were installed on the external surface of the CFRP layer. The two strain gauges were fixed at the preselected positions by applying a thin film of the Cyanoacrylate adhesive (CN-E), as shown in Figure 4.17. The location of the first strain gauge was 350 mm from the support, at the same location as the strain gauges fixed to the concrete surface. The second strain gauge was fixed at 875 mm from the support, as illustrated in Figure 4.11.

4.6. Conclusion

In this chapter, the test variables have been explained, and the test programme has been detailed. The fabrications of test specimens have been described in detail, including fabrication of the RC beams, CF sheets and bonding materials, as well as the strengthening procedure. The specifications of the instrumentation used to record the test data, such as strain gauges and laser optical displacement, have been also described.
CHAPTER 5
TEST RESULTS AND OBSERVATIONS

5.1. Introduction
In this chapter, the results for the tested beams across the three series are reported. The experimental aims were to investigate the influence of modified epoxy on the overall beam behaviour, specifically, the ductility and debonding. As outlined in Chapter 4, all the beams were tested in a four-point bending setup until failure. The load-deflection pattern, strain measurement, interface slip and mode of failure are discussed. The mechanical properties of the concrete of the two batches are reported. The test results show that the beams using the ATBN-modified epoxy exhibit more ductile behaviour than the beams using CTBN-modified epoxy or the neat epoxy. Moreover, the beams retrofitted with more than one CFRP layer failed at an early loading stage, while the beams retrofitted with one CFRP layer failed with debonding of the CFRP from the soffit of the beam.

5.2. Mechanical Properties of the Concrete
Nine standard cylindrical specimens for each batch were tested after 28 days of curing time to estimate concrete compressive strength, concrete tensile strength and concrete modulus of the concrete (E-value), as shown in Figure 5.1. The details and the average of the test results for the two batches are provided in Table 5.1.
Table 5.1: The mechanical properties of the concrete

<table>
<thead>
<tr>
<th>Batch</th>
<th>Cylinder</th>
<th>Compressive strength (MPa)</th>
<th>Tensile strength (MPa)</th>
<th>E-modulus (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>1</td>
<td>41.20</td>
<td>3.24</td>
<td>31.26</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>43.23</td>
<td>3.50</td>
<td>29.82</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>41.25</td>
<td>3.62</td>
<td>31.03</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>41.89</td>
<td>3.45</td>
<td>30.70</td>
</tr>
<tr>
<td>II</td>
<td>1</td>
<td>47.59</td>
<td>3.45</td>
<td>32.07</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>47.44</td>
<td>3.63</td>
<td>32.84</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>45.24</td>
<td>3.55</td>
<td>32.00</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>46.76</td>
<td>3.54</td>
<td>32.30</td>
</tr>
</tbody>
</table>

5.3. Load-Deflection Relationship

The load-deflection relationship of the tested beams was measured at the mid-span, and the load-deflection behaviour was compared to the beams having the same number of CFRP layers and different epoxies. The ductility index was used to compare the ductility improvement of the tested beams. The ductility index decreased with the number of CFRP layers and substantially improved when ATBN-modified epoxy was used.

5.3.1. Control Beam

The deflection of the control beam, A1, was measured using optical laser displacement during the incremental load. The load-deflection relationship is shown in Figure 5.2. The load-deflection curve was linear up to 120 kN.
Then, deflection of the beam was rapidly increased from 12 mm to 25 mm from 120 kN to reach the ultimate load at 127.5 kN due to the yield in reinforcement steel. The load suddenly dropped to 116 kN with increased deflection to 35 mm due to the crushing of the top of the concrete surface. The beam totally failed at a load of 104 kN and 120 mm deflection.

5.3.2. First Series

The first series of beams included four beams, which were strengthened with one CFRP layer, to examine and compare the effect of different types of epoxy on the beam behaviour. The four beams are referred as A2, A3, A4, and A5.

The first beam, A2, used ATBN-modified epoxy to bond the CFRP sheet. The four-point bending test was applied to the beam at each incremental load, and the deflection data were recorded. The load-deflection curve for this beam is shown in Figure 5.3. The reinforcement steel started to yield at a load of 152 kN. The maximum capacity load sustained was 160 kN before the CFRP sheet debonded, followed by the concrete cover delamination at the centre of the beam. The beam’s maximum deflection before debonding was 34 mm. The loading continued until the beam completely failed at 107 kN and 125 mm deflection.

Figure 5.3: Load vs. central deflection curve for beam A2
The second beam, A3, was retrofitted using the MBrace neat epoxy to bond the CFRP sheet on the beam soffit. Load-deflection data was recorded and plotted, as shown in Figure 5.4. The load-deflection curve demonstrated a constant linear relationship up to a load of 133 kN when the reinforcement steel started to yield. The beam deflection at the yield point was 12 mm and reached 24 mm at the maximum load of 165 kN; after this stage, the CFRP was debonded from the concrete surface and the load dropped suddenly to 120 kN. The beam was completely failed after deflection to 138 mm.

![Figure 5.4: Load vs. central deflection curve for beam A3](image)

The third beam in the first series was A4. This beam strengthened with one CFRP sheet bonded to the beam’s tension face using the CTBN-modified epoxy. The four-point bending test was applied to the beam and at each increment load the deflection data were recorded. The load-deflection curve for this beam is shown in Figure 5.5. Consistent with the previous beam, the load-deflection curve gradually rose with a fixed-slope until the reinforcement steel started to yield at 140 kN. The CFRP was debonded from the bottom face at a load of 165 kN and 34 mm central deflection.
In the next stage, the central beam deflection increased with an almost steady load of 127 kN until the beam totally failed at 150 mm central deflection.

![Figure 5.5: Load vs. central deflection curve for beam A4](image)

The fourth beam in this series was A5. It was strengthened with one CFRP layer bonded by ATBN-modified epoxy to the beam soffit. It demonstrated the same strengthening system as A2 but differed in the concrete’s compressive strength. Beam A2 had 41.81 MPa while the beam A5 had 46.76 MPa. The four-point bending test was applied to the beam and at each incremental load the deflection data were recorded. The load-deflection curve for this beam is shown in Figure 5.6. The load-deflection load increased progressively until the load of 140 kN. At this stage, the slope of the load-deflection curve decreased due to yielding of the reinforcement steel. The strengthened beam sustained a maximum load of 168 kN before the debonding occurred, at 40 mm central deflection. After this stage, the beam continued to sustain the load as would a normal beam until it started to fail at load 120 kN and 140 mm central deflection.
5.3.3. Second Series

The second series consisted of two beams: A6, which was strengthened with two layers of CFRP sheets bonded to the bottom face using CTBN-modified epoxy; and A7, which had the same strengthening scheme but used the ATBN-modified epoxy to bond the CFRP sheets to the beam soffit. The four-point bending test was applied to beam A6 and at each incremental load the deflection data were recorded. The load-deflection curve for this beam is shown in Figure 5.7. As a normal trend, the load-deflection rose with a fixed slope until the load reached the reinforcement steel and started to yield at 152 kN. The CFRP layers debonded from the beam at a load of 187 kN. The four-point bending test continued until the beam completely failed at a load of 123 kN and a central deflection of 122 mm.

The second beam, A7, used ATBN-modified epoxy to bond the CFRP layer. It had the same strengthening scheme and loading pattern as beam A6. The load-deflection curve for this beam is shown in Figure 5.8. The linear path of the load-deflection curve continued until the reinforcement steel started to yield at 155 kN. At load of 186 kN, CFRP debonding occurred when the central deflection was 24 mm. The beam completely failed at a deflection of 123 mm and a load of 118 kN.
5.3.4. Third Series

The third series of beams included three RC beams: A8, A9 and A10. This series of beams were strengthened with three CFRP layers on the soffit of the beam. However, the epoxy resin used to bond the CFRP sheet differed for each beam. Beam A8 used the neat epoxy resin to bond the CFRP sheets. Beam A9 used CTBN-modified epoxy and A10 used ATBN-modified epoxy for the same purpose.

The four-point bending test was conducted on beam A8 and at each incremental load deflection data were recorded. The load-deflection curve for this beam is shown in Figure 5.9. The load-
deflection curve showed a linear path until the maximum load was reached at 190 kN and deflection was recorded at 17 mm. At this stage, the CFRP sheets debonded from the RC beam and the load suddenly dropped to 107 kN. After that, the beam acted as the control beam with a load of 130 kN, until the beam completely failed at 122 mm of central deflection.

![Figure 5.9: Load vs. central deflection curve for beam A8](image)

The load-deflection curve for A9 is shown in the Figure 5.10. The ultimate load of the beam reached 175 kN before the CFRP layers debonded with the concrete cover delamination at 21 mm central deflection. The beam continued to sustain a load between 135 and 130 kN until it completely failed at 118 central deflection.

![Figure 5.10: Load vs. central deflection curve for beam A9](image)

The last beam of the series was A10, which used ATBN-modified epoxy to bond the CFRP sheet to the beam tension face. At each incremental load, deflection data were recorded. The
load-deflection curve for this beam is shown in Figure 5.11. The beam had an ultimate load of 177 kN at 21.5 mm central deflection after a fixed increment gradient load. The debonding of CFRP and concrete delamination occurred when the beam reached the ultimate load. The beam completely failed at 125 kN loading and 125 mm deflection.

![Figure 5.11: Load vs. central deflection curve for beam A10](image)

**5.3.5. Discussion**

Nine RC beams retrofitted with different CFRP layers and epoxy resins were tested under a four-point bending setup until failure occurred, to examine the effect of modified epoxy on the overall behaviour of the beams, particularly beam ductility. As noticed from the results of the second and the third series, in all the beams, CFRP is not preserved until the ultimate load is achieved. However, the force in combined CFRP layers cannot be sustained by the concrete, which tends to fail throughout the concrete cover delamination before full load capacity is achieved. Despite the premature failure, around 45% of the maximum load capacity was improved in all the beams.

The ultimate load capacity and the maximum central deflection of the tested beams are listed in Table 5.2.
Table 5.2: Ultimate load capacity and maximum central deflection of beams

<table>
<thead>
<tr>
<th>Series</th>
<th>Beam Reference</th>
<th>Ultimate load kN</th>
<th>Max. central deflection before debonding mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>CB</td>
<td>A1</td>
<td>127</td>
<td>----</td>
</tr>
<tr>
<td>I</td>
<td>A2</td>
<td>160</td>
<td>37</td>
</tr>
<tr>
<td></td>
<td>A3</td>
<td>165</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>A4</td>
<td>165</td>
<td>34</td>
</tr>
<tr>
<td></td>
<td>A5</td>
<td>165</td>
<td>40</td>
</tr>
<tr>
<td>II</td>
<td>A6</td>
<td>187</td>
<td>23.5</td>
</tr>
<tr>
<td></td>
<td>A7</td>
<td>186</td>
<td>24</td>
</tr>
<tr>
<td>III</td>
<td>A8</td>
<td>190</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td>A9</td>
<td>175</td>
<td>21.5</td>
</tr>
<tr>
<td></td>
<td>A10</td>
<td>177</td>
<td>23.5</td>
</tr>
</tbody>
</table>

Beam ductility is a structural property required to permit stress redistribution and provide a warning sign before failure occurs [170, 171]. From the load–deflection curve of the tested beams, the ductility index $\left( \mu_d \right)$ is defined as the ratio of the deflection at ultimate load $\left( \Delta_u \right)$ over the deflection at the yield load $\left( \Delta_y \right)$ [171-173]. Based on this definition, the ductility index compares the deflection of the beams to explain the influence of the modified epoxy resins and CFRP layers on beam ductility.

Figure 5.12: Max load vs. central deflection curve for the first series

The maximum deflection results obtained from the first series before failure occurred are shown in Figure 5.12; the central deflection at the yield load $\left( \Delta_y \right)$ for all the beams in the first
series reached 13 mm. The central deflection at ultimate load ($\Delta_u$) before CFRP debonding was 24 mm for beam A3, 34 mm for A4, 37 mm for A2, and 40 mm for A5. The summary of the ductility index ($\mu_d$) is shown in Table 5.3.

**Table 5.3: The ductility indexes for the test beams in the first series**

<table>
<thead>
<tr>
<th>Beam</th>
<th>$\Delta_y$ mm</th>
<th>$\Delta_u$ mm</th>
<th>$\mu_d$</th>
<th>% $\mu_d$ improvement</th>
</tr>
</thead>
<tbody>
<tr>
<td>A3</td>
<td>13</td>
<td>24</td>
<td>1.85</td>
<td>---</td>
</tr>
<tr>
<td>A2</td>
<td>13</td>
<td>37</td>
<td>2.85</td>
<td>54%</td>
</tr>
<tr>
<td>A4</td>
<td>13</td>
<td>34</td>
<td>2.62</td>
<td>42%</td>
</tr>
<tr>
<td>A5</td>
<td>13</td>
<td>40</td>
<td>3.08</td>
<td>66%</td>
</tr>
</tbody>
</table>

In beam A5, 66% of the ductility was improved compared with beam A3; improvements were 54% for beam A2 and 42% for A4. The ductility value difference between beams A5 and A2 was due to beam A2 failing before A5, despite both beams being retrofitted with the same CFRP layers and using the same modified epoxy.

