REINFORCING AND REHABILITATIVE PERFORMANCE OF BASALT FIBRE REINFORCED POLYMERS FOR CONCRETE BEAMS

Jason Philip Duic
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REINFORCING AND REHABILITATIVE PERFORMANCE OF BASALT FIBRE REINFORCED POLYMERS FOR CONCRETE BEAMS

By

Jason Duic

A Thesis
Submitted to the Faculty of Graduate Studies through the Department of Civil and Environmental Engineering in Partial Fulfillment of the Requirements for the Degree of Master of Applied Science at the University of Windsor

Windsor, Ontario, Canada

2017

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REINFORCING AND REHABILITATIVE PERFORMANCE OF BASALT FIBRE REINFORCED POLYMERS FOR CONCRETE BEAMS

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April 21, 2017
DECLARATION OF CO-AUTHORSHIP AND PREVIOUS PUBLICATION

I. Co-Authorship Declaration

I hereby declare that this thesis incorporates material that is the result of joint research undertaken in collaboration with MEDA Engineering & Technical Services and Dr. S. Kenno under the supervision of Dr. S. Das. The collaboration is covered in each of Chapters 2, 3, 4, and 5 of this thesis. In all cases, the key ideas, primary contributions, experimental designs, data analysis and interpretation, were performed by the author, and the contribution of co-authors was primarily through advice and assistance with experimental testing.

I am aware of the University of Windsor Senate Policy on Authorship and I certify that I have properly acknowledged the contribution of other researchers to my thesis, and have obtained written permission from each of the co-authors to include the above materials in my thesis.

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II. Declaration of Previous Publication

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<tr>
<td>Chapter 2</td>
<td>Duic, J., Kenno, S., and Das, S., “Comparison of Behaviour of BFRP and Steel Rebar Reinforced Concrete Beams,” <em>ACI Structural Journal</em></td>
<td>Submitted</td>
</tr>
<tr>
<td>Chapter 3</td>
<td>Duic, J., Kenno, S., and Das, S., “Flexural Rehabilitation and Strengthening of Concrete Beams with BFRP Composite,” <em>Journal of Composites for Construction, ASCE</em></td>
<td>Submitted</td>
</tr>
<tr>
<td>Chapter 4</td>
<td>Duic, J., Kenno, S., and Das, S., “Rehabilitation of Shear Deficient RC Beams with Basalt Fibre Reinforced Polymer Composite,” <em>Canadian Journal of Civil Engineering</em></td>
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ABSTRACT

This thesis presents the experimental results of laboratory testing conducted on full-scale concrete beams that are reinforced and rehabilitated with basalt fibre reinforced polymer (BFRP) products. The first study compares the structural behaviour of BFRP and steel reinforced beams. It was found that current design standards were able to predict the shear capacity of BFRP reinforced beams with varying accuracy. However, it was found that BFRP stirrups without proper bends did not prevent shear failure. Thus, proper BFRP stirrups need to be developed. The second study was on flexural strengthening and rehabilitation of concrete beams with BFRP composite. It was found that BFRP was effective in increasing or restoring service, yield, and ultimate load carrying capacity. It was also found that flexural crack widths are significantly reduced when BFRP is applied in flexure. However, interfacial debonding was still found to occur and was later corrected. The last study is one on the rehabilitation of shear deficient RC beams with BFRP composite. It was found that for the beam specimens with significant damage, the BFRP was effective in changing the mode of failure from brittle shear failure before yielding to flexural compression failure after yielding. Analysis of crack patterns with digital image correlation also revealed that the shear crack patterns were significantly changed between the damaged and rehabilitated specimen. It was also found that flexural crack widths are significantly increased in rehabilitated specimens. Thus, it is recommended that shear rehabilitation should be accompanied by flexural rehabilitation. Further research on more shear critical beams is also needed.
I dedicate this thesis to present and past engineers, scientists, researchers, inventors, and entrepreneurs who meticulously push(ed) the boundaries of human knowledge and innovation ever further and particularly to those who audaciously challenge(d) the status quo and attempt(ed) things that were once thought impossible.

♦♦♦

I have no special talent. I am only passionately curious.

~ Albert Einstein
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CHAPTER 1

INTRODUCTION

Reinforced concrete is one of the most widely used building materials for civil engineering applications. Whether it be used for bridges, buildings, tunnels, or even roads, reinforced concrete plays a critical role in the development of infrastructure around the world. For centuries, concrete has been reinforced using steel rebar, the mechanical properties of which are well known and extremely desirable. Steel has a very high elastic modulus and can sustain significant plastic deformation before rupture. These properties deliver a reasonable balance between safety and economy that has made steel reinforced concrete so widely accepted.

The basic premise of reinforced concrete is the same, regardless of how it is reinforced. Since the tensile strength of concrete is only about 10% of its compressive strength, a reinforcing material with a relatively high tensile strength is added internally to compensate for the concrete’s lack of tensile strength. Forces are transferred in shear through the concrete to the reinforcing materials and allow for force equilibrium to be achieved while the concrete section is cracked. After cracking of the section, tension forces are carried by the reinforcing material, while compressive forces are carried by the concrete. Shear forces are carried by a combination of both the reinforcing material and the concrete.

The corrosion of reinforcing steel has plagued reinforced concrete for about as long as it has existed. This issue is particularly accelerated in cold climates where deicing salts are used heavily. By recent estimates, one out of every nine bridges in the United States is structurally deficient, representing a $121 billion backlog of spending necessary to repair
the crumbling infrastructure [1]. The issue with corroded reinforcing steel is threefold. First, the loss of steel cross-section reduces the tensile capacity of the rebar, thus reducing the strength of the concrete element it is reinforcing. Secondly, the volume of the iron oxide is larger than that of the steel, which induces extra tensile stresses in the concrete, causing the concrete to spall and fall away from the structure, further exposing the reinforcing steel to chemical attack from deicing salts. Lastly, the corrosion of the reinforcement weakens the bond between the steel and concrete. Thus, there is a significant demand to develop new innovative reinforcing materials with good mechanical properties, but more importantly, that solve the issue of corrosion.

1.1 FIBRE-REINFORCED POLYMERS

Fibre-reinforced polymers (FRPs) have been proposed to solve the corrosion problem since the 1950s, with applications being studied since the 1970s [2]. FRPs are materials consisting of two components: namely, continuous fibres and a polymer matrix. FRPs are chemically inert and resistant to corrosion, making them ideal materials to solve the significant issue of corrosion of reinforcing steel.

1.1.1 Fibres

The most common fibres used for structural FRPs are carbon, glass, and aramids. To make the fibres, each material is melted and drawn into continuous fibres with a specific shape and size. The fibres must be treated with a coupling agent to give adequate bond between the fibres and the matrix [2]. The fibres are significantly stronger and stiffer than the matrix and typically provide more than 55% of the volume for FRP rebars [2]. Figure 1.1 shows the general relationship between the mechanical properties of the two FRP constituents relative to those of the FRP. As can be seen, the fibres are of higher
strength and stiffness than the FRP, typically ranging in strength from about 1800 to 4900 MPa. Similar to the FRP, the fibres remain linear until rupture.

![Figure 1.1: Mechanical properties of FRP constituents [2]](image)

1.1.2 Matrix

The matrix is the material through which stresses are transferred between the fibres within the FRP composite through in-plane shear. Selecting the appropriate matrix is critical in developing the desired mechanical and durability properties of the FRP. In order to best utilize the full strength of the fibres, the matrix should have a higher ultimate strain than that of the fibres [2]. There are two types of matrices that may be used: thermosetting and thermoplastic. Thermosetting matrices are used more often and cannot be reheated to change the shape of the FRP product. Unlike thermosetting matrices, a thermoplastic matrix can be reheated to reshape the FRP material, however typically at the expense of less desirable mechanical properties.

1.1.3 Manufacturing

FRP materials are typically manufactured using pultrusion, braiding, and filament winding techniques. Pultrusion involves pulling the fibres through a resin tank and then
through a heated die where they are shaped and cured. Braiding involves combining multiple fibres to create a single cohesive material. Filament winding is a process where fibres are wrapped around a mandrel and impregnated with epoxy.

1.1.4 Mechanical Properties

Unlike reinforcing steel, FRPs are linear elastic materials. This means that they do not yield or undergo any plastic deformation, nor exhibit significant strain at rupture relative to reinforcing steel. The typical constitutive relationships for various types of FRPs are shown in Figure 1.2. As can be seen, carbon FRP (CFRP) has the highest elastic modulus, highest strength, and lowest ultimate strain of any of the types of FRP. Glass FRPs (GFRPs) have a much lower strength and lower elastic stiffness than CFRP. However, they have a much higher ultimate strain. Basalt FRP (BFRP), which will be discussed in more detail in the next section, has mechanical properties close to those of GFRP. As can be seen in Figure 1.2, however, all FRPs have significantly lower rupture strain than that of reinforcing steel.

![Figure 1.2: FRP and steel constitutive relationships [2]](image-url)
The individual mechanical properties of each type of FRPs and reinforcing steel are tabulated in Table 1.1. As can be seen, the moduli of GFRP and AFRP are relatively similar, ranging from about 30 to 75 GPa. The elastic modulus of CFRP typically ranges between 150 to 175 GPa, which is similar to that of reinforcing steel. One significant drawback for CFRP is that its rupture strain is about 1%, which is half of that of GFRP and AFRPs. As will be seen later in this thesis, the elastic moduli of reinforcing materials play a critical role in determining the structural behavior of the concrete elements they reinforce.

**Table 1.1**: Mechanical properties of reinforcing steel and FRPs [2]

<table>
<thead>
<tr>
<th>Mechanical Properties</th>
<th>Reinforcing Steel</th>
<th>CFRP</th>
<th>GFRP</th>
<th>AFRP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic Modulus (GPa)</td>
<td>200</td>
<td>150-175</td>
<td>30-50</td>
<td>50-75</td>
</tr>
<tr>
<td>Tensile Strength (MPa)</td>
<td>400-500</td>
<td>1600-2400</td>
<td>500-1000</td>
<td>1200-2000</td>
</tr>
<tr>
<td>Yield Strain (%)</td>
<td>0.2</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Ultimate Strain (%)</td>
<td>~30</td>
<td>1-1.5</td>
<td>1.5-2.0</td>
<td>2-2.6</td>
</tr>
</tbody>
</table>

**1.1.5 Durability**

There are a number of durability concerns that are associated with the use of FRPs: exposure to high temperature, galvanic corrosion, ultraviolet light, and alkalinity [2]. Exposure to high temperature, particularly when it approaches the glass transition temperature of the matrix, has a significant negative effect on the strength of FRP products, especially for thermoplastic matrices. Thus, it is important to ensure that FRPs are not exposed to extremely high temperature. This issue could be significant for the design of fire resistant structures. Galvanic corrosion is of particular concern only for CFRP. When carbon comes into contact with steel, an electric current is generated, and causes corrosion of the materials. Thus, it is important to ensure CFRPs do not come into contact with steel during construction. Of lesser concern, but still noteworthy, are the issues of exposure to ultraviolet light and alkalinity. It has been found that exposure to these elements causes a slight decrease in strength of the FRP materials over time [2-3]. Thus, special care should
be taken to consider these effects in the design and construction of FRP reinforced concrete structures.

1.2 BASALT FIBRE-REINFORCED POLYMERS

Basalt is a naturally occurring igneous rock. The rock is quarried, melted and extruded into continuous fibres. Plain basalt fibres can be made into chopped fibres or fabrics (Figure 1.3a and 1.3b). When the fibres are then combined with an epoxy matrix, they can be made into basalt rebars and meshes (Figure 1.3c and 1.3d). Basalt fibres themselves have been shown to perform better than glass and carbon fibres in accelerated weathering and temperature testing [4].

![Figure 1.3: Various basalt fibre products](image)

(a) Fibres  
(b) Fabric  
(c) Rebar  
(d) Mesh
BFRP rebars have an ultimate strength of about 1000 MPa, an elastic modulus of about 50 GPa, and a rupture strain of about 2%, making them most similar to GFRP. BFRP rebars have been shown to exhibit good bond strength [5]. They have also been shown to exhibit good strength retention when subjected to accelerated weathering due to heat and alkali exposure [3].

1.2.1 Cost

The cost of basalt fibre reinforced polymers have the potential to become less than that of other types of FRPs since basalt rock is plentiful, can be extracted easily, and requires less energy for production [6]. Currently, 10 mm basalt rebar can be obtained on the market for about $2.70/m. The cost of typical 10M black steel rebar is about $2.50/m. Thus, the cost of steel rebar and BFRP rebar are relatively similar. The cost of basalt unidirectional fabric on the market is about $8.50/m². As the demand for basalt fibre products increases, it should be expected that the cost will decrease.