The maximum central deflection for the second series is shown in Figure 5.13; the ductility values are given in Table 5.4. Although the two beams used different modified epoxy resins, the ductility index had almost the same value since both beams failed at the same load. However, this series did not include a beam that used neat epoxy as a reference to estimate the ductility improvement.
Table 5.4: The ductility index for the second series

<table>
<thead>
<tr>
<th>Beam</th>
<th>$\Delta_y$ mm</th>
<th>$\Delta_u$ mm</th>
<th>$\mu_d$</th>
<th>% $\mu_d$ improvement</th>
</tr>
</thead>
<tbody>
<tr>
<td>A6</td>
<td>13</td>
<td>23.5</td>
<td>1.81</td>
<td>---</td>
</tr>
<tr>
<td>A7</td>
<td>13</td>
<td>24.0</td>
<td>1.85</td>
<td>2.2%</td>
</tr>
</tbody>
</table>

The central deflection for the third series, in which the beams were strengthened with three CFRP layers, is shown in Figure 5.14 and the ductility index is listed in Table 5.5.

![Figure 5.14: Max load vs. central deflection curve for the third series](image)

Despite the difference in deflection values for the beams of the third series, all beams failed early, before the full load capacity was achieved. Beam A10 had a higher deflection (23.5 mm) compared with beam A9 (21.5 mm) and beam A8 (18.0 mm). In A10, ductility improved 31%, and A9 improved 20%.

Table 5.5: The ductility indexes for the test beams in the third series

<table>
<thead>
<tr>
<th>Beam</th>
<th>$\Delta_y$ mm</th>
<th>$\Delta_u$ mm</th>
<th>$\mu_d$</th>
<th>% $\mu_d$ improvement</th>
</tr>
</thead>
<tbody>
<tr>
<td>A8</td>
<td>13</td>
<td>18</td>
<td>1.38</td>
<td>---</td>
</tr>
<tr>
<td>A9</td>
<td>13</td>
<td>21.5</td>
<td>1.65</td>
<td>20%</td>
</tr>
<tr>
<td>A10</td>
<td>13</td>
<td>23.5</td>
<td>1.81</td>
<td>31%</td>
</tr>
</tbody>
</table>

Based on the results across the three series, it can be concluded that both ATBN-modified and CTBN-modified epoxies improve beam ductility. The ATBN-modified epoxy beams have the
highest ductility index of the three series, particularly in the first series. However, in the first series, failure occurred at the interface between the CFRP layer and concrete surface.

5.4. Strain Analysis

To trace the strain profile for each beam during loading, the resulting strain was measured at specific locations by electrical resistance strain gauges. The strain gauge (SG1) was attached to the concrete surface at 350 mm from the right support. Two strain gauges were placed on the outer CFRP layer. The first strain gauge (SG2) was located 350 mm from the support above the concrete strain gauge (SG1) to define the differential strain between the concrete surface and extreme CFRP layer. The second strain gauge (SG3) was fixed at 875 mm on the tension face to explore the changing strain profile through the extreme CFRP layer between SG2 and SG3, as shown in Figure 5.15. The longitudinal strain profile and the differential strain were plotted for each strengthened beam and compared at the load of 140 kN. Before this load, all the beams yielded just before failure occurred.

![Figure 5.15: Positions of the strain gauges](image)

5.4.1. Strain Measurement for the First series

For beam A2, the longitudinal strain at the surfaces of the concrete and CFRP layer was measured by SG1 and SG2, and the differential strain at the same location was plotted in Figures 5.16 and 5.17. The differential strain increased with the load increment. At a load of
140 kN, the differential strain was $0.83 \times 10^{-3}$ mm/mm. This value confirms the occurrence of interface slip and the loss of interaction during loading.

The longitudinal strain on the extreme surface of the CFRP layer at SG1 and SG3 versus the incremental load is shown in Figure 5.18. The contrast between the two strain gauges began at a load of 10 kN and reached a value of $3.80 \times 10^{-3}$ mm/mm at a load of 140 kN before failure occurred. This is a confirmation of the active role of the CFRP sheet in load resisting, and interaction with concrete beams.
Figure 5.18: Longitudinal strain on the outer CFRP layer of beam A2

In the second beam, A3, the longitudinal strain on SG1 and SG2 is shown in Figure 5.19, and Figure 5.20 shows the differential strain at the same location. The differential strain was 2.36x10^{-3} \text{ mm/mm} at a load of 140 \text{ kN}. Also, the longitudinal strain development on the outer surface of the CFRP layer between SG2 and SG3 was 1.80x10^{-3} \text{ mm/mm} for the same load of 140 \text{ kN}, as shown in Figure 5.21.

Figure 5.19: Longitudinal strain on concrete and CFRP of beam A3
In the third beam, A4, the longitudinal strain on the SG1 and SG2 is plotted in Figure 5.22, and Figure 5.23 shows the differential strain at the same location.
The differential strain was $1.99 \times 10^{-3}$ mm/mm at a load of 140 kN. Also, the longitudinal strain development on the outer surface of CFRP layer between SG2 and SG3 was $2.32 \times 10^{-3}$ mm/mm for the same load of 140 kN, as shown in Figure 5.24.

In the fourth beam, A5, the longitudinal strain on SG1 and SG2 is plotted in Figure 5.25, and Figure 5.26 shows the differential strain at the same location. The differential strain was $1.10 \times 10^{-3}$ mm/mm at the load of 140 kN.
Figure 5.25: Longitudinal strain on concrete and CFRP of beam A5

Figure 5.26: Differential strain between SG1 and SG2 of beam A5

Figure 5.27: Longitudinal strain on the outer CFRP layer of beam A5
Longitudinal strain development on the outer surface of the CFRP layer between SG2 and SG3 was recorded as $3.14 \times 10^{-3}$ mm/mm for the same load of 140 kN, as shown in Figure 5.27.

The longitudinal strains at SG1, SG2 and SG3 for the beams of the first series are shown in Figures 5.28, 5.29 and 5.30, respectively. In addition, the differential strain between the concrete surface and the surface of outer CFRP layer (SG2 and SG1) is shown in Figure 5.31.

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**Figure 5.28: Longitudinal strain at SG1 location for the first series**

**Figure 5.29: Longitudinal strain at SG2 location for the first series**
5.4.2. Strain Measurement for the Second Series

In the first beam of the second series, A6, the longitudinal strain on SG1 and SG2 is plotted in Figure 5.32. Figure 5.33 shows the differential strain at the same location. The differential strain was 2.41X10^{-3} mm/mm at the load of 140 kN.
The longitudinal strain development on the outer surface of CFRP layer between SG2 and SG3 was recorded as $0.86 \times 10^{-3}$ mm/mm for the same load of 140 kN, as shown in Figure 5.34.

In the second beam, A7, the longitudinal strain on SG1 and SG2 is plotted in Figure 5.35. Figure 5.36 shows the differential strain at the same location. The differential strain was $1.56 \times 10^{-3}$ mm/mm at the load of 140 kN.
Figure 5.34: Longitudinal strain on the outer CFRP layer of beam A6

Figure 5.35: Longitudinal strain on concrete and CFRP of beam A7

Figure 5.36: Differential strain between SG1 and SG2 of beam A7
The longitudinal strain development on the outer surface of CFRP layer between SG2 and SG3 was recorded as $0.80 \times 10^{-3}$ mm/mm for the same load of 140 kN, as shown in Figure 5.37.

![Figure 5.37: Longitudinal strain on the outer CFRP layer of beam A7](image)

The longitudinal strains at SG1, SG2 and SG3 for the beams of the second series are shown in Figures 5.38, 5.39 and 5.40, respectively. In addition, the differential strain between the concrete surface and the surface of outer CFRP layer (SG2-SG1) is shown in Figure 5.41.

![Figure 5.38: Longitudinal strain at SG1 location for the second series](image)
Figure 5.39: Longitudinal strain at SG2 location for the second series

Figure 5.40: Longitudinal strain at SG3 location for the second series

Figure 5.41: Differential strain (SG2-SG1) for the second series
5.4.3. Strain Measurement for the Third Series

The longitudinal strain of the first beam in the third series, A8, measured on SG1 and SG2 is plotted in Figure 5.42. The differential strain measurement at the same location was recorded as $0.42 \times 10^{-3}$ mm/mm at the load of 140 kN, as shown in Figure 5.43.

![Figure 5.42: Longitudinal strain on concrete and CFRP of beam A8](image)

![Figure 5.43: Differential Strain between SG1 and SG2 of beam A8](image)

The longitudinal strain development on the outer surface of CFRP layer between SG2 and SG3 was recorded as $1.18 \times 10^{-3}$ mm/mm for the same load of 140 kN, as shown in Figure 5.44.
Figure 5.44: Longitudinal Strain on the outer CFRP layer of beam A8

In beam A9, the longitudinal strain measured on SG1 and SG2 is plotted in Figure 5.45. Differential strain measurement at the same locations was recorded as $1.16 \times 10^{-3}$ mm/mm at a load of 140 kN, as shown in Figure 5.46.

Figure 5.45: Longitudinal strain on concrete and CFRP of beam A9

Figure 5.46: Differential strain between SG1 and SG2 of beam A9
The longitudinal strain development on the outer surface of the CFRP layer between SG2 and SG3 was 2.50x10^{-3} mm/mm for the same load of 140 kN, as shown in Figure 5.47.

The strain measurement in the last beam, A10, on SG1 and SG2 is plotted in Figure 5.48. The differential strain measurement at the same location was recorded at 1.30x10^{-3} mm/mm at a load of 140 kN, as shown in Figure 5.49.
The longitudinal strain development on the outer surface of CFRP layer between SG2 and SG3 locations recorded $1.20 \times 10^{-3}$ mm/mm for the same load of 140 kN and shown in Figure 5.50.

The longitudinal strain at SG1, SG2 and SG3 locations for the beams of the third series are shown in Figures 5.51, 5.52 and 5.53, respectively. In addition, the differential strain between the concrete surface and the surface of outer CFRP layer (SG2-SG1) is shown in Figure 5.54.
Figure 5.51: Longitudinal strain at SG1 location for the third series

Figure 5.52: Longitudinal strain at SG2 location for the third series

Figure 5.53: Longitudinal strain at SG3 location for the third series
5.4.4. Summary of Strain Measurement

The overall strain measurement was summarised in Table 5.6.

Table 5.6: Strain measurement in the three series

<table>
<thead>
<tr>
<th>Series</th>
<th>Beam reference</th>
<th>Differential strain (SG1-SG2), x10⁻³ mm/mm</th>
<th>Strain difference (SG2-SG3), x10⁻³ mm/mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>A2</td>
<td>0.83</td>
<td>3.80</td>
</tr>
<tr>
<td></td>
<td>A3</td>
<td>2.36</td>
<td>1.80</td>
</tr>
<tr>
<td></td>
<td>A4</td>
<td>1.99</td>
<td>2.32</td>
</tr>
<tr>
<td></td>
<td>A5</td>
<td>1.10</td>
<td>3.14</td>
</tr>
<tr>
<td>II</td>
<td>A6</td>
<td>2.41</td>
<td>0.86</td>
</tr>
<tr>
<td></td>
<td>A7</td>
<td>1.56</td>
<td>0.80</td>
</tr>
<tr>
<td>III</td>
<td>A8</td>
<td>0.42</td>
<td>1.18</td>
</tr>
<tr>
<td></td>
<td>A9</td>
<td>1.16</td>
<td>2.50</td>
</tr>
<tr>
<td></td>
<td>A10</td>
<td>1.30</td>
<td>1.20</td>
</tr>
</tbody>
</table>

The differential strain between the two strain gauges (SG1 and SG2) represents the value of discontinuity strain as a result of the partial slip between the CFRP layers and concrete. However, the lower differential strain has more flexibility and healthy bonds permit the strain to transfer from the CFRP to the concrete. The difference in strain between SG2 and SG3 represents the integrity of the bond, with the strain increasing towards the centre of the beam.

As seen in Table 5, for each type of epoxy resin the differential strain between SG1 and SG2 increases with the number of CFRP layers. However, the difference in strain between SG2 and...
SG3 decreases with the number of CFRP layers, due to the crack initiated early in the beams which strengthened those with more CFRP layers.

The beams using used ATBN-modified epoxy showed less discontinuity of strain between SG1 and SG2 across all CFRP layers applied. The ATBN-modified epoxy had a healthier bond than the other two epoxy resins.

5.5. Interface Slip Measuring

The relative interface slip between the concrete surface and the CFRP matrix for all the strengthened beams was measured and is summarised in Table 5.7. The laser optical displacement took this measurement at the cut-end of CFRP sheet near the support of the RC beam as in Figure 5.55, where maximum interface slip occurs. The relative interface slip was also recorded when the applied load increased and reached the maximum value before the beam failed.