1.3 SUMMARY OF THE LITERATURE REVIEW

1.3.1 BFRP Reinforced Concrete

Many studies have been conducted on the use of FRP as a reinforcing material for concrete structural elements. The most common types of FRPs that have been researched for this purpose are GFRP and CFRP. These two types of FRPs have shown successful application as reinforcing materials in numerous studies [7-10]. BFRP has not been researched as extensively. However, studies by Tomlinson and Fam (2014) [11], Ovitigala et al. (2016) [12], and Brik (2003) [13] have shown its successful application as flexural reinforcement. Issa et al. (2015) [14] has also shown that BFRP can be successfully used as shear reinforcement. Among these limited studies, no study has been conducted that
compares the structural behaviour of BFRP reinforced concrete to that of traditional steel reinforced concrete. The previous studies also used BFRP rebars that were 8 mm or less in diameter, thereby ignoring the scale factor. Thus, there is a need to study the structural behaviour of BFRP reinforced concrete made with larger diameter bars and compare it to the behaviour of similar steel reinforced concrete beams.

1.3.2 BFRP Flexural Rehabilitation and Strengthening

Only two studies by Sim et al. (2003) [4] and Lihua et al. (2013) [6] have been conducted on the use of BFRP as a flexural strengthening material. Both studies prove the effectiveness of BFRP in strengthening reinforced concrete beams. Both studies found that flexural strengthening of reinforced concrete beams can increase the yield and ultimate load capacity of the beams. They both also found that interfacial debonding failure can occur. Lihua et al. (2013) [6] also studied the use of GFRP and CFRP and compared the results to that of BFRP and noted that the performance of BFRP lies somewhere between that of CFRP and GFRP. Numerous studies have been conducted on flexural strengthening of reinforced concrete beams with GFRP and CFRP. Green et al. (2003) [15] and Attari et al. (2012) [16] studied and successfully demonstrated the application of both materials for flexural strengthening. Al-Saidy and Al-Jabri (2011) [17] also studied the effect of CFRP on damaged concrete beams. Due to the lack of research on the use of BFRP composites for flexural strengthening, there is a need to further study the feasibility of this material for both rehabilitation and strengthening of reinforced concrete beams.

1.3.3 BFRP Shear Rehabilitation

No studies have yet been conducted that use BFRP for either shear strengthening or rehabilitation of reinforced concrete beams. However, similar to studies of flexural strengthening, the use of GFRP and CFRP has been studied for shear strengthening of
concrete beams. Chaallal et al. (1998) [18] studied the use of CFRP and found that CFRP strips were able to increase the shear strength, reduce shear cracking, and increase the ductility of RC beams. This study also examined the effect of placing the strips diagonal to the longitudinal axis and found that the diagonal scheme slightly outperformed systems where the fibres were oriented perpendicular to the longitudinal axis. Baggio et al. (2014) [19] studied the use of both GFRP and CFRP and found that both materials were effective in increasing the shear capacity of shear critical beams. However, they also found that beams strengthened with GFRP and which were not provided adequate anchorage failed by debonding. Taljsten and Elfgren (1999) [20] also studied the use of CFRP and found that it was effective in increasing shear capacity. However, the study also found that CFRP strengthened specimens can still experience brittle shear failure. Given the lack of studies on the use of BFRP for shear rehabilitation or strengthening of reinforced concrete beams, there is a need to examine the effectiveness of this material for shear strengthening.

1.4 OBJECTIVE

The purpose of this thesis is to study a new, innovative, and economical material that can help address the corrosion issue facing traditional steel-reinforced concrete structures. This thesis will evaluate the use of basalt fibre reinforced polymers both as a reinforcing material in the form of rebar, and as a strengthening and rehabilitation material in the form of externally bonded composite to solve the corrosion problem facing steel-reinforced concrete structures. These materials will be applied to full scale beam specimens and tested in the structural engineering laboratory at the University of Windsor. This thesis will also study whether existing design standards accurately predict the capacity and behaviour of BFRP reinforced, strengthened, and rehabilitated concrete beams.
1.5 METHODOLOGY

The research methodology used in this thesis consists of full scale laboratory testing, material testing, and data analysis to characterize the behaviour of BFRP reinforced, strengthened, and rehabilitated beams. The specific experimental procedure for each study is detailed within each of Chapters 2, 3, and 4.

1.6 ORGANIZATION OF THE THESIS

This thesis is written in manuscript format and is divided into six chapters as follows:

Chapter 1 serves as a general introduction to the topic of the thesis, and explains the research problem, objectives of the study, and research methodology used.

Chapter 2 is a study on the use of BFRP rebars for reinforcing concrete beams. This study compares the structural behaviour of beams reinforced with BFRP rebar to beams reinforced with traditional steel rebar.

Chapter 3 is a study on the use of externally bonded BFRP composite for strengthening and rehabilitating damaged concrete beams in flexure. The study compares the behaviour of strengthened, unstrengthened, damaged, and rehabilitated beams.

Chapter 4 is a study on the use of externally bonded BFRP composite for rehabilitating damaged concrete beams in shear. This study compares the behavior of damaged and rehabilitated beams specimens.

Chapter 5 is a field study on the application of externally bonded BFRP composite for rehabilitation of a local bridge structure.

Chapter 6 draws conclusions from the study and makes recommendations for future work based on the results.
1.7 REFERENCES


CHAPTER 2
COMPARISON OF BEHAVIOUR OF BFRP AND STEEL REBAR REINFORCED CONCRETE BEAMS

2.1 INTRODUCTION

Corrosion of steel rebar is inevitable in traditional steel-reinforced concrete structures. With the heavy use of deicing salt in cold climates, this problem is a more serious concern for durability of steel rebar reinforced concrete (RC) structures and structural elements. Hence, the use of fibre reinforced polymer (FRP) rebar as an alternative reinforcement has been gaining popularity in addressing this issue. FRP rebars are corrosion resistant and chemically inert. Presently, there are three types of FRP rebar recommended by design standards: carbon fibre reinforced polymer (CFRP), glass fibre-reinforced polymer (GFRP), and aramid fibre reinforced polymer (AFRP) rebars. Each type of FRP rebar has its advantages and disadvantages in terms of its mechanical properties, durability properties, and cost. Among the three, GFRP rebar is probably the most popular choice for field applications due to its relatively low cost with respect to CFRP and AFRP rebars. In recent years, various forms of products made of basalt fibres such as basalt fibre reinforced polymer (BFRP) rebar, fabrics, meshes, and chopped fibres (Figure 2.1) have been made available for various civil engineering applications. Basalt fibres are made of volcanic rock called basalt and hence, BFRP products are a greener alternative than other FRP products.
Sim et al. (2005) [1] conducted mechanical and durability tests on basalt fibres and compared them to glass and carbon fibres. The study found that basalt fibre performed better than both glass and carbon fibres in accelerated weathering and temperature testing. BFRP rebars have been shown to have an ultimate strength of about twice that of conventional reinforcing steel rebar. Serbescu et al. (2015) [2] studied the effect of weathering on BFRP rebars and found that they exhibit good strength retention in accelerated weathering conditions of heat and alkalinity. Bond durability has been shown to be excellent among BFRP rebars, and showed higher bond strength than GFRP rebar [3]. Nonetheless, all three FRPs show excellent resistance to electrochemical corrosion.

Many studies have been conducted on FRP reinforced concrete, with much of the research focused on the applications of CFRP and GFRP rebars. FRP rebar has demonstrated successful application as both flexural and shear reinforcement in various reinforced concrete structural elements including RC beams [4-7]. However, only very limited research has been conducted on the feasibility of BFRP rebar as a reinforcing
material to replace traditional steel rebar. Recent studies have shown that BFRP reinforced concrete beams with sufficient shear resistance can undergo a flexural mode of failure, and the failure is often initiated by crushing of concrete [8-10]. Both ACI 440.1R-15 [11] and CSA 806-12 [12] specify that FRP reinforced elements should fail by crushing of concrete in flexure. Beams can be made to fail in a flexural tension manner initiated by rupture of the longitudinal BFRP bars if the reinforcement ratio is sufficiently low [9, 10]. However, when insufficient shear reinforcement is provided, BFRP reinforced concrete beams can undergo shear failure instead of flexural failure [9].

Tomlinson and Fam (2014) [9] and Issa et al. (2015) [13] found that even if bent BFRP shear reinforcement was provided, shear failure still occurred due to rupture of the BFRP bars at the bend. Thus, shear failure is still a problem that can govern the design of BFRP reinforced concrete beams. Additionally, many types of FRPs, including BFRP, are manufactured with thermosetting resins, and thus, cannot be reheated and bent to the desired shape, further limiting the use of BFRP as shear reinforcement [14]. Hence, Tomlinson and Fam (2014) [9] and Ovitigala et al. (2015) [8] used steel stirrups in some of their specimens to avoid shear failure and to ensure flexural failure. Thus, this did not solve the problem of shear reinforcement made of BFRP rebar.

Bentz et al. (2010) [5] studied the effect of reinforcement ratio on large GFRP reinforced concrete members. The study concluded that the behaviour is similar to that of steel reinforced concrete beams. It is well-known, however, that bent FRP reinforcement tends to be dramatically weaker at the bend due to stress concentrations [14]. This weakness has been shown to be as high as 54% of the ultimate strength. In line with this, ACI
440.1R-15 [11] requires that FRP stirrup strength be reduced using the factor: $0.05r_b / d_b + 0.3$.

Though a few studies were undertaken to understand the behaviour of BFRP reinforced RC beams, none of these studies compared the behaviour of BFRP RC beams with the behaviour of steel rebar RC beams. Further, previous researchers used 8 mm or lesser diameter BFRP rebars as flexural reinforcement. Hence, in these studies, the scale factor was ignored. Therefore, the current study was designed carefully to eliminate scale factor induced error and to determine the behaviour of BFRP reinforced concrete beams and compare that with similar steel reinforced concrete beams. The research was completed using experimental methods.

2.2 EXPERIMENTAL PROCEDURE

2.2.1 Test Specimens

This study consisted of eight full-scale RC beam specimens as shown in Table 2.1. The beam specimens were 275 mm wide, 500 mm deep, and 3200 mm long and made with concrete that had a target strength of 35 MPa. Ready mix concrete from a local supplier was used to cast the beam specimens. Table 2.1 presents the different specimens tested and parameters studied. As shown in the table, the test specimens consist of steel and BFRP rebar reinforced beams. The test parameters studied were: two different reinforcement materials, two flexural reinforcement ratios, and the presence or absence of shear reinforcement. The naming of the beam specimens is intended to reflect their main attributes. The first letter of the name indicates if the beam specimen was made of steel rebar (S) or BFRP rebar (B). The next one is a number which represents the reinforcement ratio (0.41% and 0.83%). The last letter represents if the beam specimen had shear
reinforcement (Y) or not (N). Hence, specimen S41Y is a RC beam specimen made of steel rebar (S) with reinforcement ratio of 0.41% and this beam specimen had shear reinforcement (Y).

The beam specimens in Table 2.1 are divided in two phases, namely I and II. Four beam specimens were built and tested in each phase. Flexural reinforcement ratios in these two phases are different. The reinforcement ratios of the Phase I and II beam specimens were 0.41% and 0.83%, respectively, producing sections having reinforcement ratios approximately equal to and twice the FRP balanced reinforcement ratio [11-12, 15-16], respectively. Stirrups were removed from two specimens in each phase to determine the concrete contribution ($V_c$) to the total shear resistance ($V_r$). The individual material properties for each beam are also summarized in the table.

Beam specimens without shear reinforcement were constructed with just four stirrups outside of the shear span to hold the rebar cages together. The rebar cages were tied using traditional steel ties for steel cages and plastic cable ties for BFRP cages. BFRP stirrups were cut as single straight legs with no bends or hooks and were offset in the longitudinal direction to make a square shape as can be seen in Figures 2.2 and 2.3. Figure 2.3 shows the cross section of the beams with shear reinforcement. A clear cover of 30 mm was used. Vertical spacing of 30 mm was provided between layers of longitudinal rebar. Figure 2.4 shows the elevation of the beam specimens constructed with and without stirrups. The beams were cast in a lab setting, and allowed to cure in room temperature for a minimum of 28 days before testing.
Table 2.1: Test matrix

<table>
<thead>
<tr>
<th>Phase</th>
<th>Specimen ID</th>
<th>$\rho$ (%)</th>
<th>$\rho_b$ (%)</th>
<th>$\rho/\rho_b$</th>
<th>Longitudinal Rebar</th>
<th>Stirrup</th>
<th>$f'_c$ (MPa)</th>
<th>$E_f$ or $E_s$ (GPa)</th>
<th>$f_y$ or $f_{fu}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>S10Y</td>
<td>0.41</td>
<td>3.35</td>
<td>0.12</td>
<td>10M Steel</td>
<td>Bent Steel</td>
<td>41</td>
<td>200</td>
<td>440</td>
</tr>
<tr>
<td></td>
<td>S10N</td>
<td></td>
<td>(3.79)*</td>
<td>(0.11)*</td>
<td></td>
<td>-</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>B12Y</td>
<td>0.38</td>
<td>1.08</td>
<td>1.08</td>
<td>12 mm BFRP</td>
<td>Straight BFRP</td>
<td>38</td>
<td>54</td>
<td>943</td>
</tr>
<tr>
<td></td>
<td>B12N</td>
<td></td>
<td>(0.45)†</td>
<td>(0.91)†</td>
<td></td>
<td>-</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>II</td>
<td>S15Y</td>
<td>0.83</td>
<td>3.35</td>
<td>0.24</td>
<td>15M Steel</td>
<td>Bent Steel</td>
<td>41</td>
<td>200</td>
<td>430</td>
</tr>
<tr>
<td></td>
<td>S15N</td>
<td></td>
<td>(3.79)*</td>
<td>(0.22)*</td>
<td></td>
<td>-</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>B14Y</td>
<td>0.38</td>
<td>2.18</td>
<td>2.18</td>
<td>14 mm BFRP</td>
<td>Straight BFRP</td>
<td>41</td>
<td>51</td>
<td>986</td>
</tr>
<tr>
<td></td>
<td>B14N</td>
<td></td>
<td>(0.47)†</td>
<td>(1.77)†</td>
<td></td>
<td>-</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*ACI 318-14 [15] (CSA A23.3-14) [16]
†ACI 440.1R-15 [11] (CSA S806-12) [12]
1 mm = 0.039 in
1 MPa = 0.15 ksi
2.2.1.1 Material Properties

Tensile properties of the BFRP rebar were determined in accordance with ASTM D7205-11 [17]. The digital image correlation (DIC) technique was used to measure the strain over approximately 100 mm gauge length, as required by the standard. VIC-2D software [18] was used to determine the strain in the specimen (Figure 2.5). The virtual extensometer showed a strain of approximately 0.022 (2.2%) prior to rupture. Table 2.2 shows a summary of the tensile properties of the BFRP rebar used for this investigation.
Table 2.2: BFRP rebar tensile properties

<table>
<thead>
<tr>
<th>Bar size (mm)</th>
<th>Ultimate Load (kN)</th>
<th>Ultimate Stress, $f_{fu}$ (MPa)</th>
<th>Ultimate Strain, $\varepsilon_{fu}$ (%)</th>
<th>Modulus of Elasticity, $E_f$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>107</td>
<td>943</td>
<td>1.67</td>
<td>54</td>
</tr>
<tr>
<td>15</td>
<td>152</td>
<td>986</td>
<td>1.96</td>
<td>51</td>
</tr>
</tbody>
</table>

1 mm = 0.039 inches
1 kN = 0.22 kip
1 MPa = 0.15 ksi

Figure 2.5: BFRP rebar tension test

Figure 2.6: BFRP and steel rebar constitutive relationships
The steel used for this study was 400 grade. Tensile tests on the steel reinforcement were conducted in accordance with ASTM A370-14 [19]. The measured yield strengths are shown in Table 2.1, alongside the FRP rupture strengths for BFRP reinforced beams. Figure 2.6 shows a typical stress-strain plot for the 10 mm (0.4 in) BFRP and 10M steel rebar used for this study. ASTM C39-15 [20] was followed to determine the concrete compressive strengths (Table 2.1).

### 2.2.2 Test Procedure and Instrumentation

Each beam specimen was simply supported and tested in 4-point bending as shown in Figure 2.7. The beam spanned 3000 mm (118.1 in) and was supported by a roller between two plates at one end and a knife edge between two plates at the other end. Bending load was applied to the top of the beam through the steel spreader beam creating a constant moment region of 1000 mm (39.4 in). Load was measured using three load cells: one attached to the loading actuator, and two on the bottom under the supports. Displacements were measured using four linear variable differential transformers (LVDTs). One LVDT was with the loading actuator and thus, it measured the vertical displacement at the mid-span from the top of the beam specimen. Three other LVDTs were placed underneath the beam and measured the vertical displacement of the beam at the quarter, half, and three quarter points along the span (Figure 2.7).
Strain gauges were placed on the rebars at the mid-span on the longitudinal tension rebars, as well as on the stirrups in the shear span on each side. Figure 2.4 shows the locations of the strain gauges placed on both layers of tension rebars ($\varepsilon_T$) and on the stirrups ($\varepsilon_s$). All test data was acquired through a computerised data acquisition system. The beam specimens were loaded using displacement control. Loading was continued until either shear failure or flexural compression failure was observed.

### 2.3 TEST RESULTS AND DISCUSSION

This section discusses the results of this investigation in terms of the crack pattern, load-deflection response, deformability, load-strain response, ultimate capacity, and mode of failure. The effect of steel versus BFRP shear and flexural reinforcement at two different reinforcement ratios are characterised and discussed in terms of these parameters.
2.3.1 Crack Pattern

During loading, flexural cracks began to form in the extreme tension fibres of the concrete in the constant moment region, and propagated up towards the compression face as the load increased. As the load increased further, shear cracks began to form in both shear spans and propagated from the bottom face of the beam diagonally up towards the top supports. Figure 2.8 shows the crack patterns and shear crack angles for Phase I and II beams. Shear crack angles were noted only for beams that failed in shear. Among Phase I beams, it is clear that beams reinforced with BFRP rebar (B-series) experienced a higher number of flexural tensile cracks than the steel rebar reinforced beams (S-series), possibly due to the low stiffness of BFRP rebar. The cracks in B-series beams showed significant branching near the location of the reinforcement (bottom third of the beams). However, the number and spacing of flexural cracks above the mid-depth of the beam in the constant moment region were similar in both B- and S-series beams. Cracking in the shear span was also observed among all Phase I beams. Shear crack angles varied from 47 to 57 degrees among B-series beams, whereas cracks in the shear span in the S-series beams with stirrups (S41Y) were much steeper. B-series beams also experienced significantly larger number of shear cracks than the S-series beams and the shear cracks in the BFRP beams spread closer to the end support.

Among Phase II beams, similar patterns were observed to that of the Phase I beams. Flexural cracks formed in the constant moment region and the number of cracks in all the beams was similar. However, the B-series beams in Phase II did not exhibit the significant branching in the lower third of the beam, perhaps due to the increased reinforcement ratio. The shear crack angle in the B83N specimen was slightly steeper than that of the S83N specimen. However, specimen B83Y experienced a much steeper shear crack angle. The
number of shear cracks in the two BFRP reinforced specimens (B83Y and B83N) was less than the similar S-series specimens (S83Y and S83N). The B-series specimens exhibited shear cracking closer to the supports.

Figure 2.8: Beam crack patterns
2.3.2 Load-Deflection Behaviour

The load versus mid-span deflection response of Phase I and II beams are show in Figures 2.9a and 2.9b, respectively. A distinct difference in the shape of the load-displacement plots for S- and B-series beams exists. Both S- and B-series beams exhibited similar pre-cracking load-deflection behaviour. However, post-cracking load-deflection behaviours of these two beams were notably different. As can be observed in Figure 2.9, after cracking, the stiffness in load-deflection behaviour of S-series beams did not reduce much whereas the stiffness in the B-series beams reduced considerably. Among Phase I B-series beams, the ratio of post-cracking stiffness to pre-cracking stiffness in the load-deformation curve are 4.9 and 8.2 for B41N and B41Y, respectively. However, both B-series beams of Phase II exhibited a ratio of pre-cracking stiffness to post-cracking stiffness of 4.8. Hence, this study shows that stirrups in low flexural reinforcement ratio BFRP beams are effective in increasing the post-cracking stiffness.

However, after cracking, both B-series beams remained linear, while S-series beams remained linear until the steel rebars yielded. As expected, yielding of the reinforcement in S-series beams caused a plateau in the load carrying capacity and only marginal load increase occurred thereafter until flexural compression failure occurred. In Figure 2.9, the unloading path is shown only for specimens that experienced a flexural compression mode of failure. The slope of the unloading curve for S-series beams that failed in flexure was similar to that of the elastic loading path. The unloading path for beam specimens that failed in shear are not shown because the unloading path for these beams showed a sudden large drop in load carrying capacity.

Both S-series beams in Phase I exhibited larger displacement at failure than the two B-series beams. However, beam B41Y of the B-series beams held approximately 50% more
load than its S-series counterpart beam, S41Y. Specimen S83Y was the only beam in Phase II to sustain large mid-span deflection at failure. The mid-span deflection of B83N in Phase II was approximately 20% higher than specimen S83N. However, the beam B83Y showed only about half of the maximum deflection that beam S41Y exhibited.
Table 2.3 shows the service loads, $P_{\text{service}}$, for each beam. The service load in this study is defined as the least of the loads calculated using four different criteria: (i) mid-span deflection of L/360 [12, 16], (ii) mid-span deflection of L/180 [12, 16], (iii) the service

**Figure 2.9:** Load-displacement plots

2.3.3 Service Load

Table 2.3 shows the service loads, $P_{\text{service}}$, for each beam. The service load in this study is defined as the least of the loads calculated using four different criteria: (i) mid-span deflection of L/360 [12, 16], (ii) mid-span deflection of L/180 [12, 16], (iii) the service
strain in steel/FRP [21], and (iv) the maximum sustained load divided by the load factor 1.5 [9]. For all beams, the service strain criterion governed since it produced the most conservative service load. Service loads thus calculated for the B-series beams of both phases ranged from 30 to 60% less than those of the S-series beams. This is due to the fact that the stiffness of the BFRP rebar is about one quarter the stiffness of steel rebar. Thus, the stress in the BFRP bar is about 50,000 \times 0.002 = 100 \text{ MPa} (14.5 \text{ ksi}), whereas the stress in a steel bar would be 200,000 \times 0.0012 = 240 \text{ MPa} (34.8 \text{ ksi}). However, if the $P_{\text{max}}/1.5$ criterion is applied, the service loads in some B-series beams of Phase I would exceed those of their S-series counterparts. However, this trend is reversed for Phase II beams and the B-series beams would have service loads of about 25 to 50% less than that of their S-series counterparts.

**Table 2.3: Service loads**

<table>
<thead>
<tr>
<th>Phase</th>
<th>Specimen ID</th>
<th>$P_{\text{service}}$ at L/360 (kN)</th>
<th>$P_{\text{service}}$ at L/180 (kN)</th>
<th>$P_{\text{service}}$ at $\varepsilon_{\text{service}}$ (kN)</th>
<th>$P_{\text{service}}$ at $P_{\text{max}}/1.5$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>S41Y</td>
<td>188.5</td>
<td>206.2</td>
<td>123.4</td>
<td>164.7</td>
</tr>
<tr>
<td></td>
<td>S41N</td>
<td>190.9</td>
<td>202.3</td>
<td>105.8</td>
<td>153.9</td>
</tr>
<tr>
<td></td>
<td>B41Y</td>
<td>116.1</td>
<td>166.7</td>
<td>90.0</td>
<td>200.7</td>
</tr>
<tr>
<td></td>
<td>B41N</td>
<td>94.8</td>
<td>130.3</td>
<td>72.1</td>
<td>106.3</td>
</tr>
<tr>
<td>II</td>
<td>S83Y</td>
<td>295.0</td>
<td>368.9</td>
<td>192.8</td>
<td>264.9</td>
</tr>
<tr>
<td></td>
<td>S83N</td>
<td>286.0</td>
<td>373.2</td>
<td>204.6</td>
<td>265.5</td>
</tr>
<tr>
<td></td>
<td>B83Y</td>
<td>155.7</td>
<td>201.1</td>
<td>114.2</td>
<td>201.8</td>
</tr>
<tr>
<td></td>
<td>B83N</td>
<td>141.7</td>
<td>191.7</td>
<td>110.6</td>
<td>192.0</td>
</tr>
</tbody>
</table>

$L/360 = 8.3 \text{ mm (0.33 in)}, L/180 = 16.7 \text{ mm (0.66 in)}$

$\varepsilon_{\text{service}} = 0.0012$ for steel, 0.002 for BFRP

1 kN = 0.22 kip

### 2.3.4 Deformability

The fundamental mechanical difference between reinforcing steel and FRP rebar is that FRP rebar does not exhibit yielding, nor a large amount of ductility or energy absorption prior to rupture. In light of this, the Canadian Highway Bridge Design Code [22]
requires that concrete rectangular flexural elements reinforced with FRP satisfy the following requirement.

\[ J = \frac{M_{\text{ult}} \psi_{\text{ult}}}{M_c \psi_c} \geq 4.0 \]  \hspace{1cm} (2.1)

where,

\( M_{\text{ult}} \) = the ultimate moment capacity of the section

\( \psi_{\text{ult}} \) = the curvature at \( M_{\text{ult}} \)

\( M_c \) = the moment corresponding to a maximum concrete compressive strain of 0.001

\( \psi_c \) = the curvature at \( M_c \)

The term \( M_c \psi_c \) in Equation 2.1 is simply the product of moment and curvature at service. In lieu of using a strain of 0.001 in concrete, the strain at which concrete is assumed to begin nonlinearity, a service strain of 0.002 for FRP rebar and 0.0012 for steel rebar, recommended by Newhook et al. (2002) [21], was used to compute the J-factor in this study. Curvature was calculated from the LVDT data. Table 2.4 shows the summary of the J-factors for all beam specimens.

The J-factors for S41Y and B41Y are relatively close, due to the increased ultimate moment capacity (by 20%) achieved by specimen B41Y, despite having less deflection at failure (see Figure 2.9a). Service deflection for specimen B41Y is also about 30% more than that of S41Y (see Table 2.4). The J-factor of specimen S41N is about 2.6 times higher than that of B41N, primarily due to the substantially lower ultimate deflection and lower ultimate load of specimen B41N. The J-factor of S83Y is about 20% higher than that of B83Y due to both a marginally higher load and ultimate deflection capacities exhibited by
specimen B83Y. The J-factor for S83N is about 50% less than that of B83N, mostly due to the lower load capacity at service.