![Figure 5.55: Position of optical laser displacement to measure the slip](image)

Table 5.7 shows the values of maximum interface slip for all strengthened beams and the interface slip at 140 kN.
Table 5.7: The interfaces slip values for test beams

<table>
<thead>
<tr>
<th>Series</th>
<th>Beam</th>
<th>Epoxy</th>
<th>CFRP Layer</th>
<th>Interface slip, mm @140 kN</th>
<th>Max Interface slip, mm</th>
<th>Slip/Slip NE @140 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>A3</td>
<td>Neat Epoxy</td>
<td>1</td>
<td>0.14</td>
<td>0.22 @ 154 kN</td>
<td>----</td>
</tr>
<tr>
<td></td>
<td>A2</td>
<td>ATBN-Modified</td>
<td>1</td>
<td>0.36</td>
<td>0.55 @ 157 kN</td>
<td>2.57</td>
</tr>
<tr>
<td></td>
<td>A4</td>
<td>CTBN-Modified</td>
<td>1</td>
<td>0.08</td>
<td>0.27 @ 149 kN</td>
<td>0.57</td>
</tr>
<tr>
<td></td>
<td>A5</td>
<td>ATBN-Modified</td>
<td>1</td>
<td>0.26</td>
<td>0.30 @ 149 kN</td>
<td>1.86</td>
</tr>
<tr>
<td>II</td>
<td>A6</td>
<td>CTBN-Modified</td>
<td>2</td>
<td>0.53</td>
<td>0.55 @ 145 kN</td>
<td>3.76</td>
</tr>
<tr>
<td></td>
<td>A7</td>
<td>ATBN-Modified</td>
<td>2</td>
<td>0.32</td>
<td>0.43 @ 168 kN</td>
<td>2.29</td>
</tr>
<tr>
<td>III</td>
<td>A8</td>
<td>Neat Epoxy</td>
<td>3</td>
<td>0.70</td>
<td>0.80 @ 174 kN</td>
<td>5.00</td>
</tr>
<tr>
<td></td>
<td>A9</td>
<td>CTBN-Modified</td>
<td>3</td>
<td>0.58</td>
<td>1.01 @ 166 kN</td>
<td>4.14</td>
</tr>
<tr>
<td></td>
<td>A10</td>
<td>ATBN-Modified</td>
<td>3</td>
<td>0.64</td>
<td>0.88 @ 166 kN</td>
<td>4.57</td>
</tr>
</tbody>
</table>

The values of interface slip were plotted against the applied load, as shown in Figure 5.56. The influence of varying the CFRP layers and modified epoxy resins on the interface slip is detailed in Figure 5.56.

Figure 5.56: Interface slip for all strengthened beams
5.5.1. Influence of the Number of CFRP Layers

The interface slip values for the first series of beams (strengthened with one CFRP layer) are shown in Figure 5.57. The value of interface slip at 140 kN was 0.14 mm for beam A3, which used the neat epoxy resin. Beam A4, which used CTBN-modified epoxy, measured 0.08 mm interface slip at 140 kN. Beams A2 and A5, which used ATBN-modified epoxy were recorded as 0.26 mm and 0.36 mm respectively, at a load of 140 kN.

Figure 5.57: Interface slip for the beams strengthened with one CFRP layer

![Figure 5.57: Interface slip for the beams strengthened with one CFRP layer](image)

Figure 5.58: Interface slip for the beams strengthened with two CFRP layers

![Figure 5.58: Interface slip for the beams strengthened with two CFRP layers](image)
The interface slip values for the second series of beams (strengthened with two CFRP layers) are shown in Figure 5.58. At a load of 140 kN, the interface slip was 0.53 mm for beam A6 and 0.32 mm for beam A7.

The interface slip values for the third series of beams (strengthened with three CFRP layers) are shown in Figure 5.59; at a load of 140 kN, this value equalled 0.70 mm for beam A8, 0.58 mm for beam A9 and 0.64 mm for beam A10.

![Figure 5.59: Interface slip for the beams strengthened with three CFRP layers](image)

**5.5.2. Influence of the Modified Epoxy Resin**

The interface slip values for beams using neat epoxy to bond the CFRP sheet are shown in Figure 5.60. The value of the interface slip at a load of 140 kN was 0.14 mm for beam A3 and 0.70 mm for beam A8.

The interface slip values for beams that used CTBN-modified epoxy are shown in Figure 5.61. At a load of 140 kN, the values were 0.08 mm for beam A4, 0.53 mm for beam A6 and 0.58 mm for beam A9.
The interface slip values of beams using ATBN-modified epoxy are shown in Figure 5.62.
The interface slip values at the load of 140 kN are 0.36 mm for beam A2, 0.26 mm for beam A5, 0.32 for beam A7 and 0.64 mm for beam A10.
5.5.3. Conclusion

In relation to the test beams’ interface slip, it can be concluded that the use of the rubber modified epoxy allowed for more interface slip to occur than did the neat epoxy, and hence delayed debonding failure and increased failure load. Additionally, there was no sign of debonding failure if the interface slip in continues forms with incremental load. Failure occurred when the interface slip value rapidly increased [14].

5.6. Failure Modes

Three failure modes were observed among the beam tests. The first failure mode was debonding of the CFRP sheets from the extreme concrete surface, which occurred for three beams of the first series. The concrete cover delamination failure was observed in all beams of the second and third series. The third failure mode was CFRP debonding accompanying concrete cover delamination. This mode was observed for the first beam, A2. All the failure modes and their associated beams are listed in Table 5.8. The concrete cover delamination
and/or CFRP debonding failure is assumed to occur when the force in the CFRP composite cannot be sustained by the concrete substrate [62].

### 5.6.1. CFRP Debonding Failure Mode

The debonding of the CFRP sheet from the soffit beam was the predominant failure for three beams of the first series, namely A3, A4 and A5. These beams were strengthened with one CFRP layer.

The CFRP sheet was debonded from beam A3’s soffit when the maximum sustained load reached 165 kN and the deflection at the beam centre was recorded at 24 mm, as shown in Figure 5.63. The CFRP debonding mode occurred in beam A4 when the load reached 165 kN and 34 mm of central deflection, as illustrated in Figure 5.64. In the last beam, A5, CFRP debonding occurred at a load of 165 kN and a 40 mm central deflection as shown in Figure 5.65.

![Figure 5.63: Failure mode of beam A3](image)

<table>
<thead>
<tr>
<th>Series</th>
<th>Beam</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>A2</td>
<td>Concrete cover delamination + CFRP debonding</td>
</tr>
<tr>
<td></td>
<td>A3</td>
<td>CFRP debonding CFRP</td>
</tr>
<tr>
<td></td>
<td>A4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>A5</td>
<td></td>
</tr>
<tr>
<td>II</td>
<td>A6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>A7</td>
<td></td>
</tr>
<tr>
<td>III</td>
<td>A8</td>
<td>Concrete cover delamination</td>
</tr>
<tr>
<td></td>
<td>A9</td>
<td></td>
</tr>
<tr>
<td></td>
<td>A10</td>
<td></td>
</tr>
</tbody>
</table>
5.6.2. Concrete Cover Delamination Failure Mode

The concrete cover delamination failure was predominant among all beams in the second and third series.

In the second series, in which the beams were strengthened with two CFRP layers, the concrete cover delamination failure mode followed the same trend for two beams. These two beams failed at almost the same ultimate load and almost the same deflection. Beam A6 failed at a load of 187 kN through the longitudinal path of the steel reinforcement and 23.5 mm deflection, as shown in Figure 5.66. Beam A7 failed at a load of 186 kN and 24 mm central deflection, as shown in Figure 5.67.
Concrete cover delamination also occurred in all the beams of the third series. Beam A8 failed on reaching the ultimate load of 190 kN and 18 mm central deflection, as shown in Figure 5.68. Both A9 and A10 beams failed at almost the same ultimate load (177 kN and 175 kN, respectively) and central deflection (21.5 mm and 23.5 mm, respectively), as shown in Figures 5.69 and 5.70.
5.6.3. CFRP Debonding / Concrete Cover Delamination Failure Mode

The combination of debonding of CFRP and concrete cover delamination failure mode occurred in beam A2 from the first series, which was strengthened with one CFRP layer, as shown in Figure 5.71. The failure occurred at a load of 160 kN and 37 mm deflection, and started with the concrete cover delamination at the edge of the CFRP layer, followed by CFRP debonding.
5.6.4. Failure Mode Discussion

The CFRP debonding failure mode was the first observed failure mode that occurred in beams A3, A4, and A5 from the first series. It was initiated by flexure and/or shear crack and propagated along the adhesive layer that tends to debond the CFRP sheet from the concrete surface, as shown in Figures 5.63 to 5.65.

The concrete cover delamination failure mode occurred for all beams of the second and third series, which were strengthened with two and three CFRP layers respectively. Concrete cover delamination takes place when more than one CFRP layer is bonded to the (relatively thick) concrete surface [174, 175]. This failure mode is induced by the high concentration of stresses at the end of the CFRP sheet, and was initiated by a shear crack at the end of the CFRP laminate, as shown in Figures 5.66 to 5.70. With a further load, the crack propagated along the level of the steel reinforcement that takes off the concrete substrate away and exposed the steel reinforcement.

The third failure mode was a combination of concrete cover delamination and debonding of CFRP sheet, occurring in beam A2 from the first series. This failure commenced with concrete cover delamination at the maximum bending zone, as shown in Figure 5.71. It is induced by the intermediate flexure crack that tends to split the concrete at the level of steel reinforcement that peels away the CFRP sheet [174].

The difference in the failure modes of beams A2 and A5 (both from the first series), which were both strengthened with one CFRP layer using the same ATBN-modified epoxy, was due to a difference in concrete compressive strength. The average compressive strength of beam A2 from the first batch (41.89 MPa) was less than the average compressive strength of beam A5 from the second batch (46.76 MPa). The variation in the concrete compressive strength in the two beams explains the difference in failure mode due to the differing elastic modulus of concrete [176] and may due to different in the transfer of shear stresses from the FRP to the
concrete [177]. The compressive strength of concrete is a primary factor in structural applications needed to estimate the elastic modulus of concrete, which is one of the most commonly used parameters to describe a material property. Designers rely on approximations using the concrete compressive strength to estimate these properties for their designs [176].

5.7. Conclusion

Ten RC beams tested were under a four-point bending setup until failure, to investigate the influence of modified epoxy on beam behaviour. The ductility of beams using modified epoxy resin was improved up to 65% compared with beams using neat epoxy to bond the CFRP sheets. The discontinuity strain value of the test beams was lowest among those using ATBN-modified epoxy, indicating healthier bonds than in the beams using the other two epoxies. Moreover, beams using modified epoxy allowed more interface slip than those using the neat epoxy, and hence delayed the bonding failure.

Three modes of failure were observed in the test beams. The first mode was CFRP debonding, which occurred for three beams from the first series. The fourth beam of the first series failed with a combination of debonding of CFRP and concrete cover delamination. All test beams of the second and third series failed with concrete cover delamination.
CHAPTER 6
MATHEMATICAL MODEL

6.1. Introduction

In this chapter, a reliable mathematical model is used to predict the general behaviour of a retrofitted RC beams with a multilayered CFRP matrix that allows for inter-layer slip and non-linear material properties. Predictions are verified against previous and current experimental results and include central deflection, end slip and strain profile. The assumed element of the composite beam was subject to a system of forces that satisfies equilibrium of forces and compatibility of deformations. The inter-layer slip was incorporated by relating differential strain at the interface of the CFRP layers and concrete to the longitudinal shear flow at the corresponding interface through the shear stiffness of the adhesive layer.

In this research, the new contribution has been done to develop the elastic model presented by Al-Ameri and Al-Mahaidi [88] to allow for non-linear material properties that will enable the prediction of the general behaviour of retrofitted beam up to failure. The new contribution is redefined the stress-strain relationship for concrete in the composite section and redefine the shear stiffness of the bonding agent.

In the elastic model presented by Al-Ameri and Al-Mahaidi [88] the secant values of the material property of the concrete \((E_c)\) is constant and obtained from the stress-strain realtionship of the concrete section. Morover, the shear stiffness of the bonding agent \((K_s)\) is also taken as a constant value and obtained from the load-slip realtionship of the bonding agent. However, in the developed model in this research, the ultimate compresive strain of the concrete is limited to 0.0035 and the material property of the concrete \((E_c)\) is taken as a function of strain. This can be achieved by dividing the cross-sectional area of the concrete element into a number of elemental strips and take the summation of strain in the elemental strips in order to obtain the the non-linear material property of the concrete \((E_c)\). Moreover, the shear stiffness
(K_s) of the bonding agent is defined mathematically as the secant value for the load-slip curve. The shear load (P) for the bonding agent obtained from the curve fitting equations of the load-slip of single shear tests carried out previously on the concrete prisms. Then the shear stiffness (K_s) obtained by deriving the uniform shear flow with respect to the interface slip. The new contribution of the redefine of the material property of the concrete (E_c) and redefine the shear stiffness (K_s) of the bonding agent are detailied in sections (6.3), (6.4), (6.5) and (6.7).

6.2. Proposed Model

The mathematical model adapted non-linear material properties, including epoxy resin. The non-linear differential equations expressed in a finite different form and solved iteratively using purpose built software. An element of length (δ_x) from the strengthened beam has been limited with (3) layers of CFRP. The reason for using (3) layers of CFRP instead of (n) layers, in the original model presented by Al-Ameri and Al-Mahaidi [88] is to verify the model prediction with the test beams’ results in this research that retrofitted up to three layers of CFRP sheets.