<table>
<thead>
<tr>
<th>Phase</th>
<th>Specimen ID</th>
<th>$P_c$ (kN)</th>
<th>$P_{ult}$ (kN)</th>
<th>$\Delta_c$ (mm)</th>
<th>$\Delta_{ult}$ (mm)</th>
<th>J-factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>S41Y</td>
<td>123.4</td>
<td>247.1</td>
<td>3.4</td>
<td>61.2</td>
<td>36.0</td>
</tr>
<tr>
<td>I</td>
<td>S41N</td>
<td>105.8</td>
<td>230.8</td>
<td>2.9</td>
<td>50.6</td>
<td>38.0</td>
</tr>
<tr>
<td></td>
<td>B41Y</td>
<td>90.0</td>
<td>301.0</td>
<td>4.5</td>
<td>39.0</td>
<td>29.0</td>
</tr>
<tr>
<td></td>
<td>B41N</td>
<td>72.1</td>
<td>159.5</td>
<td>3.6</td>
<td>24.1</td>
<td>14.8</td>
</tr>
<tr>
<td></td>
<td>S83Y</td>
<td>192.8</td>
<td>397.4</td>
<td>4.8</td>
<td>55.3</td>
<td>23.7</td>
</tr>
<tr>
<td>II</td>
<td>S83N</td>
<td>204.6</td>
<td>398.2</td>
<td>4.7</td>
<td>25.5</td>
<td>10.6</td>
</tr>
<tr>
<td></td>
<td>B83Y</td>
<td>114.2</td>
<td>302.7</td>
<td>4.6</td>
<td>31.1</td>
<td>17.9</td>
</tr>
<tr>
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<td>110.6</td>
<td>288.0</td>
<td>4.8</td>
<td>31.3</td>
<td>17.0</td>
</tr>
</tbody>
</table>

$1 \text{kN} = 0.22 \text{kip}$

$1 \text{mm} = 0.039 \text{in}$

2.3.5 Load-Strain Response

The load-strain response of strain gauges placed on the tension and shear reinforcement are shown in Figure 2.10. Both tension and shear steel reinforcements behaved similarly. As can be found in these figures, both tension and shear strain gauges became noticeably engaged in tension around the load that first initiated cracking of the section. However, the strain values obtained from BFRP reinforcement increased at a much faster rate than the strain values obtained from steel reinforcement. After cracking initiated, the strain in B-series tension reinforcement suddenly increased without any increase in the load. This increase was more pronounced in Phase I beams, which agrees with the findings by Issa et al. (2015) [13]. The tension strain for specimen B41N is not shown since this strain gauge did not function.
2.3.6 Ultimate Capacity and Mode of Failure

Table 2.5 summarizes the ultimate moment and shear capacities of the beam specimens and the modes of failure.

2.3.6.1 Cracking Moments and Ultimate Moment Capacity

As can be found in Table 2.5, the experimental (shown by E in the table) cracking moments of all beam specimens ranged between 40 to 60 kN-m (29.5 to 60.0 kip-ft), since it depends primarily on the gross concrete section, and not the presence of reinforcing bars. Cracking moments from the tests were determined by the load at first crack, or where a noticeable increase in strain was observed in the tensile reinforcement, as can be seen in

![Figure 2.10: Load-strain response](image)
Figures 2.10ai and 2.10bi for Phases I and II, respectively. Distinct increases in the strain are noticeable at a load of approximately 100 kN (22.5 kip). In both Phase I and II specimens, the cracking moments of the S-series beams are 30 to 50% higher than those of B-series beams. This is due to the additional contribution of the rebar to the gross cross-sectional inertia and the difference in stiffness between the two types of rebar. Thus, the test data shows that the contributions of the rebar area and rebar stiffness influence the cracking moment which is currently ignored by the design standards.

The experimental ultimate moment of each beam is also presented in Table 2.5. All theoretical calculations (shown by T in Table 2.5) were performed using CSA A23.3-14 [16] and CSA S806-12 [12], but setting any material resistance factors equal to one. Ultimate moment capacities of steel reinforced beams obtained from the tests are in good agreement with theoretical moment capacities, with theoretical values being slightly conservative. Theoretical ultimate moment capacities of B-series beams ranged between 1.5 to 3 times greater than those obtained experimentally. This is since the BFRP beams did not experience a flexural mode of failure, but rather these specimens failed in shear before achieving maximum moment capacity. CSA S806-12 [12] also requires that beams reinforced with FRPs satisfy $\frac{M_c}{M_{cr}} > 1.5$ and all B-series beams in this study satisfied this requirement.

2.3.6.2 Ultimate Load and Mode of Failure

Figure 2.11 shows the failure of each specimen used in the study. Among Phase I beams, S41Y experienced a flexural tension mode of failure followed by flexural compression and S41N experienced a flexural tension mode of failure followed by shear failure when mid-span deflection was around 55 mm (2.2 in). However, all B-series beams
in Phase I experienced shear failure. Flexural tension modes of failure in S-series beams were evidenced by a plateau in the load-deformation plots, as well as yielding of the steel flexural reinforcement, when the strain become greater than 0.002. Flexural compression modes of failure were observed when the concrete in the compression zone crushed. In some cases, this happened more suddenly. Shear failure was always evidenced by the presence of a large diagonal crack in the shear span, followed by complete separation along the crack and a sudden large drop in load capacity. The shear capacity was calculated at the ultimate load for each specimen. In Phase I, specimen B41Y carried approximately 50% more load than its S-series counterpart, S41Y. However, specimen B41N carried 25% less load than S41N.

Among Phase II beams, S83Y experienced flexural tension failure followed by flexural compression. However, beam S83N exhibited flexural tension failure followed by shear failure at around 25 mm (1.0 in) deflection at the mid-span. Both B-series beams in Phase II experienced shear failure at a load 25% less than the ultimate capacity of S83Y and S83N.
Figure 2.11: Specimen failure modes

- **S41Y**: Slight compression failure
- **S83Y**: Compression failure
- **S41N**: Shear failure
- **S83N**: Shear failure
- **B41Y**: Shear failure
- **B83Y**: Shear failure
- **B41N**: Shear failure
- **B83N**: Shear failure
2.3.7 Ultimate Shear Capacity

The experimental and theoretical shear capacities of the specimens are presented in Table 2.5, as well as an analysis showing the contribution of the concrete and stirrups to the total shear resistance, \( V_r \), of each specimen. The theoretical ACI [11, 15] and CSA [12, 16] standard predictions of shear capacity, also divided into concrete contribution and stirrup contributions, are presented.

2.3.7.1 Effect of Steel vs BFRP on \( V_c \)

Specimens B41Y and B41N both experienced shear failure, and thus the effect of the BFRP stirrups can be analyzed. The experimental and theoretical shear resistance of the concrete section, \( V_c \), is presented in Table 2.5 for these specimens. For Phase I B-series beams, the ACI 440.1 [11] standard is unconservative in predicting \( V_c \), whereas the CSA S806 [12] standard is conservative. No comparisons can be drawn between the S- and B-series shear reinforced beams since both S41Y and S41N experienced flexural tension failure. However, analysis of the theoretical values of \( V_c \) indicates that the concrete contribution to shear for B-series specimens is approximately 40% less than their steel counterparts. This is due to lower stiffness of the BFRP rebar relative to the steel rebar.

A similar trend can be observed among Phase II B-series beams. The CSA S806 [12] standard is conservative in predicting \( V_c \). However, the ACI 440.1 [11] standard predicts a similar shear capacity to the experimental value. Since specimen S83N experienced shear failure, a direct comparison of \( V_c \) can be made between B- and S-series beams. Table 2.5 shows that the experimental value of \( V_c \) is approximately 30% less for B-series beams than for S-series beams. Both the ACI 318 [15] and CSA A23.3 [16] standards were very conservative in predicting \( V_c \) for the steel reinforced beams.
2.3.7.2 Effect of Stirrups on Shear Capacity

Table 2.5 presents the steel and FRP stirrup contributions ($V_s$ or $V_f$) to the shear resistance as well as comparisons to ACI 440.1 [11], ACI 318 [15], CSA S806 [12], and CSA A23.3 [16] standards. Beams B41Y carried approximately 50% more load than B41N, which can be attributed to the addition of BFRP stirrups. However, regardless of the addition of stirrups, beam B41Y still failed in shear. The fact that the stirrups did not have any hooks or bend suggests that the reason for the shear failure is insufficient development. As stated previously, BFRP rebar is currently made with a thermosetting epoxy resin, and thus cannot be bent without compromising the strength of the bar. Table 2.5 shows the experimental value of $V_s$ or $V_f$ alongside the theoretical predictions of ACI [11, 15] and CSA [12, 16] standards. Since the ACI [11] standard indicates that 160% of the theoretical contribution of $V_f$ was achieved, and the CSA [12] standard indicates that 80% of $V_f$ was achieved, it can be concluded that, for the Phase I B-series beams, the stirrups were effective in postponing the shear failure, despite the shear failure by debonding/slippage of the stirrups that occurred due to insufficient development.

The B83Y and B83N beams of Phase II both failed in shear, and at a similar load. The fact that they failed at similar loads highlights the insufficient development and lack of hooks or bends in the relatively short legs of the stirrups, becoming ineffective after a certain load. Analysis of stirrup strains at failure in Figures 2.10a and Figure 2.10b for both B41Y and B83Y confirms this. Stirrups in both beams experienced a strain of approximately 0.0012 at failure of the specimen. The ACI 440.1R-15 [11] standard implicitly limits the strain in the stirrups to 0.004. Table 2.5 also confirms that the effect of the stirrups in the Phase II B-series beams was minimal. The ACI 440.1 [11] and CSA S806 [12] standards show that 20% and 10% of the theoretical $V_f$ was achieved. Previous
tests by Tomlinson and Fam (2014) [9] and Bentz et al. (2010) [5] have shown that FRP stirrups tend to fail by rupture at the bend. Bentz et al. (2010) [5] also showed that, if multiple layers of flexural reinforcement are used, the FRP stirrups can be made to rupture away from the bend due to the lack of stress concentrations produced by the shear stress distribution in the cross section. Hence, it may be concluded that BFRP rebar as a stirrup material is not realistic yet until a thermoplastic resin is used in manufacturing the BFRP rebar or the stirrups are manufactured with thermosetting resin in the desired shapes.
Table 2.5: Ultimate capacity analysis

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Mode of Failure</th>
<th>$P_{ult}$ (kN)</th>
<th>$P_y$ (kN)</th>
<th>$M_r$ (kN-m)</th>
<th>$M_c$ (kN-m)</th>
<th>$V_r$ (kN)</th>
<th>$V_c$ (kN)</th>
<th>Test/predicted $V_r$ or $V_c$ (kN)</th>
<th>Test/predicted $V_r$ or $V_c$ (kN)</th>
<th>$V_r$ (kN)</th>
<th>Test/predicted $V_r$ or $V_c$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S10Y</td>
<td>Flexural tension</td>
<td>247.1</td>
<td>183.4</td>
<td>44</td>
<td>50</td>
<td>85</td>
<td>91.7</td>
<td>1.8</td>
<td>123.6</td>
<td>128.6</td>
<td>114.1</td>
</tr>
<tr>
<td>S10N</td>
<td>Flexural tension</td>
<td>230.8</td>
<td>187.2</td>
<td>44</td>
<td>50</td>
<td>85</td>
<td>93.6</td>
<td>1.9</td>
<td>115.4</td>
<td>128.6</td>
<td>107.1</td>
</tr>
<tr>
<td>B12Y</td>
<td>Shear</td>
<td>301.3</td>
<td>N/A</td>
<td>41</td>
<td>38</td>
<td>216</td>
<td>150.7</td>
<td>4.6</td>
<td>155.7</td>
<td>117.1</td>
<td>107.2</td>
</tr>
<tr>
<td>B12N</td>
<td>Shear</td>
<td>159.5</td>
<td>N/A</td>
<td>41</td>
<td>40</td>
<td>216</td>
<td>79.8</td>
<td>2.0</td>
<td>79.8</td>
<td>117.1</td>
<td>79.8</td>
</tr>
<tr>
<td>S15Y</td>
<td>Flexural tension</td>
<td>397.4</td>
<td>353.3</td>
<td>42</td>
<td>50</td>
<td>170</td>
<td>176.7</td>
<td>3.5</td>
<td>198.7</td>
<td>103.1</td>
<td>98.1</td>
</tr>
<tr>
<td>S15N</td>
<td>Flexural tension</td>
<td>398.2</td>
<td>371.2</td>
<td>46</td>
<td>60</td>
<td>170</td>
<td>185.6</td>
<td>3.1</td>
<td>199.1</td>
<td>103.1</td>
<td>199.1</td>
</tr>
<tr>
<td>B14Y</td>
<td>Shear</td>
<td>302.7</td>
<td>N/A</td>
<td>46</td>
<td>42</td>
<td>304</td>
<td>151.4</td>
<td>3.6</td>
<td>151.4</td>
<td>146.4</td>
<td>146.4</td>
</tr>
<tr>
<td>B14N</td>
<td>Shear</td>
<td>288.0</td>
<td>N/A</td>
<td>46</td>
<td>40</td>
<td>304</td>
<td>144.0</td>
<td>3.6</td>
<td>144.0</td>
<td>144.0</td>
<td>144.0</td>
</tr>
</tbody>
</table>

$T = $ Theoretical; $E = $ Experimental

*CSA S806-12 [12]
‡S-series: CSA A23.3-14 [16]; B-series: CSA S806-12 [12]
1 kN = 0.22 kip
1 kN-m = 0.74 kip-ft


2.4 CONCLUSIONS

Based on the results of the test program, the following conclusions are drawn. However, the conclusions are limited to the specific test specimens studied.

1. At the low reinforcement ratio, BFRP reinforced beams exhibited greater number of flexural cracking and shear cracking than their steel counterparts. At the higher reinforcement ratio, less shear cracking and slightly steeper shear crack angles are exhibited by BFRP reinforced beams.

2. Despite the low elastic modulus and low energy absorption of BFRP rebar, BFRP reinforced beams exhibited acceptable deformability according to CSA S6-14.

3. The cracking moments for S-series concrete beams are approximately 30 to 50% higher than those of B-series beams. Hence, this study suggests that the contribution of the rebar to the cracking moment should be in considered.

4. Although BFRP reinforced beams can be made to fail in flexural tension or flexural compression, shear failure can still govern the design of FRP reinforced concrete containing stirrups. The BFRP stirrups in this study were effective in delaying the shear failure for the low reinforcement ratio ($\rho/\rho_b < 1.0$). Stirrups in BFRP reinforced beams in Phase I was effective in increasing post-cracking stiffness.

5. $V_c$ is 30-40% less for BFRP reinforced beams. The CSA S806 standard is conservative in predicting $V_c$, whereas the ACI 440.1 standard is unconservative, but in some cases accurately predicts $V_c$.

6. $V_c$ is 30-40% less for BFRP reinforced beams. The ACI code is more conservative than the CSA code in predicting $V_c$. 
7. Stirrups without hooks have insufficient development. Stirrups with bends undergo brittle failure often initiated at the bend. The BFRP stirrups were effective in postponing the shear failure for the low reinforcement ratio ($\rho/\rho_b < 1.0$).

2.5 ACKNOWLEDGEMENTS

The authors wish to express their appreciation for MEDA Engineering & Technical Services for providing technical assistance necessary for this investigation. The financial assistance for this project was provided by NSERC.

2.6 REFERENCES


CHAPTER 3

FLEXURAL REHABILITATION AND STRENGTHENING
OF CONCRETE BEAMS WITH BFRP COMPOSITE

3.1 INTRODUCTION

Infrastructure around the world is subject to overuse and degradation. In North America, degradation of concrete structures is most often caused by the use of deicing salts, which accelerate the corrosion of reinforcing steel and causes spalling of concrete. Thus, there is a need to either replace or rehabilitate these concrete structures. The cost of rehabilitation may be orders of magnitude less than the cost of replacement, and thus, it may be a more economically feasible option to extend the service life of a structure, rather than replace it. The use of fibre reinforced polymers (FRPs) have been studied to address rehabilitation of concrete structures. Presently, glass and carbon fibre reinforced polymers (GFRP and CFRP) are the two primary types of FRP which have been commonly studied for strengthening and repair of reinforced concrete structures. New fibres are being introduced for the rehabilitation of reinforced concrete structures. In particular, basalt fibre products have been increasingly applied to civil engineering applications, especially in the form of rebar and chopped fibres. Sim et al. (2005) [1] studied the mechanical and durability properties of basalt fibres and found that they exhibit superior durability in accelerated weathering and high temperature testing compared to glass and carbon fibres. Further, basalt fibres are made of volcanic rock and hence, basalt fibre is a greener option. Basalt fibres are available in many different forms including fabric which can be used as
externally bonded composites for strengthening and rehabilitation of various structural components (Figure 3.1).

Strengthening beams in flexure involves applying externally bonded FRP composites to the bottom face with the fibres oriented in the longitudinal direction. Only two previous studies by Sim et al. (2005) [1] and Lihua et al. (2013) [2] were conducted on the use of BFRP externally bonded composite as a flexural strengthening material for reinforced concrete (RC) beams. In these studies, RC beams were strengthened with varying number of layers of unidirectional BFRP sheets and found that a higher number of layers of BFRP increases both the yield load and ultimate load capacities of the beams. However, these studies also found that if insufficient anchorage length is used, the failure can occur by interfacial debonding which is not desirable. Lihua et al. (2013) [2] also compared the performance of RC beams rehabilitated with BFRP composite to the performance of RC beams rehabilitated with GFRP and CFRP composites. The study found that the performance of beams rehabilitated with BFRP composite lies somewhere between the performance of RC beams rehabilitated with GFRP and CFRP composites. However, the study noted that on a cost-to-performance basis, BFRP is superior to both.

have shown that repairing concrete beams in flexure can both restore the capacity of the beam and even increase the ultimate load. One drawback of FRP rehabilitation, however, is that it can cause a reduction in ductility. The loss in ductility can be kept to a minimum if fibres with a sufficiently high elongation at rupture are used [4].

Al-Saidy and Al-Jabri (2011) [6] studied the effect of the use of CFRP fabrics on rehabilitation of corroded RC beams. This study determined and compared the effect of replacing the damaged concrete cover using a mortar patch versus using U-shaped CFRP sheets as cross straps to enhance the bond when the concrete cover is not replaced with a mortar patch. The study found that CFRP was effective in increasing the yield and ultimate load capacities of corroded concrete beams and that the U-shaped strips had a similar effect on the ultimate capacity and ductility as replacing the concrete cover because it was successful in preventing debonding failure. RC beams retrofitted with FRPs can fail in various modes; however, ideally, they should fail by yielding of steel reinforcement, followed by rupture of FRP, followed by compression failure of the concrete [7]. Other failure modes include compression failure, shear failure, debonding of FRP at concrete interface, debonding of concrete along the rebar, and peeling due to shear cracks. The research conducted to date has indicated that occurrence of any of these failure modes is possible.
Previous research has shown that flexural strengthening and rehabilitation of RC beams with CFRP and GFRP composites can improve the yield and ultimate load capacities of RC beams. Only two studies are available on the use of externally bonded basalt fibre reinforced polymers (BFRPs) for flexural strengthening or rehabilitation of corroded concrete beams. Thus, there is need to investigate the use of BFRP materials for flexural strengthening and rehabilitation of corroded concrete beams. Hence, this research was designed and executed to study the feasibility of using basalt fabrics as externally bonded composite for flexural strengthening and rehabilitating RC beams. The study was completed using full-scale tests.

3.2 EXPERIMENTAL PROGRAM

3.2.1 Test Specimens and Material Properties

Table 3.1 shows the test matrix. A total of seven beam specimens were prepared and tested in this study. The naming convention for the beams consists of a number indicating the corrosion percentage (0 or 20) followed by the longitudinal rebar size (10M or 15M) followed by the number of layers of BFRP composite (0L, 3L, or 8L). Hence, specimen 20-15M-0L is a beam specimen that had an area loss of 20% due to corrosion in flexural steel rebar. This beam specimen was made with 15M steel rebars. Each 10M rebar has cross-sectional area of 100 mm² and each 15M rebar has cross-sectional area of 200 mm². Since this was a control specimen it has no BFRP layers which is indicated by the 0L. It is worth indicating that in Table 3.1, specimen 0-15M-0L serves as a control specimen for both phases. The beams were made with concrete supplied by a local supplier. The target strength of the concrete was 35 MPa. The specimens are divided into two phases. The first phase had four specimens with no corrosion and these beam specimens were made
with two different reinforcement ratios (0.41% and 0.83%). Two of these specimens (0-10M-0L and 0-15M-0L) were control specimens and the other two (0-10M-3L and 0-15M-3L) were strengthened specimens. The latter two specimens were strengthened with 3 layers of BFRP composite externally bonded to the bottom face of the beams. In previous studies, a maximum of three layers of BFRP fabric was used for strengthening of RC beams [1]. The second phase of specimens consists of the 0-15M-0L specimen but with 20% corrosion (20-15M-0L) and rehabilitated with 8 layers of BFRP composite and with two different cross-strapping schemes: mid-span and bottom schemes (Table 3.1). The individual material properties of the concrete and steel are also shown in Table 3.1. In this table, $f'_c$ and $f_y$ are the specified compressive strength of concrete and yield strength of steel rebar, respectively.

Various details of the RC beam specimens used in this study are shown in Figure 3.2. The cross-section of the beams is shown in Figure 3.2d. The beams measure 500 mm deep, 275 mm wide, and 3200 mm long. Figures 3.2a and 3.2b show the elevation views of the specimens. All specimens contained stirrups spaced at 250 mm throughout the entire length of the specimen. The strengthened specimens (Phase 1) are shown in Figure 3.2a and rehabilitated specimens (Phase 2) are shown in Figure 3.2b. The Phase 2 specimens were deliberately cast with a missing concrete patch in the mid-span to simulate spalling concrete. The details of the patch are shown in Figures 3.2b and 3.2c.
Table 3.1: Test specimens

<table>
<thead>
<tr>
<th>Phase</th>
<th>Specimen ID</th>
<th>ρ</th>
<th>Corrosion (%)</th>
<th>Longitudinal Rebar</th>
<th>No. of BFRP Layers</th>
<th>Cross-strapping scheme</th>
<th>$f'_c$ (MPa)</th>
<th>$f'_y$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0-10M-0L</td>
<td>0.41%</td>
<td>0</td>
<td>5 x 10M</td>
<td>0</td>
<td>n/a</td>
<td>37</td>
<td>430</td>
</tr>
<tr>
<td></td>
<td>0-15M-0La</td>
<td>0.83%</td>
<td>0</td>
<td>5 x 15M</td>
<td>0</td>
<td>Mid-span</td>
<td>425</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0-10M-3L</td>
<td>0.41%</td>
<td>0</td>
<td>5 x 10M</td>
<td>3</td>
<td>Mid-span</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0-15M-3L</td>
<td>0.83%</td>
<td>0</td>
<td>5 x 15M</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>0-15M-0La</td>
<td>0.83%</td>
<td>0</td>
<td>5 x 15M</td>
<td>0</td>
<td>n/a</td>
<td>37</td>
<td>425</td>
</tr>
<tr>
<td></td>
<td>20-15M-0L</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>20-15M-8L</td>
<td>0.83%</td>
<td>20</td>
<td>5 x 15M</td>
<td>8</td>
<td>Mid-span</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>20-15M-8LXS</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Bottom</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*aSame specimen
*bRepeat specimen to fix debonding using bottom span scheme

Simulation of spalling was implemented by means of a missing patch of concrete with an irregular surface. High density rigid insulation foam board was used to form the patch during casting (Figure 3.3a). Corrosion in the steel rebar was introduced by machining. Twenty percent of the total area of the tension steel was removed in the lower three bars. Patching of the missing concrete was done using repair mortar (Figure 3.4). The
manufacturer’s instructions were followed for mixing and application of the mortar. Forms were placed on both sides of the patch in order to make it flush with the existing outside face of the beam.

![Figure 3.3: Spalling and corrosion simulation](image1.png)

(a) Foam insert in formwork  (b) Machined rebars

![Figure 3.4: Patchwork](image2.png)

(a) Patching  (b) Final patch

Prior to application of epoxy, the surface was prepared by first cleaning it with compressed air and then priming it. The primer was allowed to set up for 24 hours until it was tack-free before applying epoxy. Two-part epoxy was used to apply the basalt unidirectional fabric. A “dry lay-up” method of applying the fabric was used. First, a thick layer of epoxy was applied to the primer coat of the beam and rolled on using a paint roller (Figure 3.5). Then dry layers of fabric were applied over the epoxy that was rolled onto the
beam. More epoxy was subsequently applied over the previously applied layer and rolled into the fibres applying some pressure. This process was completed for each layer applied. Cross strapping was applied in the direction perpendicular to the longitudinal fabric after all longitudinal layers were applied for the mid-span scheme (Figure 3.6a). For the bottom scheme (Figure 3.6b), the cross strapping was applied after the $4^{th}$, $6^{th}$, and $8^{th}$ layers of the longitudinal fabric. The epoxy was allowed to cure for a minimum of seven days before testing. Once the specimens for Phase 2 were cured, one face of each specimen was painted for the application of the digital image correlation (DIC) strain measurement technique.

![Application of epoxy](image1.png)

(a) Epoxying longitudinal fabric  
(b) Cross-strapping: bottom scheme

**Figure 3.5**: Application of epoxy
ASTM D3039-14 [8] was followed to determine the mechanical properties of the BFRP composite used in this study and the test setup is shown in Figure 3.7. The BFRP fabric was cut to length and the fibre was immersed in the epoxy. Tabs were epoxied at the end to avoid stress concentrations. The specimens were allowed to cure for seven days and then cut into 15 mm wide strips. The average thickness of the strips is 0.33 mm. The specimens were tested in a 50 kN capacity universal testing machine. Specimens were loaded at a rate of 1 mm/min until rupture. The load data (stress values) were obtained through the loadcell attached to the universal testing machine, whereas the displacement data (strain values) were obtained using digital image correlation (DIC) technique. The mechanical properties of the composite are reported in Table 3.2. The mechanical properties of the composite are reported in Table 3.2.
properties of the rebar were determined according to ASTM A370-14 [9]. The yield
strengths are reported in Table 3.1. ASTM C39-15 [10] was followed to determine the
compressive strength of the concrete (Table 3.1).