6.2.1. Assumption

The theoretical model in this research is based on the following assumptions:

- The composite section is subject to a uniformly distributed load along the beam length.
- The CFRP has a linear elastic property until failure in both tension and compression.
- The concrete and the shear stiffness of the adhesive material per unit length of the interface between two adjacent layers are assumed to be a non-linear function of strain.
- Plane sections of each material remain plane after bending.
- CFRP layer components have the ability to sustain axial load only, and the bending stiffness of CFRP layers are neglected (non-flexural).
- A friction effect between layers is ignored.
- The concrete and CFRP layers have the same amount of deflection at any point of the composite section and no separation considering between two adjacent layers of interface.
- The strain induced by construction sequence and shrinkage in the concrete layer is neglected.

![Composite element diagram](image)

**6.2.2. Formulation**

The developed model of the composite element is shown in Figure 6.1. It consists of a layer of concrete with a length of \( (\delta_x) \) and (3) layers of CFRP attached to its tension face by a bonding layer of negligible thickness. The bonding layer has a shear stiffness per unit length of \( (K_s) \). It is subjected to bending moment \( (M_c) \), shear force \( (V_c) \) and axial forces \((F_1), (F_2), (F_3)\). The subscript \((c)\) indicates the concrete, and the subscripts \((1, 2, 3)\) refer to the CFRP layers.
The centroid of the concrete layer is considered as the point of origin of the \((x - z)\) coordinate system of the composite element.

### 6.2.2.1. Displacement, Strain and Stress Resultants

According to the assumption that the plane section remains plane after bending, the axial strain \((\varepsilon)\) can be defined in terms of displacement relative to \((x)\) passed through the centroid of the concrete element.

In the concrete composite element, the axial strain \((\varepsilon_c)\) is expressed as in the following equation in terms of longitudinal displacement \((U_c)\) and vertical displacement \((W_c)\), in addition to the strain induced by construction sequence \((\varepsilon_{cc})\) and shrinkage \((\varepsilon_{cf})\) of the concrete.

\[
\varepsilon_c = U_{c,x} - Z_c W_{c,xx} + \varepsilon_{cc} - \varepsilon_{cf} \quad \text{................................................................. (6.1)}
\]

In which the term \((Z_c)\) is the distance from the concrete surface to the origin of axes, and subscript \((x)\) denotes the differentiation with respect to \((x)\).

The assumption that of the concrete and CFRP layers have the same deflections at any point will lead to \((W_c = W_1 = W_2 = W_3 = W)\). Moreover, \((\varepsilon_{cc})\) and \((\varepsilon_{cf})\) is neglected according to the assumption that the strain is induced by construction sequence and shrinkage in the concrete layer. Therefore, equation 6.1 can be rewritten as the following:

\[
\varepsilon_c = U_{c,x} - Z_c W_{xx} \quad \text{................................................................. (6.2)}
\]

The curvature of the CFRP layers is supposed to have no influence on the horizontal displacement of each layer due to its small thickness compared with the concrete element. Hence, the horizontal displacement of each CFRP layer is assumed constant \((W_{xx} = 0)\) across the depth of the layer. Therefore, the strain of each layer of CFRP is directly proportional to the axial displacement \((U)\) in that layer as follows:

\[
\varepsilon_1 = U_{1,x} \quad \text{................................................................. (6.3)}
\]
\[
\varepsilon_2 = U_{2,x} \quad \text{................................................................. (6.4)}
\]
\[ \varepsilon_3 = U_{3,x} \]  
(6.5)  

Now, the stresses \( (\sigma) \) can be obtained from the relationship to the strain through the material property \( (E) \) of concrete and CFRP.

\[ \sigma = E. U_x \]  
(6.6)  

The elastic modulus of CFRP layers \( (E_{(1,2,3)}) \) is considered constant at any point of load stage due to the material behaviour of CFRP. Moreover, the elastic modulus for concrete \( (E_c) \) is also considered to have a constant stress-strain relationship in this stage. Hence, the stresses in the concrete and the CFRP layer can be defined as:

\[ \sigma_c = E_c(U_{c,x} - Z_c W_{xx}) \]  
(6.7)  

\[ \sigma_1 = E_1.U_{1,x} \]  
(6.8)  

\[ \sigma_2 = E_2.U_{2,x} \]  
(6.9)  

\[ \sigma_3 = E_3.U_{3,x} \]  
(6.10)  

Therefore, the axial forces can now be obtained by integrating the stresses over the cross-sectional area of the concrete and CFRP as follows:

\[ F_c = \int \sigma_c.dA_c \]  
(6.11)  

\[ F_1 = \int \sigma_1.dA_1 \]  
(6.12)  

\[ F_2 = \int \sigma_2.dA_2 \]  
(6.13)  

\[ F_3 = \int \sigma_3.dA_3 \]  
(6.14)  

Substituting the stress values from equations (6.7 to 6.10) into the force equations (6.11 to 6.14) will give:

\[ F_c = \int E_c(U_{c,x} - Z_c W_{xx}).dA_c \]  
(6.15)  

\[ F_1 = \int E_1.U_{1,x}.dA_1 \]  
(6.16)  

\[ F_2 = \int E_2.U_{2,x}.dA_2 \]  
(6.17)  

\[ F_3 = \int E_3.U_{3,x}.dA_3 \]  
(6.18)
where the moment of the concrete section is:

\[ M_c = -\int \sigma_c \cdot dA_c \]  \hspace{1cm} (6.19)

substituting the value of \( \sigma_c \) from equation (6.7) into the moment equation above, gives

\[ M_c = -\int E_c (U_{c,x} - Z_c W_{xx}) \cdot dA_c \]  \hspace{1cm} (6.20)

**6.2.2.2. Equilibrium and Compatibility Equations**

The strain has been expressed in terms of five independent displacement variables, which are \( (W, U_c, U_1, U_2 \text{ and } U_3) \). Therefore, five equations are required to obtain a solution of these five independents. These equations can be formulated by considering the equilibrium for the forces and the compatibility of deformation at the interface between the two adjacent materials.

The vertical equilibrium in the z-direction of the composite element of length \( (\delta x) \) and subjected to uniformly distributed load \( (\rho) \) will lead to the following equations, noting that \( (\rho) \) is the total applied load including the self weight:

\[ \rho = \frac{\delta V_c}{\delta x} = V_{c,x} \]  \hspace{1cm} (6.21)

Taking the moment of the entire composite section forces is about the origin of coordinates at the concrete element, gives

\[ M_{c,x} = V_c + F_{1,x} d_1 + F_{2,x}(d_1 + d_2) + F_{3,x}(d_1 + d_2 + d_3) \]  \hspace{1cm} (6.22)

In which \( (d_1) \) denotes the distance between centroid of the concrete element and centroid of first CFRP layer, \( (d_2) \) is the distance between centroids of the first and second CFRP layers and \( (d_3) \) is the distance between centroids of the second and third CFRP layers as shown in Figure 6.2.
Differentiating equation (6.22) with respect to $(x)$ and replacing $(V_{c,x})$ with $(\rho)$ gives:

$$M_{c,xx} - F_{1,xx}d_1 - F_{2,xx}(d_1 + d_2) - F_{3,xx}(d_1 + d_2 + d_3) = \rho \quad \text{(6.23)}$$

Equation (6.23) is the first equilibrium equation required for the basic solution.

Considering equilibrium of the composite element in the $x$-direction, gives:

$$\delta F_c + \delta F_1 + \delta F_2 + \delta F_3 = 0 \quad \text{................................. (6.24)}$$

Dividing equation (6.24) by $(\delta x)$, gives the second equilibrium equation required for the basic solution, as follows:

$$F_{c,x} + F_{1,x} + F_{2,x} + F_{3,x} = 0 \quad \text{................................. (6.25)}$$

Assuming that there is no separation (i.e., vertical differential deflection) between the two adjacent layers leaves the interface with only slip to be controlled for compatibility. Slip is defined as the horizontal differential displacement at the interface between two adjacent layers.
The total horizontal displacement of the concrete component \((U_{ci})\) at the interface can be expressed as:

\[
U_{ci} = U_c - Z_{ci} \cdot W_{c,x} \hspace{2cm} (6.26)
\]

Where \((U_c)\) is the centroidal displacement of the concrete component in the x-direction, \((W_c)\) is the centroidal displacement of the concrete component in the z-direction, and \((Z_{ci})\) is the z-coordinate of the first interface relative to the main axes of the section.

The thickness of CFRP layers is too small compared with the concrete element and hence, the influence of the curvature on the longitudinal displacement of each layer can be ignored. Therefore, the longitudinal displacement of the CFRP layer is considered constant across the depth of the layer and will be taken as below:

\[
U_{1i} = U_1 \hspace{2cm} (6.27)
\]

\[
U_{2i} = U_2 \hspace{2cm} (6.28)
\]

\[
U_{3i} = U_3 \hspace{2cm} (6.29)
\]

In which \([U_{1i}, U_{2i}, and U_{3i}]\) represent the total longitudinal displacement of CFRP layers at each interface.

The slip at the interface between the concrete and first CFRP layer \((U_{c,1})\) can be defined as:

\[
U_{c,1} = U_{ci} - U_{1i} \hspace{2cm} (6.30)
\]

Substituting the values of \((U_{ci})\) and \((U_{1i})\) from equations (6.26) and (6.27) into equation (6.30), gives:

\[
U_{c,1} = (U_c - Z_{ci} \cdot W_{c,x}) - U_1 \hspace{2cm} (6.31)
\]

Similarly, the slip between the two adjacent CFRP layers is
IMPROVING BOND STRENGTH FOR CFRP-RC BEAMS INTERFACE

\[ U_{1,2} = U_1 - U_2 \]  \hspace{1cm} (6.32)

\[ U_{2,3} = U_2 - U_3 \]  \hspace{1cm} (6.33)

\( (K_{s1}) \) is the shear stiffness of the bonding agent per unit length of the interface between the concrete element and the first CFRP layer and \((Q_1)\) represents the longitudinal shear force for the same interface. Then,

\[ Q_1 = K_{s1} \cdot (U_{c1}) \]  \hspace{1cm} (6.34)

Similarly, the longitudinal shear force for the subsequent interfaces between CFRP layers can be defined as follows

\[ Q_2 = K_{s2} \cdot (U_{1,2}) \]  \hspace{1cm} (6.35)

\[ Q_3 = K_{s3} \cdot (U_{2,3}) \]  \hspace{1cm} (6.36)

Considering the force equilibrium at the concrete element alone in the x-direction as in Figure 6.3 gives:

\[ \delta F_c = Q_1 \cdot \delta x \]  \hspace{1cm} (6.37)

Figure 6.3: Force equilibrium for concrete component

Then, dividing equation (6.37) above by \((\delta x)\) and substituting for the \((Q_1)\) from equation (6.35), gives:
\[ F_{c,x} = K_{s1} \cdot (U_{c,1}) \] .................................(6.38)

Substituting the interface slip between the concrete and first CFRP layer \((U_{c,1})\) from equation (6.31) into equation (6.38) above and rearranging the terms gives:

\[ F_{c,x} - K_{s1} \cdot [(U_c - Z_{ei}, W_{c,x}) - U_1] = 0 \] .................................(6.39)

The above equation is the first compatibility equation and hence the third equation required for the basic solution.

Similarly, considering the force equilibrium for the second CFRP layer as shown in Figure 6.4, gives:

\[ F_{1,x} = Q_2 - Q_1 \] .................................(6.40)

Figure 6.4: Force equilibrium for the second CFRP layer

Substituting the values of \((Q_2 \& Q_1)\) from equations (6.34) and (6.35) into equation (6.40), and rearrange will give:

\[ F_{c,x} + F_{1,x} - K_{s2} \cdot (U_1 - U_2) = 0 \] .................................(6.41)

This equation is the second compatibility equation and hence the fourth equation required for the basic solution.

Finally, the force equilibrium in the last (third) CFRP layer as shown in Figure 6.5, and is given:

\[ F_{3,x} + Q_3 = 0 \] .................................(6.42)
Figure 6.5: Force equilibrium for the third CFRP layer

The final form of equation (6.42) after substituting the value of \( Q_3 \) from equation (6.36) and rearranging the terms is:

\[
F_{3,x} + K_{33} \cdot [U_2 - U_3] = 0 \quad \text{.................................................................} \quad (6.43)
\]

Equation (6.34) is the fifth (last) equation required for the basic solution.

6.3. Material Properties

In order to incorporate non-linear material properties into the mathematical model, a well-defined stress-strain relationship for each material in the composite section is required. Hence, in the following section the material properties of CFRP and concrete and shear stiffness of bonding agent are defined.

6.3.1. Stress-Strain Relationship

CFRP under loading has an elastic material property until failure [21, 22]. Therefore, the modulus of elasticity of the CFRP layers is constant and taken as a slope of the stress-strain curve at any loading point. Hence, \( E_i \) for CFRP layers at any loading point is equal to 230 GPa as taken from the manufacture’s data sheet, as shown in Figure 6.6.

The ultimate compressive strain for concrete is limited to 0.0035. However, the tensile strength of the concrete element is ignored. The modulus of elasticity of concrete \( E_c \) is taken as a function of strain.
According to Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings (BS EN 1992-1-1:2004+A1:2014) [178], the stress-strain relationship for non-linear structural analysis shown in Figure 6.7 is given as below:

\[
\frac{\sigma_c}{f_{cm}} = \frac{k\eta - \eta^2}{1 + (k - 2)\eta}, \quad \{0 < |\varepsilon_{c1}| < |\varepsilon_{cu1}|\} \quad \text{.................................................. (6.44)}
\]

where:

\[
\eta = \frac{\varepsilon_c}{\varepsilon_{c1}} \quad \text{.................................................. (6.45)}
\]

\[
k = \frac{1.05f_{cm}\varepsilon_{c1}}{f_{cm}} \quad \text{.................................................. (6.46)}
\]
σ_c is compressive stress in the concrete, ε_{c1} is the strain at ultimate stress, ε_{cu1} is ultimate compressive strain in the concrete, E_{cm} is scant modules of elasticity of concrete, and f_{cm} is the mean value of concrete cylinder compressive strength.

The stress-strain relationship used for the design cross-section of the concrete is shown in Figure 6.8.

\[ \sigma_c = f_{cd} \left[ 1 - \left( 1 - \frac{\varepsilon_c}{\varepsilon_{c2}} \right)^n \right] \quad \text{for } 0 \leq \varepsilon_c \leq \varepsilon_{c2} \] .................................(6.47)

\[ \sigma_c = f_{cd} \quad \text{for } \varepsilon_{c2} \leq \varepsilon_c \leq \varepsilon_{cu2} \] .................................(6.48)

\[ f_{cd} = \alpha_{cc} f_{ck} / \gamma_c \] .................................(6.49)

where \( n \) is the exponent, \( \varepsilon_{c2} \) the strain at reaching the maximum strength, \( \varepsilon_{cu2} \) is the ultimate strain, \( f_{cd} \) is the design value of concrete compressive strength, \( f_{ck} \) is the characteristic compressive cylinder strength of concrete at 28 days, \( \alpha_{cc} \) is the coefficient, and \( \gamma_c \) is the partial safety factor of the concrete.

The equation (6.47) can be simplified for the normal concrete, as below:

\[ \sigma_c = f_{ck} (1.0 \times 10^3 \varepsilon_c - 2.5 \times 10^5 \varepsilon_c^2) \] .................................(6.50)
All the expression values in equations (6.44 – 6.50) are given in Table 6.1.

Table 6.1: Strength and deformation characteristics of concrete reproduced [178]

<table>
<thead>
<tr>
<th>Strength classes for concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{ck}$ (MP)</td>
</tr>
<tr>
<td>$f_{cm}$ (MPa)</td>
</tr>
<tr>
<td>$E_{cm}$ (GPa)</td>
</tr>
<tr>
<td>$\varepsilon_{c1} %$</td>
</tr>
<tr>
<td>$\varepsilon_{c2} %$</td>
</tr>
<tr>
<td>$\varepsilon_{cu1} %$</td>
</tr>
<tr>
<td>$\varepsilon_{cu2} %$</td>
</tr>
<tr>
<td>$n$</td>
</tr>
<tr>
<td>$\alpha_{cc}$</td>
</tr>
<tr>
<td>$\gamma_c$</td>
</tr>
</tbody>
</table>

The secant modulus of the concrete for non-linear structural analysis can be expressed by deriving of equation (6.50) with respect to compressive strain in the concrete ($\varepsilon_c$), as below:

$$E_c = \frac{\delta f_c}{\delta \varepsilon_c}$$  \hspace{1cm}  \text{.......................... (6.51)}

$$E_c = f_{ck} (1.0 \times 10^3 - 5.0 \times 10^5 \varepsilon_c)$$  \hspace{1cm}  \text{.......................... (6.52)}

6.3.2. Shear Stiffness of Bonding Agent

The shear stiffness of the bonding agent ($K_s$) is defined mathematically as the secant value for the load-slip curve as shown in Figure 6.9.

The shear load ($P$) for the three epoxies can be obtained from the curve fitting equations of the load-slip of single shear tests carried out previously on the concrete prisms as shown in Figure 6.9 as follows:

$$P_{NE} = -11.5(Slip)^2 + 33.5(Slip)$$  \hspace{1cm}  \text{.......................... (6.53)}

$$P_{CTBN} = -17.0(Slip)^2 + 28.5(Slip)$$  \hspace{1cm}  \text{.......................... (6.54)}
\[ P_{ATBN} = -11.5(Slip)^2 + 26.0(Slip) \]  \hspace{1cm} (6.55)

In which subscripts NE, CTBN and ATBN refers to neat epoxy, CTBN-modified epoxy and ATBN-modified epoxy, respectively.

The uniform shear flow \( q \) of the bonding agent at the interface can be obtained by dividing the shear load \( P \) by the bond length, as follows:

\[ q = \frac{P + 1000}{\text{bond length}}, \frac{N}{\text{mm}} \]  \hspace{1cm} (6.56)

In which, the bond length is taken from test specimens as 80 mm.

Then the shear flow \( q \) can be obtained as below:

\[ q_{NE} = -143.7(Slip)^2 + 418.7(Slip) \]  \hspace{1cm} (6.57)

\[ q_{CTBN} = -212.5(Slip)^2 + 325.0(Slip) \]  \hspace{1cm} (6.58)

\[ q_{ATBN} = -143.7(Slip)^2 + 325.713(Slip) \]  \hspace{1cm} (6.59)

Now, the shear stiffness \( K_s \) can be obtained by deriving the uniform shear flow \( q \) with respect to the interface slip \( \frac{\delta q}{\delta \text{slip}} \) as follows:
\[ K_{s,NE} = -287.4(Slip) + 418.7 \] \hspace{1cm} (6.60)

\[ K_{s,CTBN} = -425.0(Slip) + 421.838 \] \hspace{1cm} (6.61)

\[ K_{s,ATBN} = -287.4(Slip) + 325.713 \] \hspace{1cm} (6.62)

### 6.4. Numerical Integration of Force-Displacement Equations

The five equations (6.23, 6.25, 6.39, 6.41 and 6.43) required for the basic solution can be re-ordered as follows:

\[ M_{c,xx} - F_{1,xx}d_1 - F_{2,xx}(d_1 + d_2) - F_{3,xx}(d_1 + d_2 + d_3) = \rho \] \hspace{1cm} (6.63)

\[ F_{c,x} + F_{1,x} + F_{2,x} + F_{3,x} = 0 \] \hspace{1cm} (6.64)

\[ F_{c,x} - K_{s1}[(U_c - Z_{ct}W_{c,x}) - U_1] = 0 \] \hspace{1cm} (6.65)

\[ F_{c,x} + F_{1,x} - K_{s2}[U_1 - U_2] = 0 \] \hspace{1cm} (6.66)

\[ F_{3,x} + K_{s3}[U_2 - U_3] = 0 \] \hspace{1cm} (6.67)

When the material properties \((E)\) are constants, equations (6.15-6.18) and (6.20) can be integrated analytically to give the axial forces \((F)\) and moments \((M)\) in terms of displacements \((U)\) and \((W)\). Otherwise, if the material properties are non-linear functions of strain, equations (6.11 and 6.20) can be evaluated numerically.

In the case of CFRP layers, the material properties are constants. Therefore, equations (6.16 to 6.18) can be integrated analytically to determine the axial force \((F_i)\) on the CFRP layer as follows:

\[ F_1 = E_1 A_1 U_{1,x} \] \hspace{1cm} (6.68)

\[ F_2 = E_2 A_2 U_{2,x} \] \hspace{1cm} (6.69)

\[ F_3 = E_3 A_3 U_{3,x} \] \hspace{1cm} (6.70)
Where the strain ($\varepsilon_i$) is taken as a direct function of the axial displacement in that layer (i) as below:

$$\varepsilon_1 = U_{1,x} \hspace{2cm} (6.71)$$

$$\varepsilon_2 = U_{2,x} \hspace{2cm} (6.72)$$

$$\varepsilon_3 = U_{3,x} \hspace{2cm} (6.73)$$

On the other hand, for the concrete, the material properties are nonlinear functions of strain. Therefore, equations (6.15 and 6.20) can be evaluated numerically to give the axial force ($F_c$) and moments ($M_c$). This can be achieved by dividing the cross-sectional area of the concrete element into a number of elemental strips (s) having area of ($\delta A_c)_s$ at distance ($z)_s$, from the origin of co-ordinates, as shown in Figure 6.10, and replacing the integrals with a summation over the appropriate area. Therefore, equations (6.15 and 6.20) can be redefined as follows:

$$F_c = \sum_s (E_c)_s \left( (U_{c,x})_s - (Z_c)_s W_{xx} \right) (\delta A_c)_s \hspace{2cm} (6.74)$$

$$M_c = -\sum_s (E_c)_s \left( (U_{c,x})_s - (Z_c)_s W_{xx} \right) (Z_c \delta A_c)_s \hspace{2cm} (6.75)$$

![Figure 6.10: Subdivision of concrete cross-section into elemental areas](image-url)
Then, equations (6.68-6.70, 6.74, and 6.75) are substituted into equations (6.63 to 6.67). These equations provide a set of simultaneous differential equations to the fourth order in terms of displacements \((W, U_c, U_1, U_2, U_3)\) as follows

\[
\left[ \sum_1^s (E_c)_s (l_c)_s W_{xxxx} \right] - E_1A_1U_{1,xxx}d_1 - E_2A_2U_{2,xxx}(d_1 + d_2) - E_3A_3U_{3,xxx}(d_1 + d_2 + d_3) = \rho \quad \text{................................................. (6.76)}
\]

\[
\left[ \sum_1^s (E_c)_s (A_c)_s U_{c,xxx} \right] + E_1A_1U_{1,xx} + E_2A_2U_{2,xx} + E_3A_3U_{3,xx} = 0 \quad \text{.............. (6.77)}
\]

\[
\left[ \sum_1^s (E_c)_s (A_c)_s U_{c,xxx} \right] - Ks_1 \left\{ [\sum_1^s (U_c - (Z_c)_s W_x)] - U_1 \right\} = 0 \quad \text{.............. (6.78)}
\]

\[
\left[ \sum_1^s (E_c)_s (A_c)_s U_{c,xxx} \right] + E_1A_1U_{1,xx} - Ks_2(U_1 - U_2) = 0 \quad \text{.............. (6.79)}
\]

\[
E_3A_3U_{3,xx} + Ks_3[U_2 - U_3] = 0 \quad \text{................................................. (6.80)}
\]

### 6.5. Finite Difference Analysis

The basic simultaneous equations (6.65 to 6.69) can be solved by expressing the displacement derivatives in finite difference form and solving the resulting set of algebraic equations iteratively. These differential equations have fourth order derivatives in vertical displacement \((W)\) and third order derivatives in longitudinal displacement \((U)\). These derivatives can be converted to the form of central difference by using five node points as shown in Figure 6.11.

As an example for node \((m)\), the derivatives of vertical displacement \((W)\) can be extracted as follows:

\[
W_x = \frac{W_{(m+1)} - W_{(m-1)}}{2\Delta x} \quad \text{................................................. (6.81)}
\]

\[
W_{xx} = \frac{W_{(m+1)} - 2W_m + W_{(m-1)}}{\Delta x^2} \quad \text{................................................. (6.82)}
\]
\[ W_{xxx} = \frac{W_{(m+2)} - 2W_{(m+1)} + 2W_{(m-1)} - W_{(m-2)}}{2\Delta x^3} \] ................................. (6.83)

\[ W_{xxxx} = \frac{W_{(m+2)} - 4W_{(m+1)} + 6W_{(m)} - 4W_{(m-1)} + W_{(m-2)}}{\Delta x^4} \] ................................. (6.84)

In which, \((W_m)\) denotes the value of \((W)\) at node \((m)\), and \(\Delta x\) indicates the distance between two adjacent nodes.

![Central finite difference boundary node](image)

The longitudinal displacement \((U_c, U_1, U_2, U_3)\) can be expressed in central finite difference with the same method above.

### 6.6. Boundary Conditions

The solution of a set of five algebraic equations needs a particular boundary condition at each support. In general, two external nodes, as shown in Figure 6.11, are required to produce a full solution including the boundary nodes. However, if each external node is assigned (5) degrees of freedom to be consistent with the internal nodes, a total of (10) boundary conditions at each support are required to obtain a solution. A total of (10) boundary conditions at each support can be provided by the constraints enforced upon the concrete beam and CFRP layers end, as explained herein. The boundary conditions for a simply supported RC beam retrofitted with
(3) layers of CFRP, are given below in which \((L)\) denotes the beam span, \((R)\) denotes support reactions and subscripts \((R)\) and \((L)\) denote right and left supports as follows:

- **B.C. 1**: the deflection on the both beam supports assumed equal to zero.
- **B.C. 2**: there is no bending moment at both beam-ends (free rotation).
- **B.C. 3**: the horizontal displacement is assumed to constrain at one end only.
- **B.C. 4**: The concrete beam is assigned the whole shear force at the supports (the reactions) as the CFRP layers are assumed to have no shear stiffness.
- **B.C. 5 – B.C. 7**: the longitudinal strain at both ends of each layer is taken to be zero. However, the CFRP layers are assumed to have free ends in terms of longitudinal displacements \((U)\).
- **B.C. 8 – B.C. 10**: the fourth derivative of the longitudinal displacement at both ends of each CFRP layer is equal to zero.