![BFRP composite test setup](image1)

![BFRP composite DIC](image2)

**Figure 3.7: BFRP composite tension test setup**

<table>
<thead>
<tr>
<th>Fabric Weight (g/m²)</th>
<th>Ultimate Load (kN/mm/layer)</th>
<th>Ultimate Stress, $f_u$ (MPa)</th>
<th>Ultimate Strain, $\varepsilon_{fu}$ (%)</th>
<th>Modulus of Elasticity, $E_f$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>0.17</td>
<td>493</td>
<td>2.52</td>
<td>20.4</td>
</tr>
</tbody>
</table>

**Table 3.2: BFRP composite tensile properties**

3.2.2 Test Procedure and Instrumentation

The test set-up is shown in Figure 3.8. The beams spanned 3000 mm in a simply
supported boundary condition with a roller between two plates at one end and a knife edge
between two plates at the other. The beam was loaded through two point loads at the top
spaced 1000 mm apart. The load was applied from a universal loading actuator through a
steel spreader beam. Mid-span displacement was measured using a linear variable
differential transformer (LVDT) which was attached to the loading actuator (Figure 3.8).
Strain gauges were placed on the tension reinforcement for each specimen. Figures 3.2a
and 3.2b show the location of the tension gauges ($\varepsilon_t$). Strain gauges were also externally
applied to the BFRP at the mid-span (not shown in Figures 3.2a and 3.2b). For Phase 2 specimens, the DIC technique was used, and thus, two cameras were placed to be able cover the entire span of the beam specimen (Figure 3.8b). Ten photo frames per minute were collected during the test and these photos were saved on a computer. Displacement control was used for the application of the load. Loading was continued until a clear BFRP debonding or a rupture, or an obvious flexural compression failure was observed.
Figure 3.8: Test setup

(a) Schematic view

(b) Photo
3.3 EXPERIMENTAL RESULTS AND DISCUSSION

3.3.1 Crack Pattern and Crack Width

The crack patterns for Phase 1 specimens are shown in Figure 3.9. There are no noticeable differences in the patterns between the unstrengthened and their respective strengthened specimens. The crack spacing and number of cracks appears to be similar among strengthened and unstrengthened specimens. However, as expected, the specimens with a higher reinforcement ratio (0-15M-0L and 0-15M-3L) developed more shear cracking than those with a lower reinforcement ratio (0-10M-0L and 0-10M-3L).

![Figure 3.9: Phase 1 crack patterns](image)

Figure 3.9: Phase 1 crack patterns

Figure 3.10 shows analysed DIC data obtained from Phase 2 beam specimens at the yield load. The colours in the photos are representative of the magnitude of the longitudinal strain. No significant difference in crack spacing and crack pattern can be observed between the control specimens (Figures 3.10a and 3.10b) and rehabilitated specimens (Figures 3.10c and 3.10d). Figure 3.10 also shows the crack width and spacing of each of
the Phase 2 specimens. The vertical axis represents the infinitesimal change in length of
the extreme tension fibre of the beam and the horizontal axis represents the position along
the beam from the bottom-left corner of the beam. Each step represents a crack at that
location and the vertical magnitude of the change represents the crack width. In the
uncorroded control specimen, the largest crack width at the yield load was approximately
0.5 mm wide. The cracks in the corroded control specimen were also similar in width as
those in the uncorroded control specimen. Cracks in both of the rehabilitation specimens,
20-15M-8L and 20-15M-8LXS, are significantly reduced in width, indicating that the
longitudinal BFRP composite reduced the crack width at the yield load. This trend was also
observed at the service load.
3.3.2 Load-Deflection Response

Figures 3.11a and 3.11b show the load-deflection responses of the Phase 1 and 2 specimens, respectively. The deflection presented in this figure was obtained from the LVDT attached to the loading actuator and hence, it is the mid-span deflection. The pre-cracking load-deflection behaviour of all Phase 1 specimens was similar. Between post-cracking and yielding of the tension reinforcement, the slope of the load-deflection curve (called “stiffness” in this chapter for the sake of the discussion) of the specimens were also similar. As expected, the Phase 1 specimens with a lower steel reinforcement ratio sustained less load at yield than the higher reinforcement ratio specimens. After yielding
of the longitudinal reinforcement, the stiffness of the specimen reduced considerably. The post-yielding stiffness of the strengthened specimens was higher than that of the unstrengthened specimens. On average, the post-yielding stiffness of the strengthened specimens (Phase 1) increased by a factor of 3.8. For the Phase 1 specimens, the test was stopped immediately after rupture or debonding of the BFRP composite occurred and the specimen was unloaded. When failure occurred, the load dropped to approximately that of the unstrengthened specimen.

The pre-cracking behaviour of all Phase 2 specimens was also similar. Between cracking of the section and yielding of the steel, the stiffness was also similar. As expected, the specimen with 20% corrosion carried less load than the control specimen at yield. After yielding, both control specimens (0-15M-0L and 20-15M-0L) experienced a significant reduction in stiffness, whereas this reduction was less significant in the two rehabilitated specimens (20-15M-8L and 20-15M-8LXS). The post-yielding stiffness of the two rehabilitated specimens in Phase 2 increased by a factor of 5.6 relative to the control specimen. After yielding of the tension steel, the specimens continued to carry increasing load until either rupture or debonding of the BFRP occurred. After rupture or debonding occurred, the load dropped to approximately that of the corroded control specimen, 20-15M-0L. For Phase 2 specimens, the tests were continued until a flexural-compression failure was observed.
Table 3.3 presents the service loads for each specimen. The service load in this study is defined as the most conservative of the load calculated using four different criteria. The first two criteria are the load at a mid-span deflection of $L/360$ and $L/180$ [11, 12]. The next criterion is the load at the service strain in steel of 0.0012 as recommended by Newhook et al. (2002) [13]. The last condition is the maximum sustained load divided by
the load factor 1.5 [14]. In every case, the service load was governed by the service strain criterion recommended by Newhook et al. (2002) [13]. The service load did not change significantly for strengthened specimens (0-10M-3L and 0-15M-3L). However, in each of the rehabilitated specimens of Phase 2, 20-15M-8L and 20-15M-8LXS, the service load was successfully restored to the level of the uncorroded control specimen, 0-15M-3L. The service load was increased from approximately 165 kN for 20-15M-0L to approximately 190 kN for the two rehabilitated specimens, representing an increase of about 15%.

**Table 3.3: Service loads**

<table>
<thead>
<tr>
<th>Phase</th>
<th>Specimen ID</th>
<th>Load at different criteria (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>at L/360</td>
</tr>
<tr>
<td>1</td>
<td>0-10M-0L</td>
<td>184.7</td>
</tr>
<tr>
<td></td>
<td>0-15M-0L</td>
<td>295.0</td>
</tr>
<tr>
<td></td>
<td>0-10M-3L</td>
<td>201.7</td>
</tr>
<tr>
<td></td>
<td>0-15M-3L</td>
<td>273.5</td>
</tr>
<tr>
<td>2</td>
<td>0-15M-0L</td>
<td>295.0</td>
</tr>
<tr>
<td></td>
<td>20-15M-0L</td>
<td>276.4</td>
</tr>
<tr>
<td></td>
<td>20-15M-8L</td>
<td>296.8</td>
</tr>
<tr>
<td></td>
<td>20-15M-8LXS</td>
<td>308.2</td>
</tr>
</tbody>
</table>

$L/360 = 8.3 \text{ mm}, L/180 = 16.7 \text{ mm}$

$\varepsilon_{\text{service}} = 0.0012$

**3.3.3 Ductility**

The ductility of each specimen was calculated two ways and is presented in Table 3.4. The first method used is the deflection ductility index ($\mu_\Delta$) as defined by Equation 3.1.

$$\mu_\Delta = \frac{\Delta_u}{\Delta_y} \quad (3.1)$$

The term $\Delta_u$ is the mid-span displacement at ultimate load. The term $\Delta_y$ is the mid-span displacement at the yield load. As can be seen in Table 3.4, based on the deflection ductility index, ductility is reduced by about 30% in the Phase 1 strengthened specimens.
Among the Phase 2 specimens, it is clear that the corroded control specimen (20-15M-0L) exhibited the highest ductility, followed by the uncorroded control specimen (0-15M-0L). The two rehabilitated specimens exhibited less ductility than the uncorroded control specimen. However, the reduction in ductility remained less than 30%. A similar trend was also observed by Attari et al. (2012) [4] who used GFRP, CFRP, and GFRP-CFRP hybrid fabrics for rehabilitation. Attari et al. (2012) [4] also observed that all strengthened specimens exhibited less ductility than the control specimens. However, their study found that the reduction in ductility was less than 20%.

The second method for calculating ductility is the energy ductility index ($\mu_E$) as defined by Equation 3.2.

$$\mu_E = \frac{E_u}{E_y}$$  \hspace{1cm} (3.2)

The term $E_u$ is the energy absorption at the ultimate load (integration of load-displacement curve up to the ultimate load). The term $E_y$ is the energy absorption at yield (integration of load-displacement curve up to the yield load). Similar trends can be observed to those of the deflection ductility index. The strengthened specimens (Phase 1) exhibited less ductility than that of their respective unstrengthened specimens. Among Phase 2 specimens, 20-15M-0L has the highest ductility, followed by 0-15M-0L. The two rehabilitated specimens in Phase 2 also showed less ductility than the uncorroded control specimen if the deflection ductility index is used.
Table 3.4: Ductility

<table>
<thead>
<tr>
<th>Phase</th>
<th>Specimen ID</th>
<th>Deflection at Yield (mm)</th>
<th>Deflection at Ultimate (mm)</th>
<th>Ductility Index</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Deflection</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(μ∆)</td>
</tr>
<tr>
<td>1</td>
<td>0-10M-0L</td>
<td>6.8</td>
<td>61.8</td>
<td>9.1</td>
</tr>
<tr>
<td></td>
<td>0-15M-0L</td>
<td>10.7</td>
<td>40.2</td>
<td>3.8</td>
</tr>
<tr>
<td></td>
<td>0-10M-3L</td>
<td>7.5</td>
<td>40.1</td>
<td>5.3</td>
</tr>
<tr>
<td></td>
<td>0-15M-3L</td>
<td>13.1</td>
<td>39.4</td>
<td>3.0</td>
</tr>
<tr>
<td>2</td>
<td>0-15M-0L</td>
<td>10.7</td>
<td>40.2</td>
<td>3.8</td>
</tr>
<tr>
<td></td>
<td>20-15M-0L</td>
<td>10.5</td>
<td>50.0</td>
<td>4.8</td>
</tr>
<tr>
<td></td>
<td>20-15M-8L</td>
<td>10.3</td>
<td>22.9</td>
<td>2.2</td>
</tr>
<tr>
<td></td>
<td>20-15M-8LXS</td>
<td>9.7</td>
<td>26.9</td>
<td>2.8</td>
</tr>
</tbody>
</table>

3.3.4 Strain Response

The load-strain data obtained from the strain gauges placed on the tension steel rebar (εT in Figures 3.2a and 3.2b) and BFRP composite is shown in Figure 3.12. In the Phase 1 beams, the strain gages mounted on tension steel rebar and BFRP showed a very similar behaviour indicating good composite action between the RC beam and BFRP composite (Figure 3.12a). Similar behaviour was also observed in all Phase 2 beams. However, specimen 20-15M-8L of Phase 2 experienced a temporary cessation in strain increase after yielding (between points C and D1 in Figure 3.12b). Visual inspection rules out any global debonding between the BFRP composite and the RC beam at this stage. The strain gauge’s strain data represents a local strain and hence, the strain gauge data may be affected by the presence of a small crack or other localised defect, if present at that location. At point D1 for beam specimen 20-15M-8L, crack formation possibly caused the local strain to increase to point D2 and then relax to point E. At point E, the BFRP composite debonded causing sudden drop in the load to point F while the strain did not change. The strain path between F to G became similar to that of the uncorroded control specimen (as seen in specimen 20-15M-0L in Figure 3.12b) since there was no contribution of BFRP in
this range. Nonetheless, all specimens from both phases including specimen 20-15M-8L indicated that there was good composite action between the BFRP composite and RC beam within the elastic range.

3.3.5 Moment Capacity and Mode of Failure

The theoretical and experimental cracking moments and resisting moments are presented in Table 3.5. Theoretical cracking moments (indicated by T in Table 3.5) were calculated in accordance with the guidelines of CSA S806-12 [11]. In most cases, the theoretical cracking moments are less than the experimental ones. This is due to the additional moment of inertia contributed by the steel and FRP composite, which is ignored in the theoretical calculation.