Therefore, the boundary conditions needed at each support are reordered as following:

\[
\begin{align*}
\text{at } (x = 0) & \quad \text{at } (x = L) \\
(B.C.1) & \quad W = 0 \quad W = 0 \\
(B.C.2) & \quad W_{xx} = 0 \quad W_{xx} = 0 \\
(B.C.3) & \quad U_c = 0 \quad U_{c,x} = 0 \\
(B.C.4) & \quad V_c = R_L \quad V_c = R_R \\
(B.C.5) & \quad U_{1,x} = 0 \quad U_{1,x} = 0 \\
(B.C.6) & \quad U_{2,x} = 0 \quad U_{2,x} = 0 \\
(B.C.7) & \quad U_{3,x} = 0 \quad U_{3,x} = 0 \\
(B.C.8) & \quad U_{1,xxxx} = 0 \quad U_{1,xxxx} = 0 \\
(B.C.9) & \quad U_{2,xxxx} = 0 \quad U_{2,xxxx} = 0
\end{align*}
\]
The shear force \( V_c \) of the concrete element in (B.C. 4) equation (6.89) can be defined in terms of vertical displacement \( W \) and longitudinal displacement \( U_c \) by taking the moment of the concrete composite alone about the centroid of the section, as follows:

\[
V_c = M_{c,x} + F_{c,x} \quad \text{......................................................... (6.96)}
\]

Differentiating equation (6.74) and (6.75) with respect to \( x \) will give:

\[
F_{c,x} = \sum_i^1 (E_c)_s \cdot (A_c)_s \cdot U_{c,xx} \quad \text{......................................................... (6.97)}
\]

\[
M_{c,x} = \sum_i^1 (E_c)_s \cdot (I_c)_s \cdot W_{xxx} \quad \text{......................................................... (6.98)}
\]

Substituting the values of \( F_{c,x} \) and \( M_{c,x} \) in equation (6.96) will give

\[
V_c = \sum_i^1 (E_c)_s \cdot (I_c)_s \cdot W_{xxx} + \sum_i^1 (E_c)_s \cdot (A_c)_s \cdot U_{c,xx} \quad \text{......................................................... (6.99)}
\]

Hence, (B.C. 4) becomes

\[
\sum_i^1 (E_c)_s \cdot (I_c)_s \cdot W_{xxx} + \sum_i^1 (E_c)_s \cdot (A_c)_s \cdot U_{c,xx} = R_L = R_R \quad \text{......................................................... (6.100)}
\]

### 6.7. Solution Technique

To solve a set of nonlinear equations numerically, a computer program has been built using Matlab software (version R2014a-Research) to implement the proposed mathematical model. In this programme, the proposed model applied to the RC beam retrofitted with (3) CFRP layers and the span length \( l \) is divided to \( m \) nodes. Two imaginary nodes are required on each side of the beam, and the distance between two adjacent nodes is \( \Delta l = \frac{l}{(m-1)} \), as shown in Figure 6.12.
The first step of the computer programmes is inputting data of dimension and the mechanical properties of the retrofitted beam and CFRP layers as well as, number of nodes \((m)\), and the shear stiffness \((K_s)\) of the epoxy resin.

The second step is expressing and defining the fourth order derivatives in vertical displacement \((W)\) and third order derivatives in longitudinal displacement \((U)\) and converting to the form of central difference by using five node points as shown in Figure 6.11.

The third step of the computer programmes defines the set of five simultaneous equations in terms of displacement unknowns \((W_c, U_1, U_2, U_3)\) for each node required to find the basic solution. The set of \((5m)\) simultaneous equations can be expressed in matrices form as follows:

\[
AX = B
\]

(6.101)

Where \(A\) and \(B\) are known matrices called matrix of coefficients and matrix of constants, respectively. \(X\) is the unknown matrix of displacement unknowns.

Matrix \(A\) contains \((5)\) displacement coefficients for each node up to the node \((m)\), in addition to \((10)\) displacement coefficients of the boundary conditions representing the coefficient of displacements for the imaginary nodes that result in a square-banded matrix size of \((5(m + 4))\). Matrix \(B\) contains \((5)\) constants of each node up to node \((m)\) in addition to
(10) constants of the boundary conditions that result in a column matrix of size \((5(m + 4), 1)\).

Matrix \(X\) contains (5) displacement unknowns for each node up to node \((m)\), in addition to
(10) displacement unknowns of the boundary conditions that result in a column matrix of
size \((5(m + 4), 1)\).

To solve \((5(m + 4))\) simultaneous equations, both sides of equation (6.101) are multiplied by
the inverse of the matrix \(A\), giving:

\[
A^{-1}AX = A^{-1}B \quad \text{................................................................. (6.102)}
\]

But \(A^{-1}A = I\) \quad \text{................................................................. (6.103)}

Where \(I\) is the identity matrix. Substituting equation (6.103) into equation (6.102) gives:

\[
IX = A^{-1}B \quad \text{................................................................. (6.104)}
\]

However, multiplying any matrix by an identity matrix \(I\) of the appropriate size leaves
the matrix unaltered, so equation (6.104) becomes:

\[
X = A^{-1}B \quad \text{................................................................. (6.105)}
\]

This result solves the \((5(m + 4))\) simultaneous equations and hence, all the displacements
\((W, U_c, U_1, U_2, U_3)\) for each node are known values.

After the fourth step, the material properties are assumed linear in the first stage of the solution,
and a set of nodal displacements corresponding to the first step of applied loading are
determined. Then the displacements \((W, U_c, U_1, U_2, U_3)\) slip at the interface and strains
throughout the composite beam, including the strain for each concrete subdivision as presented
in Figure 6.10, is determined. This strain is used to define the secant values of the material
properties of the concrete and the shear stiffness of the epoxy resin for the second stage of the
solution. The nonlinear material properties \((E_c)_s\) can be found iteratively for each concrete
subdivision using the equation (6.52). The process is repeated until the calculated displacements have converged, according to a prescribed criterion. For subsequent values of the applied loading, the iterative procedure is commenced with secant values of the material properties corresponding to the previously converged solution as shown in Figure 6.13, which reduces the number of iterations required.

After the material properties converged, the deflection, interface slip, and the longitudinal and differential strain at every node during the load stage were estimated. When the strain of concrete ($E_c$) reached the maximum limit (0.0035), the computer programme ended and the maximum load, maximum deflection, interface slip, and the longitudinal and differential strain were recorded.

![Figure 6.13: Iterative method procedure for $E_c$](image)

### 6.8. Conclusion

In this chapter, the mathematical model presented by Al-Ameri and Al-Mahaidi [88] has been developed to incorporate with inelastic properties of the RC beam retrofitted with multilayered CFRP sheets, to enable the prediction of the general behaviour of the retrofitted beam up to failure. A set of five differential equations was derived from the equilibrium and compatibility
conditions at each node. These differential equations have derivatives of the fourth order in vertical displacement \( W \) and the third order in longitudinal displacement \( U \), and solved by expressing the displacement derivatives in finite difference form using a Matlab computer programme. Beam behaviour including deflection, interface slip, and the longitudinal and differential strain was obtained at every node during the load stage. The computer programme was terminated when the stain of concrete \( E_c \) reached the maximum limit 0.0035; the maximum load, maximum deflection, interface slip, and the longitudinal and differential strain were recorded.
CHAPTER 7
MATHEMATICAL MODEL VERIFICATION AND PARAMETRIC STUDY

7.1. Introduction

In this chapter, the mathematical model presented in Chapter 6 will be verified with the previous and current test results. Key test outcomes such as maximum strength, maximum deflection, interface slip, and strain profile will be compared with model predictions. Once verified, the mathematical model will be used to investigate the influence of the major parameters on the general behaviour of RC beams retrofitted with multilayer CFRP composites. The parametric study will include the influence of variant shear stiffness of the epoxy resin and influence of CFRP layers. Additionally, the strain distribution along the concrete beam and CFRP layer will be analysed.

7.2. Model Verification

The maximum strength, maximum deflection, interface slip and strain profile results for the test beams were verified with the mathematical model prediction, as described in the following section.

7.2.1. First Series of Beams

The test results and predicted value of the load versus deflection up to failure for the first series are shown in Figures 7.1 to 7.4. The curves of the test and predicted results of beams are almost matching. After the beams first yield, the prediction of maximum load capacity and maximum deflection of the beams became close to the test results. Table 7.1 shows the comparison of predicted and recorded values for the first series of beams.
Figure 7.1: Experimental and predicted load-deflection curve for test beam A2

Figure 7.2: Experimental and predicted load-deflection curve for test beam A3

Figure 7.3: Experimental and predicted load-deflection curve for test beam A4
Figure 7.4: Experimental and predicted load-deflection curve for test beam A5

Table 7.1: Experimental and predicted load-deflection for the first series

<table>
<thead>
<tr>
<th>Beam</th>
<th>Ultimate load, kN</th>
<th>Max. central deflection at failure, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test</td>
<td>Predict</td>
</tr>
<tr>
<td>A2</td>
<td>160</td>
<td>165</td>
</tr>
<tr>
<td>A3</td>
<td>165</td>
<td>170</td>
</tr>
<tr>
<td>A4</td>
<td>165</td>
<td>170</td>
</tr>
<tr>
<td>A5</td>
<td>165</td>
<td>180</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td></td>
</tr>
</tbody>
</table>

The longitudinal strain was measured at 350 mm from the support on both the concrete surface (SG1) and the extreme CFRP layer (SG2), as described in Chapter 5. The predicted and measured differential strain curves for the first series are shown in Figures 7.5 to 7.8. The predicted values are close to the test values.
Figure 7.6: Experimental and predicted differential strain (SG2-SG1) of beam A3

Figure 7.7: Experimental and predicted differential strain (SG2-SG1) of beam A4

Figure 7.8: Experimental and predicted differential strain (SG2-SG1) of beam A5
The values of the predicted and measured differential strain for the first series of beams at a load of 140 kN are listed in Table 7.2.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Differential strain (SG1-SG2), x10⁻³ mm/mm @ 140 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test</td>
<td>Predict</td>
</tr>
<tr>
<td>A2</td>
<td>0.83</td>
</tr>
<tr>
<td>A3</td>
<td>2.36</td>
</tr>
<tr>
<td>A4</td>
<td>1.99</td>
</tr>
<tr>
<td>A5</td>
<td>1.10</td>
</tr>
</tbody>
</table>

The comparison between the experimental and predicted interface slips for the first series of test beams are shown in Figures 7.9 to 7.12. The predicted values of interface slip are close to the measured values.
The test results confirmed the predicted results. The measured and predicted interface slip values at a load of 140 kN are listed in Table 7.3.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Interface slip, mm @ 140 kN</th>
<th>Predict/Test</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test</td>
<td>Predict</td>
</tr>
<tr>
<td>A2</td>
<td>0.36</td>
<td>0.30</td>
</tr>
<tr>
<td>A3</td>
<td>0.14</td>
<td>0.16</td>
</tr>
<tr>
<td>A4</td>
<td>0.08</td>
<td>0.14</td>
</tr>
<tr>
<td>A5</td>
<td>0.26</td>
<td>0.31</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>1.22</td>
</tr>
</tbody>
</table>
7.2.2. Second Series of Beams

In the second series, the experimental and predicted results for load-deflection for test beams A6 and A7 are shown in Figures 7.13 and 7.14. The predicted results were close to the test results. The predicted ultimate load for both beams is slightly lower than the experimental results. However, the predicted maximum deflection is slightly higher than the experimental results. Table 7.4 show the values of predicted and experimental results.

![Figure 7.13: Experimental and predicted load-deflection curve for test beam A6](image)

![Figure 7.14: Experimental and predicted load-deflection curve for test beam A7](image)

<table>
<thead>
<tr>
<th>Beam</th>
<th>Ultimate load, kN</th>
<th>Max. deflection before failure, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test</td>
<td>Predict</td>
</tr>
<tr>
<td>A6</td>
<td>187</td>
<td>180</td>
</tr>
<tr>
<td>A7</td>
<td>166</td>
<td>180</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>1.02</td>
</tr>
</tbody>
</table>

Table 7.4: Experimental and predicted load-deflection for the second series
The predicted differential strain values for both of the test beams in the second series were close to the test values, as shown in Figures 7.15 and 7.16.