Table 3.5 also presents the theoretical flexural resisting moments (indicated by T in Table 3.5) and experimental moment resistance (indicated by E in Table 3.5). Theoretical resisting moments were calculated using CSA A23.3-14 [12] for beams without any BFRP composite and CSA S6-14 [15] for beams with BFRP composite. Table 3.5 shows that theoretical and experimental values are in good agreement and the difference was no more than 5%. It is important to note that the resisting moments of BFRP strengthened or

![Figure 3.12: Load-strain plots for tension steel and FRP](image)

(a) Phase 1 beams  
(b) 20-15M-8L specimen

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rehabilitated beams are calculated when FRP strain reaches 0.006 as specified by CSA S806-12 [11].

As can be observed in Table 3.5, the ultimate sustained moments for strengthened and rehabilitated beam specimens are significantly higher in each case than the resisting moments. This is due to the increase in the flexural capacity provided by the BFRP composite. Further, the CSA S6-14 [15] standard limits the strain in the BFRP composite to 0.6% for the calculation of the resisting moment. However, the rupture strain of BFRP composite is about 2.5% (Table 3.2). Hence, the Canadian standard CSA S6-14 [15], utilizes only about 25% of the tensile capacity of the BFRP composite while calculating the resisting moment.

Among Phase 1 specimens, the ultimate sustained load for 0-10M-3L is about 25% higher than 0-10M-0L. Likewise, the ultimate sustained load for 0-15M-3L is about 25% higher than that of 0-15M-0L. Among Phase 2 specimens, the ultimate sustained load of 20-15M-8L is about 15% higher than the uncorroded control specimen 0-15M-0L. It should be noted that specimen 20-15M-8L failed due to interfacial debonding. However, the ultimate sustained load of 20-15M-8LXS is about 30% higher than that of the uncorroded control specimen (0-15M-0L) since 20-15M-8LXS failed due to rupture of BFRP composite. Thus, the “bottom scheme” cross-strapping was effective in eliminating the debonding failure and significantly increasing the ultimate sustained load.

Yield loads for strengthened specimens in Phase 1 also increased. The yield load for 0-10M-3L increased by about 10% compared to the control specimen 0-10M-0L. The yield load for 0-15M-3L was also increased by about 10% compared to 0-15M-0L. Among Phase 2 specimens, the yield load for the two rehabilitated specimens was successfully
restored to approximately that of the uncorroded control specimen 0-15M-0L. Hence, the yield load capacity can be restored or even increased with the application of BFRP composite.

Figure 3.13 shows the various failure modes for specimens of both phases. The various modes of failure are also summarized in Table 3.5. Where there was more than one mode of failure, the modes are shown in the order that they occurred. As can be found in this table, all beams first experienced a flexural tension (FT) mode of failure resulting from yielding of steel rebar. For all control specimens and Phase 2 specimens, the final failure mode was always flexural compression (FC). The second mode of failure for all strengthened and rehabilitated specimens was rupture (R) or debonding (D) of the BFRP composite. Rupture was a much more progressive failure, where sections of the BFRP composite would rupture while others remained intact. As the specimen was further loaded, more sections of the composite ruptured gradually. This is particularly true for the mid-span scheme of cross-strapping. The bottom scheme of cross-strapping controlled the location of the rupture to a single location, and hence the rupture occurred for all fibres almost simultaneously (Figure 3.13b). Debonding failure was always sudden, energetic, and loud. For Phase 2 rehabilitated specimens, the test was continued until flexural compression failure was observed. Thus, the Phase 2 rehabilitated specimens have three modes of failure.
Table 3.5: Cracking, resisting, and ultimate moments

<table>
<thead>
<tr>
<th>Phase</th>
<th>Specimen ID</th>
<th>Mode of Failure</th>
<th>( P_{ult} ) (kN)</th>
<th>( P_y ) (kN)</th>
<th>( *P_{eff} ) (kN)</th>
<th>( M_{cr} ) (kN-m)</th>
<th>( M_r ) (kN-m)</th>
<th>( M_{ult} ) (kN-m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0-10M-0L</td>
<td>FT, FC</td>
<td>247.1</td>
<td>183.4</td>
<td>-</td>
<td>44</td>
<td>50</td>
<td>92.6</td>
</tr>
<tr>
<td></td>
<td>0-15M-0L</td>
<td>FT, FC</td>
<td>397.4</td>
<td>353.3</td>
<td>-</td>
<td>42</td>
<td>50</td>
<td>172.2</td>
</tr>
<tr>
<td></td>
<td>0-10M-3L</td>
<td>FT, R</td>
<td>313.2</td>
<td>201.4</td>
<td>219.2</td>
<td>41</td>
<td>32</td>
<td>103.3</td>
</tr>
<tr>
<td></td>
<td>0-15M-3L</td>
<td>FT, D</td>
<td>486.5</td>
<td>391.3</td>
<td>407.8</td>
<td>41</td>
<td>46</td>
<td>199.1</td>
</tr>
<tr>
<td>2</td>
<td>0-15M-0L</td>
<td>FT, FC</td>
<td>397.4</td>
<td>353.3</td>
<td>-</td>
<td>42</td>
<td>50</td>
<td>172.2</td>
</tr>
<tr>
<td></td>
<td>20-15M-0L</td>
<td>FT, FC</td>
<td>361.7</td>
<td>305.4</td>
<td>-</td>
<td>43</td>
<td>49</td>
<td>145.3</td>
</tr>
<tr>
<td></td>
<td>20-15M-8L</td>
<td>FT, D, FC</td>
<td>451.9</td>
<td>330.1</td>
<td>392.4</td>
<td>43</td>
<td>64</td>
<td>185.7</td>
</tr>
<tr>
<td></td>
<td>20-15M-8LXS</td>
<td>FT, R, FC</td>
<td>507.5</td>
<td>346.1</td>
<td>386.2</td>
<td>42</td>
<td>65</td>
<td>179.8</td>
</tr>
</tbody>
</table>

FT = Flexural Tension  
FC = Flexural Compression  
D = Debonding  
R = Rupture  
\(*_{eFRP} = 0.006\)
Figure 3.13: Failure modes

(a) Phase 1 specimens

(b) Phase 2 specimens
3.4 CONCLUSIONS

Based on the results of this study, the following conclusions can be drawn. However, the conclusions may be limited to the specific test specimens studied.

1. BFRP composite is a green and suitable material for the use in rehabilitation of damaged concrete beams and strengthening of strength-deficient RC beams. Application of BFRP composite was found to be effective in restoring the service, yield, and ultimate load capacities to the level of the uncorroded RC beam. Increase in load carrying capacities in strengthened beams was about 25% with only three layers of BFRP composite.

2. Application of BFRP composite in flexure resulted in significantly reduced crack widths.

3. Failure of a rehabilitated or strengthened RC beam due to debonding must be avoided since debonding is a sudden failure. Further, premature debonding failure can result in a large reduction in load carrying capacity. Hence, the use of appropriate cross-strapping is important in avoiding debonding failure. The “bottom scheme” method of cross-strapping used in this study was effective in avoiding failure by debonding.

4. Strengthening and rehabilitation of RC beams using BFRP composites resulted in a reduction in the ductility. However, the reduction can be limited to 30% if BFRP composite is used.

3.5 ACKNOWLEDGEMENTS

The authors wish to express their sincere appreciation for MEDA Engineering & Technical Services for providing technical assistance necessary for this investigation as
well as Sahan Jayasuriya and Amirreza Bastani for conducting the tension tests on the BFRP composite and Eric Hughes for helping with specimen preparation. Financial assistance in the form of graduate scholarship for the project was provided by NSERC to Mr. Duic.

3.6 REFERENCES


CHAPTER 4

REHABILITATION OF SHEAR DEFICIENT RC BEAMS WITH BASALT FIBRE REINFORCED POLYMER COMPOSITE

4.1 INTRODUCTION

The durability of reinforced concrete (RC) structures has remained a significant problem that has plagued its use, especially in cold climates where deicing salts are used extensively. Deficiencies arising from the corrosion of the reinforcing steel and spalling of concrete can affect the shear capacity of RC elements. In the United States alone, approximately 1 in 9 bridges is considered structurally deficient, representing approximately $120 billion of necessary infrastructures spending [1]. Thus, there is a need to repair these deficiencies, especially for RC structures. The use of fibre reinforced polymers (FRPs) has been proposed to solve this problem since early 1970s and has been researched ever since. The use of carbon fibre reinforced polymers (CFRPs) and glass fibre reinforced polymers (GFRPs) have been well researched for application as shear rehabilitation materials for RC structures. However, a new eco-friendly material, namely basalt fibre reinforced polymer (BFRP) has been introduced for applications in structural engineering. Figure 4.1 shows basalt unidirectional fabric. Sim et al. (2003) [2] studied the mechanical and durability of basalt fibres and found that this fibre performed better in accelerated weathering conditions compared to glass and carbon fibres. Only two studies are reported in the literature where BFRP composites were used for strengthening of RC
beams [2-3]. However, both studies have investigated the use of BFRP composites for flexural strengthening of RC beams.

Chaallal et al. (1998) [4] studied the effect of CFRP strips in shear strengthening RC beams and found that the strips were effective in increasing the shear strength, reducing shear cracking, and increasing ductility. The use of strips applied either perpendicular or diagonal to the longitudinal axis of the beams was studied and it was found that diagonal strips slightly outperformed perpendicular strips. Baggio et al. (2014) [5] also studied the use of both CFRP and GFRP for shear strengthening of RC beams. Additionally, this research studied the use of FRP anchors as well as full and partial depth wrapping of RC beams. This study found that the use of CFRP and GFRP both increased the capacity of shear critical beams. The beams strengthened with GFRP failed by debonding, while beams strengthened with CFRP did not. However, beams strengthened with GFRP that had anchorage avoided failure by debonding. This study also found that the code equations provided by CSA A23.3-14 [6] and CSA S806-12 [7] were accurate in predicting shear capacities of strengthened and unstrengthened beams. Taljsten and Elfgren (1999) [8] also studied the use of CFRP as a shear strengthening material for RC beams and found that it was effective in increasing the shear capacity. This study also found that strengthened specimens still experienced a brittle shear failure. Pellegrino and Modena (2006) [9]
studied the interaction of externally bonded FRP and internal steel shear reinforcement and found that there is an interaction and that some design standards overestimate the FRP contribution to shear because of this interaction. Teng et al. (2002) [10] also studied this interaction and found that some codes were non-conservative in predicting the contribution of FRP to the shear resistance of the beam specimens.

To the best of the authors’ knowledge, no research has been conducted on the feasibility of using BFRP composites for shear rehabilitation or shear strengthening of RC beams. However, several studies in the past have been conducted on the use of CFRP and GFRP composites for shear strengthening of RC beams. Given the lack of research on the use of BFRP as a shear strengthening material for reinforced concrete beams, there is a need to investigate its effectiveness. Thus, the following experimental program was designed to study the effectiveness of BFRP composite as a material for shear rehabilitation of RC beams with varying levels of corrosion damage.

4.2 EXPERIMENTAL PROCEDURE

4.2.1 Test Specimens and Material Properties

This investigation consists of seven large reinforced concrete (RC) beam specimens and they are shown in Table 4.1. The beams are named according to the corrosion percentage followed by a hyphen followed by the number of BFRP composite layers used. Thus, specimen 50-3L is a beam specimen with 50% loss in the area of shear reinforcement due to corrosion and this specimen was repaired with three layers of BFRP composite. The first beam specimen (0-0L) in Table 4.1 is an uncorroded control (virgin) specimen. The following six specimens are grouped into sets of two. The first specimen in each group is a corroded control specimen (20-0L, 50-0L, and 100-0L). These specimens had a shear
deficiency of 20%, 50%, and 100%, respectively. All specimens contained the same amount of flexural reinforcement. Each of these specimens had a companion specimen which was rehabilitated using three layers of BFRP composites (20-3L, 50-3L, and 100-3L). For example, specimens 50-0L and 50-3L are companion specimens. The specimen 50-0L is the corroded control specimen whereas, specimen 50-3L is an identical corroded specimen which was rehabilitated with three layers of BFRP composites. The specified compressive strength of concrete, \( f'_{c} \), and the actual yield strength of steel, \( f_{y} \), are also shown in Table 4.1.