![Figure 7.15: Experimental and predicted differential strain (SG2-SG1) of beam A6](image1)

![Figure 7.16: Experimental and predicted differential strain (SG2-SG1) of beam A7](image2)

In Table 7.5, the predicted differential strain of the beams was confirmed by the test beam results.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Differential strain (SG1-SG2), x10^{-3} mm/mm @ 140kN</th>
<th>Predict</th>
<th>Predict/Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>A6</td>
<td>2.41</td>
<td>2.23</td>
<td>0.93</td>
</tr>
<tr>
<td>A7</td>
<td>1.56</td>
<td>1.49</td>
<td>0.96</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
<td>0.95</td>
</tr>
</tbody>
</table>
The interface slip values for both beams are shown in Figures 7.17 and 7.18; the predicted values are close to the measured values for both beams.

![Figure 7.17: Experimental and predicted interface slip of beam A6](image1)

![Figure 7.18: Experimental and predicted interface slip of beam A7](image2)

The predicted and measured values of differential strain at 140 kN are listed in Table 7.6. The predicted values are confirmed by the test results.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Interface slip, mm @ 140 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test</td>
</tr>
<tr>
<td>A6</td>
<td>0.53</td>
</tr>
<tr>
<td>A7</td>
<td>0.32</td>
</tr>
<tr>
<td></td>
<td>Average</td>
</tr>
</tbody>
</table>
7.2.3. Third Series of Beams

In relation to the third series, the load-deflection pattern for beam A8 is shown in Figure 7.19. The predicted value of deflection is higher than the experimental result, while the test result for the maximum sustained load is greater than the predicted value.

![Figure 7.19: Experimental and predicted load-deflection curve for beam A8](image)

The predicted results for beams A9 and A10 matched and were close to the experimental results as shown in Figures 7.20 and 7.21. The experimental and predicted results for the third series are listed in Table 7.7.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Ultimate load, kN</th>
<th>Max. deflection @ failure, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test</td>
<td>Predict</td>
</tr>
<tr>
<td>A8</td>
<td>190</td>
<td>180</td>
</tr>
<tr>
<td>A9</td>
<td>175</td>
<td>180</td>
</tr>
<tr>
<td>A10</td>
<td>175</td>
<td>180</td>
</tr>
<tr>
<td>Average</td>
<td>1.00</td>
<td>Average</td>
</tr>
</tbody>
</table>
The predicted differential strain for the third series is shown in Figures 7.22 to 7.24. The predicted differential strain for all the beams of the third series is shown a satisfied match with the test results at a load of 140 kN.
The experimental and predicted differential strain values for the third series beams are listed in Table 7.8.

**Table 7.8: Predicted and test values of differential strain for the third series**

<table>
<thead>
<tr>
<th>Beam</th>
<th>Differential strain (SG1-SG2), x10-3 mm/mm @ 140kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test</td>
</tr>
<tr>
<td>A8</td>
<td>0.42</td>
</tr>
<tr>
<td>A9</td>
<td>1.16</td>
</tr>
<tr>
<td>A10</td>
<td>1.30</td>
</tr>
<tr>
<td></td>
<td>Average</td>
</tr>
</tbody>
</table>
The predicted and measured interface slips are shown in Figures 7.25 to 7.27 and are listed in Table 7.9. All the predicted values are close to the measured values and show some deviation from the benchmark at a load of 140 kN.

Figure 7.25: Experimental and predicted interface slip of beam A8

Figure 7.26: Experimental and predicted interface slip of beam A9

Figure 7.27: Experimental and predicted interface slip of beam A10
Table 7.9: Experimental and predicted interface slip for the third series

<table>
<thead>
<tr>
<th>Beam</th>
<th>Interface slip, mm @ 140 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test</td>
</tr>
<tr>
<td>A8</td>
<td>0.70</td>
</tr>
<tr>
<td>A9</td>
<td>0.58</td>
</tr>
<tr>
<td>A10</td>
<td>0.64</td>
</tr>
<tr>
<td></td>
<td>Average</td>
</tr>
</tbody>
</table>

7.3. **Comparison with Previous Tests**

To extend the verification, the model was validated with previous experimental work carried out by several researchers, as discussed below.

7.3.1. **Gao et al. Test**

Gao et al. [89] performed a four-point loading flexure test on eight RC beams strengthened with CFRP sheet using the different CTBN-modified epoxy content. The beams were 150 mm x 200 mm in cross-section, 2000 mm in length and 1800 mm in span, as shown in Figure 7.28. Four beams were chosen from Gao et al.’s experimental work because beams A0 and B0 used neat epoxy and beams A2 and B2 used CTBN-modified epoxy. The specifications of these beams are listed in Table 7.10.

The epoxy used to bond the CFRP plate for beams A0 and B0 was MRL-A3 resin and is considered a neat epoxy, while 20% of the CTBN-modified epoxy was used to bond the CFRP plate for beams A2 and B2, which is considered to be a CTBN-modified epoxy used in this research. The specifications of these beams are listed in Table 7.10.

The ultimate load and the maximum deflection prediction generated by the model are within 10% of the experimental results, as listed in Table 7.11.
IMPROVING BOND STRENGTH FOR CFRP-RC BEAMS INTERFACE

Figure 7.28: Geometry and dimensions of RC beam specimen test by Gao et al. [89]

Table 7.10: Beams designation and its material properties [89]

<table>
<thead>
<tr>
<th>Beam</th>
<th>CFRP thickness</th>
<th>Epoxy</th>
<th>Concrete compressive strength</th>
<th>Concrete modulus</th>
<th>CFRP modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>A0</td>
<td>0.22 mm</td>
<td>0% CTBN</td>
<td>35.7 MPa</td>
<td>25 GPa</td>
<td>235 GPa</td>
</tr>
<tr>
<td>A2</td>
<td>0.22 mm</td>
<td>20% CTBN</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B0</td>
<td>0.44 mm</td>
<td>0% CTBN</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B2</td>
<td>0.44 mm</td>
<td>20% CTBN</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 7.11: Comparison data between predicted and Gao et al. test

<table>
<thead>
<tr>
<th>Beam No</th>
<th>Epoxy</th>
<th>Ultimate load, kN</th>
<th>Deflection, mm</th>
<th>Predict/Test</th>
<th>Predict/Test</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Exp.</td>
<td>Predict</td>
<td>Predict/Test</td>
<td>Exp.</td>
<td>Predict</td>
</tr>
<tr>
<td>A0</td>
<td>NE</td>
<td>80.7</td>
<td>90</td>
<td>1.12</td>
<td>13.1</td>
</tr>
<tr>
<td>A20</td>
<td>CTBN</td>
<td>87.9</td>
<td>90</td>
<td>1.02</td>
<td>14.3</td>
</tr>
<tr>
<td>B0</td>
<td>NE</td>
<td>86.4</td>
<td>100</td>
<td>1.16</td>
<td>9.5</td>
</tr>
<tr>
<td>B20</td>
<td>CTBN</td>
<td>96.8</td>
<td>100</td>
<td>1.03</td>
<td>10.8</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>1.08</td>
<td>Average</td>
<td>1.00</td>
<td></td>
</tr>
</tbody>
</table>

7.3.2. Obaidat et al. Test

Obaidat et al. [179] conducted a four-point loading flexural test on four RC beams strengthened with different lengths of CFRP laminates using a neat epoxy resin. The beams were 150 mm x 300 mm in cross-section, 1960 mm in length and 1560 mm in span as shown in Figure 7.29.

Figure 7.29: Geometry and dimensions of RC beam specimen tested by Obaidat et al. [179]
The beam RF1 was retrofitted with CFRP laminate along the beam span among the other beams, which is similar to the procedure for testing beams in this research, as shown in Figure 7.30. The beam RF1 designation and its material properties are listed in Table 7.12.

![Figure 7.30: Lengths of CFRP laminate in test beam RF1 tested by Obaidat et al. [179]](image)

<table>
<thead>
<tr>
<th>Beam</th>
<th>CFRP thickness</th>
<th>Epoxy</th>
<th>Concrete compressive strength</th>
<th>Concrete modulus</th>
<th>CFRP modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>RF1</td>
<td>1.2</td>
<td>Neat</td>
<td>40 MPa</td>
<td>32 GPa</td>
<td>165 GPa</td>
</tr>
</tbody>
</table>

The predicted results are listed in Table 7.13, and compared with Obaidat et al.’s test results. The predicted ultimate load was 180 kN compared with 166 kN from the test, while the predicted maximum deflection was 7.1 mm compared with 7.9 mm from the beam test. Overall, the predicted results are within 10% of the test result for RF1.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Epoxy</th>
<th>Ultimate load, kN</th>
<th>Deflection, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Test</td>
<td>Predict</td>
</tr>
<tr>
<td>RF</td>
<td>NE</td>
<td>166</td>
<td>180</td>
</tr>
</tbody>
</table>

### 7.3.3. Wenwei and Guo Test

Wenwei and Guo [180] tested seven RC beams under a four-point loading setup. Six RC beams strengthened with two CFRP layers were tested under a different loading scheme. The beam’s uniform cross-sectional dimension was 150 mm x 250 mm; it was 2700 mm in length and 2400 mm in span, as shown in Figure 7.31.
The beam chosen to verify the model is CFC30 because it was strengthened with two CFRP layers and tested under a continuous loading scheme up to failure. The other beams were tested under different load systems. The beam details and materials properties are listed in Table 7.14.

Table 7.14: Material properties of the tested beam [180]

<table>
<thead>
<tr>
<th>Beam</th>
<th>CFRP dimension</th>
<th>CFRP layers</th>
<th>Epoxy</th>
<th>Concrete compressive strength</th>
<th>Concrete modulus</th>
<th>CFRP modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFC30</td>
<td>2300x150x0.222 mm</td>
<td>2</td>
<td>Neat</td>
<td>40.3 MPa</td>
<td>32.7 GPa</td>
<td>212 GPa</td>
</tr>
</tbody>
</table>

A comparison of the predicted and test results for beam CFC30 is provided in Table 7.15. The predicted results are within 10% of the test result.

Table 7.15: Comparison data between predicted and Wenwei-Guo test

<table>
<thead>
<tr>
<th>Beam</th>
<th>Status</th>
<th>Load, kN</th>
<th>Deflection, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Test</td>
<td>Predict</td>
</tr>
<tr>
<td>CFC30</td>
<td>@ yield</td>
<td>90</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>@ failure</td>
<td>140</td>
<td>150</td>
</tr>
</tbody>
</table>

7.3.4. Kotynia et al. Test

Kotynia et al. [181] tested 10 rectangular RC beams under four-point loading. The beams were strengthened in flexure with a different, externally bonded CFRP configuration to investigate the effect of using U- and L-shaped CFRP anchorage on the load carrying capacity and CFRP
strength utilisation ratio. All the tested beams were conducted with a rectangular cross-section 150 mm x 300 mm and a clear span of 4200 mm as shown in Figure 7.32.

Two beams, namely B-08S and B-08M, were chosen because they were strengthened without any anchorage system. The beams’ designation and material properties are shown in Figure 7.33 and listed in Table 7.16.

The predicted maximum carrying the load and the maximum predicted strain on the CFRP sheet are compared with the experimental results in Table 7.17.
Table 7.17: Comparison data for predicted and Kotynia et al. test

<table>
<thead>
<tr>
<th>Beam</th>
<th>Load, kN</th>
<th>CFRP strain, %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test</td>
<td>Predict</td>
</tr>
<tr>
<td>B-08S</td>
<td>96</td>
<td>110</td>
</tr>
<tr>
<td>B-08M</td>
<td>140</td>
<td>130</td>
</tr>
<tr>
<td>Average</td>
<td>1.04</td>
<td>Average</td>
</tr>
</tbody>
</table>

7.4. Parametric Study

The influence of shear stiffness of the epoxy resin \((K_s)\) and the number of CFRP layers on the beam behaviour were investigated on a virtual beam, in terms of the maximum load, central deflection and interface slip. In addition, the strain distribution and the differential strain of the virtual beam were analysed.

The cross-section of the virtual beams was 200 mm in width and 350 mm in depth with a clear span of 4000 mm. The concrete compressive strength of the virtual beam was 40 MPa, and shear stiffness of the epoxy resin had a variant value. The predicted results of the parametric study are discussed below.

7.4.1. Epoxy Resin Shear Stiffness \((K_s)\)

The shear stiffness of the adhesive is a significant parameter affecting the integrity of the bond and hence its overall beam behaviour. A larger interface slip is predicted with a lower shear stiffness \((K_s)\) value of the epoxy, which in turn improves the composite ductile behaviour and delays failure [11]. The shear stiffness values of 250 N/mm², 500 N/mm² and 1000 N/mm² were chosen to apply to the virtual beam which were strengthened with one CFRP layer, to predict the maximum central deflection and interface slip.

In Figures 7.34 and 7.35, the predicted maximum load, central deflection and interface slip of these virtual beams are shown; the values are listed in Table 7.18.
In Figure 7.34, there is a small variation in the ultimate load of the virtual beams with varying $K_s$ of around 10 kN in difference. However, the central deflection of the beam with the value of 250 of $K_s$ was recorded as 96.23 mm, which is higher than the 500 and 1000 values of $K_s$ by 8% and 26% respectively, as shown in Table 7.18.

<table>
<thead>
<tr>
<th>Beam</th>
<th>$K_s$, N/mm²</th>
<th>Max. load, kN</th>
<th>Central deflection, mm</th>
<th>Interface slip, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>250</td>
<td>205</td>
<td>96.23</td>
<td>0.79</td>
</tr>
<tr>
<td>2</td>
<td>500</td>
<td>210</td>
<td>89.40</td>
<td>0.34</td>
</tr>
<tr>
<td>3</td>
<td>1000</td>
<td>215</td>
<td>76.58</td>
<td>0.20</td>
</tr>
</tbody>
</table>
From Figure 7.35, the higher interface slip value that occurred in the virtual beam used an epoxy resin had shear stiffness of 250 N/mm². The interface slip decreased with increased shear stiffness of the bond.

It is clear from the predicted results that the flexible bond has a lower shear stiffness value, which in turn improves the composite ductile behaviour and delays failure.

### 7.4.2. Number of CFRP Layers

The second parameter is the impact of varying CFRP layers on the beam behaviour. Increasing the number of CFRP layers to retrofit the concrete beam tends to increase the amount of CFRP reinforcement that will reduce beam ductility despite increasing the load capacity [14]. The failure mode is changed from CFRP rupture in the middle region to delamination of the CFRP layer in the concrete substrate by increasing CFRP layers from three to six [65].

The virtual beams were retrofitted with one, two and three retrofitted CFRP layers and epoxy resin with a shear stiffness value of 250 N/mm² was used to predict the beam behaviour. The predicted maximum load and central deflection are shown in Figures 7.36 and the predicted values are listed in Table 7.19.

In Figure 7.36, the maximum predicted load increased with increasing CFRP layers. The virtual beam retrofitted with one CFRP layer reached 205 kN, in contrast with 220 kN for the beam retrofitted with two CFRP layers and 240 kN for the beam retrofitted with three CFRP layers.

<table>
<thead>
<tr>
<th>Beam</th>
<th>CFRP layers</th>
<th>Max. load, kN</th>
<th>Central deflection, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>205</td>
<td>95.23</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>220</td>
<td>71.95</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>240</td>
<td>45.80</td>
</tr>
</tbody>
</table>

Although the maximum load increased with increasing CFRP layers, the beam ductility decreased. The deflection of the virtual beam retrofitted with one CFRP layer reached 95.23 mm in contrast to 71.95 mm for the beam retrofitted with two CFRP layers and only 45.80 mm for the beam retrofitted with three CFRP layers.
7.4.3. Strain Distribution

In this section, the strain profile along the virtual beam and the differential strain between the concrete surface and extreme CFRP layer were analysed with increment load until failure. Figure 7.37 shows the predicted longitudinal strain on the concrete surface along the virtual beam span for various load levels. The difference in the predicted value of longitudinal strain increased towards the midspan with load increment because of increased beam momentum.

The longitudinal strain values predicted on the CFRP layer along the virtual beam with the increment load are shown in Figure 7.38. As with the strain in concrete, the difference in the predicted value of longitudinal strain increased towards the midspan with load increment.
Figure 7.38: Longitudinal strain along the CFRP layer of the virtual beam

Figure 7.39 shows the predicted differential strain between the concrete surface and the CFRP layer at the quarter and centre of the virtual beam. The differential strain increased with load increment and increased toward the beam midspan. The predicted differential strain values at the quarter and beam centre are listed in Table 7.20.

![Differential strain plot](image)

**Figure 7.39: Predicted differential strain at central and quarter of the beam**

<table>
<thead>
<tr>
<th>Measurement Location</th>
<th>Differential strain, x10^{-3} mm/mm @40 kN</th>
<th>@140 kN</th>
<th>@200 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quarter of the beam (1000 mm)</td>
<td>0.108</td>
<td>1.146</td>
<td>1.984</td>
</tr>
<tr>
<td>Central of the beam (2000 mm)</td>
<td>0.146</td>
<td>1.546</td>
<td>2.698</td>
</tr>
</tbody>
</table>

**Table 7.20: Differential strain at various load levels**
7.5. Conclusion

In this chapter, the ultimate load, the maximum central deflection, differential strain and interface slip were verified using a mathematical model for different beams strengthened with CFRP layers using different bonding agents, namely MBrace epoxy, CTBN-modified epoxy and ATBN-modified epoxy. The predicted values were close to the experimental results. The experimental work carried out by four previous researchers, Gao et al., Obaidat, Wenwei et al. and Kotynia et al., verified the predicted results.

The soft epoxy resin used to bond the CFRP to the RC beam with a low shear stiffness had a greater predicted interface slip and central deflection than the other values. The minimum deflection value predicted when to use a higher shear stiffness bond.

The overall behaviour of the beam is affected by varying the CFRP layers. However, the beam ductility significantly decreased with increasing CFRP layers. The maximum load increased with increasing CFRP layers.

The longitudinal strain along the beam increased towards the beam centre on both the concrete surface and extreme CFRP layer. The differential strain between the concrete surface and CFRP layer increased with increased increment load and increased towards the central beam.
CHAPTER 8
CONCLUSIONS AND RECOMMENDATIONS

8.1. Introduction

CFRP sheets are commonly used to retrofit concrete structures. CFRP is lightweight, easy to install and repair, resistant to environmental conditions and has superior mechanical properties, making it a favoured material for the rehabilitation of infrastructure compared with conventional materials such as steel plates. However, structural engineers have concerns about the premature failure of CFRP sheets at the early loading stage. Debonding failure occurs when the concrete surface is not able to sustain the force transferred from CFRP. The other key parameter contributes to premature failure is the low toughness of the bonding agent used to bond the CFRP sheet to the concrete member. The main role of the bonding agent is to transfer stress from the CFRP to the retrofitted member. The high shear stiffness of the bonding agent does not permit to interlayer slip, which causes premature failure. The resin epoxy available for this application is stiff and has low toughness properties.

As outlined in the literature review, attempts to modify epoxy resins with a reactive liquid rubber is one of the methods used to improve epoxy toughness; this provides the bond’s flexibility and allows for interface slip, and therefore, improves the ductility of the retrofitted member.

The aim of this investigation was to improve beam behaviour (i.e., improve ductility) and delay debonding using a rubber modified epoxy.
8.2. Conclusions

From the experimental results, the following conclusions can be stated.

8.2.1. Neat Epoxy modification

The neat epoxy resin was modified using two types of liquid polymer butadiene-acrylonitrile rubber, namely 1300X13 CTBN and 1300X16 ATBN, to improve the toughness characteristic and consequently the behaviour of the retrofitted member.

The neat epoxy was modified with varying amounts of CTBN and ATBN modifiers (20 g, 25 g and 30 g in 100 g of neat epoxy). A single lap shear test was applied to 21 samples of steel plates to test the bond behaviour of different modified epoxy mixes using the CTBN and ATBN modifiers. The results indicate that 30 g of CTBN-modified epoxy and 25 g of ATBN-modified epoxy have the best toughness by 54% and 238% respectively when compared with the neat epoxy. The results were confirmed on nine concrete prisms and tested under a single-lap shear test which the toughness improved by 20% and 40% for CTBN and ATBN modified epoxy respectively when compared with the neat epoxy. DTMA was conducted to measure the dynamic mechanical properties of the modified epoxies, including the storage modulus and the glass transition temperature, at a range of temperatures to confirm the shear-lap results. The ATBN-modified epoxy exhibited more ductility than the CTBN-modified epoxy by . However, both modified epoxies demonstrated improvement in toughness compared with the neat epoxy.

8.2.2. Improved Behaviour of the Retrofitted Member

To investigate the effect of modified epoxy on the behaviour of the retrofitted member (with varying CFRP sheet layers), 10 RC beams were cast and tested under a four-point bending test. The maximum load capacity of the RC beam, maximum deflection, differential strain and interface slip were examined. The test beams that were strengthened with two and three CFRP layers failed at early stages. The concrete surface did not have the ability to sustain the force...
transferred from the CFRP sheets, due to the weakness of concrete in compared to the CFRP composite. The beams strengthened with one CFRP layer failed by debonding the CFRP sheet with different deflection and interface slip measurements. The RC beams using the modified epoxy showed a significant improvement in ductility behaviour compared with the same beams using the neat epoxy resin. The ductility index of the RC beam retrofitted using the ATBN-modified and CTBN-modified epoxy resins was improved up to 66% and 42% respectively, compared with beams using neat epoxy. Further, the beams using ATBN-modified epoxy recorded a higher interface slip than the beams using CTBN-modified epoxy and neat epoxy. These results confirm bonds with low shear stiffness allow interface slip, permit ductile behaviour and delay debonding failure. Moreover, the differential strain of beams using ATBN-modified epoxy was the lowest across the three series, representing a higher integrity of bond than the other two epoxy.

8.2.3. Failure Modes

Three types of failure modes occurred for the beams tested in this research. The first type was CFRP debonding failure mode, which occurred in beams A3, A4 and A5, all from the first series. The second failure mode was concrete cover delamination, which occurred for all the beams strengthened with two and three CFRP layers, regardless of the epoxy used. The third type of failure mode was the combination of concrete cover delamination and debonding CFRP layer occurred for the beam A2, which strengthened with one CFRP layer and used ATBN-modified epoxy.

The vast difference in tensile strength between the CFRP composite and concrete was one of the parameters leading to premature failure. Moreover, the compressive strength of the concrete influenced the failure mode.
8.2.4. Mathematical Model

A mathematical model has been developed to deal with the beam behaviour retrofitted with multilayered CFRP layers that allow for interlayer slip and non-linear properties. The assumed element of the composite beam was considered subject to a system of forces that satisfy equilibrium and compatibility of deformations. The interlayer slip was incorporated by relating the differential strain at the interface between CFRP layers and concrete to the longitudinal shear flow at the corresponding interface through the shear stiffness of the adhesive layer. Equilibrium and compatibility equations were solved numerically using Matlab software. Predicted beam behaviours, such as maximum load, maximum deflection, interface slip and longitudinal and differential strain, could be obtained at any node.

8.2.5. Model Verification

The mathematical model was verified with experimental results obtained from this research and previous work carried out by several researchers.

The predicted results were close to the experimental results for load-deflection, differential strain and interface slip. Moreover, the experimental work carried out by Gao et al., Obaidat, Wenwei et al. and Kotynia et al. predicted experimental results within 10% error.

8.2.6. Parametric Study

To examine the effect of varying shear stiffness of the bond on beam behaviour, a virtual beam retrofitted with different CFRP layers was investigated. The bond of lower shear stiffness showed more ductile behaviour than the bond with higher shear stiffness. The impact of varying shear stiffness of the bond on the ultimate and maximum deflection was examined. The predicted results show that changing the shear stiffness of the bond has little effect on the
ultimate load capacity of the retrofitted member. However, a lower value of bond shear stiffness tends to more deflection compared with the same retrofitted member using a higher value of the shear stiffness.

Despite improvements to the ultimate load, the ductility of the retrofitted beam significantly decreased with increasing CFRP layers. The failure mode of the beam changed from debonding to concrete delamination with increased CFRP layers.

The strain distribution of the composite beam was analysed. The longitudinal strain of the concrete and CFRP layer increased with load increment and increased from support towards the beam's midspan. Further, the differential strain between the concrete surface and the extreme CFRP layer increased with load increment until failure.

**Overall**, the retrofitted RC beam using the rubber-modified epoxy to bond a multilayer CFRP sheet exhibited more ductility and hence delayed debonding failure compared with beams using the neat epoxy. The ATBN-modified epoxy demonstrated a better combination of properties in relation to ductility and toughness than did the CTBN-modified epoxy and neat epoxy. The ductility of the retrofitted beam using ATBN-modified epoxy and CTBN-modified epoxy was improved up to 66% and 42%, respectively, compared with beams using the neat epoxy.

### 8.3. Future Recommendations

Based on the experimental results of the research, the following recommendations should be addressed in future research to ensure the safe use of CFRP sheets to externally strengthened RC beams:

a) The available commercial carbon fibre sheets were designed and fabricated for industry (non-infrastructure) applications. The modulus of elasticity for the available carbon fibre is seven times higher than the modulus of the concrete. This weakness means
concrete is susceptible to premature failure at an early stage. However, the CFRP matrix consists of two parts; carbon fibre and the epoxy resin. The epoxy resin has been studied in this research and modified it to obtain more ductile behaviour. Then, the reduce of the modulus of elasticity of carbon fibre will turn to delay the premature failure. Therefore, it is recommended that carbon fibre sheets should be designed and fabricated to have a low modulus and fibre density for infrastructure applications.

b) Further extensive research for epoxy modification is required; in particular, research investigating different types of neat epoxy and various types of hardener. Moreover, mixing weights between 20 g and 30 g of both CTBN and ATBN reactive polymers need to be extensively studied to determine the most precise mixing weight. A more accurate mixing weight will ensure a better combination of toughness and ductility for the bond agent. It predicted will be more softener and could delay failure and improve the ductility of the retrofitted member.

c) The surrounding environment sensitively affects the bond integrity. Therefore, it is recommended a further study on the influence of freezing conditions, dry and wet harsh environments such as high temperature, high humidity, and saline conditions.
9. REFERENCES


62. ACI, 440.2R-08: Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures. Farmington Hills, MI: American Concrete Institute, 2008.