The beam cross sections and elevation views are shown in Figure 4.2. The cross section of each beam was 500 mm deep by 275 mm wide and each beam measured 3200 mm in length with a span of 3000 mm. Each beam had five 15M steel rebars, producing a section with a reinforcement ratio of \( \rho = 0.83\% \). Cross-sectional area of each 15M rebar is 200 mm². As can be seen in the figure, the beams were cast with a patch of concrete missing in the shear span on both sides of the beam. This was done to simulate the spalling of concrete and the associated section loss. The damaged specimens were then patched with mortar and subsequently rehabilitated with BFRP composite as shown in Figure 4.3

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Corrosion (%)</th>
<th>No. of BFRP Layers</th>
<th>( f'_{c} ) (MPa)</th>
<th>( f_{y} ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-0L</td>
<td>0</td>
<td>0</td>
<td>38</td>
<td>425</td>
</tr>
<tr>
<td>20-0L</td>
<td>20</td>
<td>0</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>20-3L</td>
<td>20</td>
<td>3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>50-0L</td>
<td>50</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
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<td>50</td>
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<td></td>
</tr>
<tr>
<td>100-0L</td>
<td>100</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>100-3L</td>
<td>100</td>
<td>3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
All the beam specimens for this study were cast using concrete supplied by a local ready-mix plant. For the specimens with 20% corrosion, the loss of cross-sectional area in the stirrups due to corrosion was simulated by means of machining (Figure 4.4a). The stirrups were machined on both legs to remove 20% of the total cross-sectional area. For the specimens with 50% area loss due to corrosion, every other stirrup was removed starting with the second one. For the 100% corrosion specimen, all stirrups were removed. Spalling of concrete was simulated by means of a patch made of rigid insulation foam placed in the formwork (Figure 4.4b). Patching of the missing concrete was done with a
concrete repair mortar and allowed to cure fully prior to application of the BFRP composite for rehabilitation (Figure 4.5). The section of the beam to which BFRP composite was to be applied was first primed and allowed to cure for 24 hours (Figure 4.6a). The BFRP composite was then applied to the surface of the concrete. The fabric used in this study was uni-directional (Figure 4.1) and hence, the fabric was applied with the fibres oriented perpendicular to the longitudinal axis of each beam specimen using a dry lay-up method (Figure 4.6b). Figure 4.3 shows the location of the BFRP composite in elevation and section view. After repairing each specimen, it was allowed to cure for seven days. The specimens were then painted for implementation of the digital image correlation (DIC) technique.

Figure 4.4: Corrosion and spalling simulation

(a) 20% corrosion stirrups  (b) Foam insert
The mechanical properties of the steel rebar were found according to ASTM A370-14 [11] and reported in Table 4.1. ASTM C39-15 [12] was followed to determine the compressive strength of the concrete (Table 4.1). ASTM D3039-14 [13] was followed to determine the tensile properties of the BFRP composite used in this investigation. Figure 4.7 shows the test setup for determining the tensile properties of the composite. The basalt fabric was cut to length and the fibre was immersed in the epoxy. Tabs were epoxied at the ends to avoid stress concentrations. The specimens were allowed to cure for seven days and then cut into 15 mm strips. The average thickness of the strips is 0.33 mm. The specimens were tested in a 50 kN universal testing machine. Specimens were loaded at a
rate of 1 mm/min until rupture occurred. The load data was collected through a loadcell attached to the universal testing machine and the strain data was acquired using the DIC technique. The mechanical properties of the BFRP composite are reported in Table 4.2.

<table>
<thead>
<tr>
<th>Fabric Weight (g/m²)</th>
<th>Ultimate Load (kN/mm)</th>
<th>Ultimate Stress, $f_u$ (MPa)</th>
<th>Ultimate Strain, $\varepsilon_{fu}$ (%)</th>
<th>Modulus of Elasticity, $E_f$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>0.17</td>
<td>493</td>
<td>2.52</td>
<td>20.4</td>
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</tbody>
</table>

**Table 4.2: BFRP composite tensile properties**

**4.2.2 Test Procedure and Instrumentation**

The beam specimens were tested in four point bending as shown in Figure 4.8. The beams had a clear span of 3.0 m and were supported by a pin and a roller at the ends. Load was applied from a loading actuator onto a steel loading beam which spread the load to two points 1.0 m apart. Load was applied using displacement control method until either a shear failure or a flexural compression failure was observed. Displacement and load data were acquired through the linear variable differential transformer (LVDT) and loadcell attached to the loading actuator. Additional loadcells were also placed at each end of the beam to
verify that the load was spread evenly. Pictures for DIC were collected through two cameras so that they covered the entire span. The photos were taken periodically and saved to the hard drive on the data acquisition system.

Figure 4.8: Test setup
4.3 TEST RESULTS AND DISCUSSION

4.3.1 Crack Width and Distribution

Figure 4.9 shows the crack patterns and crack widths at the service and yield loads for the uncorroded control (virgin) specimen, 0-0L, obtained from the DIC data. The horizontal axis on the plot represents the position on the specimen of the extreme tension fibre measured from the left corner. The vertical axis represents the infinitesimal change in length of the extreme tension fibres of the beam at that location. Thus, the vertical steps in the plot represent cracks at that location and the magnitude of the step indicates the crack width measured in the horizontal direction (x-direction). However, the shading in the photo are representative of the diagonal strain, $\varepsilon_{xy}$. This was done to highlight the shear cracks. As can be seen in the figure, the maximum shear crack width (measured in the x-direction) at the service and yield loads are approximately 0.2 mm and 0.5 mm, respectively. Shear and flexural crack spacing appears to be uniform, with about three distinct shear cracks visible in each shear span.

![Figure 4.9: Crack widths of 0-0L](Image)

Figure 4.10 shows the width and distribution of shear cracks on the damaged specimens (corroded control specimens) and companion rehabilitated specimens obtained
from the DIC data. Again, the photos in the figure show the diagonal strain, $\varepsilon_{xy}$, and thus, the diagonal tension cracks are shown in these figures. In these photos, the flexural cracks are less visible since the strains are shown in the diagonal (x-y) direction. However, the widths of flexural cracks are shown in the corresponding plots. Analysis of the data indicates that for each of the rehabilitated specimens, the widths of the flexural cracks at the mid-span are larger than those of the corresponding damaged specimen (corroded control specimen). This is probably due to the increased stiffness in the shear span causing more deformation to occur in the mid-span and less in the shear span relative to the unrehabilitated counterpart specimens (corroded control specimen). Another research project that is being undertaken at the University of Windsor by the authors indicates that the flexural crack widths are significantly reduced when flexural strengthening or rehabilitation is performed on beams with flexural deficiencies. Thus, it is recommended that shear rehabilitation should accompany the application of flexural rehabilitation to limit the widths of the flexural cracks.

Between specimens 20-0L and 20-3L, the pattern of shear cracking in the damaged specimen (corroded control specimen) and localised areas of high strain in the rehabilitated specimen appear to be similar (Figure 4.10a and 4.10b). However, the values of shear crack widths in the rehabilitated beam specimen may not be accurately represented in the DIC since the DIC data were acquired on the outside surface of the BFRP composite. Nonetheless, this study shows that DIC can be used as a reliable and easy-to-use technology that allows capturing of the strain contour for the entire specimen. This study also shows that the DIC is able to show the areas of localized high tensile strain in the diagonal direction even if the surface of concrete is covered by BFRP composite.
For specimens with the 50% corrosion defect, 50-0L and 50-3L, the shear cracking has a slightly different pattern than the specimens with 20% corrosion (20-0L and 20-3L). There appears to be a discontinuity in the shear crack on either shear span of both specimens with 50% corrosion, 50-0L and 50-3L, as can be seen in Figures 4.10c and 4.10d. This is probably due to the presence of only one stirrup in each shear span. The BFRP composite did not change this pattern as it is visible in both the damaged (corroded control) specimen (50-0L) and rehabilitated specimen (50-3L).

For the specimens with 100% corrosion, however, the shear crack patterns between corroded control specimen, 100-0L and companion rehabilitated specimen, 100-3L were different (Figures 4.10e and 4.10f). In the damaged (corroded control) specimen 100-0L, there appears to be predominantly one large shear crack in each shear span at failure extending diagonally from approximately the top load points of the spreader beam to the bottom supports. The pattern of shear cracking was significantly different in the corresponding rehabilitated specimen 100-3L. The DIC data indicates that there were three shear cracks with a steeper inclination. These cracks are probably much finer than the single shear crack that occurred in specimen 100-0L. The term “probably” is used here because the strain measurement in the shear span on rehabilitated specimens was that of the BFRP composite itself, not the concrete. Thus, crack widths cannot directly be measured due to the presence of the composite. After the test was completed and the specimen was unloaded, the BFRP was removed from the shear span on one side (Figure 4.11). The cracks closed due to unloading of the beam, however, the location and pattern of the cracks was visible. It was observed that the shear cracks in specimen 100-3L
(Figure 4.11) correlate well with the cracks (lines of strain concentration) shown in the DIC photo (Figure 4.10f).

**Figure 4.10**: DIC analysis

(a) 20-0L

(b) 20-3L

(c) 50-0L

(d) 50-3L

(e) 100-0L

(f) 100-3L

Stirrups

Crack

~750 mm
4.3.2 Load-Deflection Response

The deflection data shown in any load-deflection plot in this paper was obtained from the LVDT attached to the loading actuator and hence, it represents the mid-span deflection. The load-deflection responses of the beam specimens are shown in Figure 4.12. The figure shows the plots for each of the three different damage levels in comparison to the uncorroded control specimen. This figure shows that the pre-cracking behaviour is similar for all specimens, as is the behaviour between post-cracking and yielding of the longitudinal reinforcement. In specimens where yielding of the longitudinal reinforcement occurred, the slope of the load-deflection curve decreased thereafter since the specimen softened. After yielding, the specimen experienced significant inelastic deformation until flexural compression failure occurred. The unloading path is shown for specimens that experienced a flexural compression mode of failure.

Among the beam specimens with 0-20% corrosion damage, the load-deflection response is similar among the uncorroded control (virgin) (0-0L), the corroded control (20-0L), and the rehabilitated specimens (20-3L) as can be found in Figure 4.12a. Minor differences in the ultimate load values and corresponding deflections are observed in
specimen 20-0L. This may be due to the variability in concrete or the presence of some localised defect near the compression face of the beam in the mid-span causing crushing of concrete at a much lower deformation.

For the beam specimens with 50% corrosion, similar behaviours were observed (Figure 4.12b). The deflection at the ultimate load for the rehabilitated specimen (50-3L) in this set is about half of that of the corroded (50-0L) and uncorroded control (0-0L) specimens. This is most likely due to variability in the concrete or presence of localised defects in the beam. This is probably not due to the rehabilitation since this behaviour was not observed in rehabilitated specimen 20-3L.

The corroded control specimen with 100% corrosion, 100-0L, failed in shear at a mid-span deflection of approximately 11 mm and the failure occurred before yielding of the longitudinal reinforcement (Figure 4.12c). There is a slight drop in the load after the shear crack initiated. The specimen then continued to accept higher loads until the shear crack grew and widened sufficiently to reduce the aggregate interlocking. It is clear from the load-displacement plot that the rehabilitated specimen, 100-3L was loaded beyond yielding of the tension reinforcement and sustained significantly higher mid-span displacement before failing in compression.
The ductility of each beam specimen was calculated using Equation 4.1. This equation calculates the ductility ratio ($\mu_\Delta$).

$$\mu_\Delta = \frac{\Delta_u}{\Delta_s}$$  \hspace{1cm} (4.1)

In Equation 4.1, the term $\Delta_u$ is the mid-span displacement at the ultimate load. The term $\Delta_s$ is the mid-span displacement at the service load. It should be noted that this equation has been modified from the traditional definition of ductility where the term $\Delta_s$ is normally $\Delta_y$, the mid-span deflection at yield load. The equation has been modified to

Figure 4.12: Load-deflection behaviours

4.3.3 Ductility

(a) Specimens with 20% corrosion

(b) Specimens with 50% corrosion

(b) Specimens with 100% corrosion
accommodate the 100-0L specimen, which achieved its service load and failed before reaching the yield load.

As can be seen in Table 4.3, the most notable difference in ductility is the difference between the 100-0L and 100-3L specimens. This is due to the difference in failure modes. The BFRP rehabilitation changed the mode of failure from catastrophic shear failure in the elastic range, to a flexural compression failure well beyond yielding. The ductility of this specimen was increased by a factor of approximately four. The ductility indexes for specimens with 20% and 50% corrosion are not consistent, and thus, a conclusion on the change in ductility for specimens which did not experience a shear failure cannot be made.

Among the rehabilitated specimens with 20% and 50% corrosion, the 20-3L specimen exhibited more ductility than the damaged (corroded control) specimen, 20-0L. However, the rehabilitated specimen with 50% corrosion, 50-3L, experienced less ductility than the damaged (corroded control) specimen, 50-0L. This is most likely due to variability in the concrete as noted earlier. All rehabilitated specimens exhibited less ductility than the uncorroded control (virgin) specimen 0-0L.

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Deflection at Service (mm)</th>
<th>Deflection at Ultimate (mm)</th>
<th>Deflection Ductility Index (μΔ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-0L</td>
<td>4.8</td>
<td>50.0</td>
<td>10.4</td>
</tr>
<tr>
<td>20-0L</td>
<td>6.6</td>
<td>43.6</td>
<td>6.6</td>
</tr>
<tr>
<td>20-3L</td>
<td>6.4</td>
<td>51.2</td>
<td>8.0</td>
</tr>
<tr>
<td>50-0L</td>
<td>5.0</td>
<td>54.3</td>
<td>10.9</td>
</tr>
<tr>
<td>50-3L</td>
<td>5.8</td>
<td>32.8</td>
<td>5.7</td>
</tr>
<tr>
<td>100-0L</td>
<td>6.0</td>
<td>11.4</td>
<td>1.9</td>
</tr>
<tr>
<td>100-3L</td>
<td>5.1</td>
<td>42.0</td>
<td>8.2</td>
</tr>
</tbody>
</table>
4.3.4 Load-Strain Response

Figure 4.13 shows the load-strain response of the stirrup (labeled $\varepsilon_s$ in Figure 4.2) for the specimens with 20% and 50% corrosion damage (corroded control) specimens and corresponding rehabilitated specimens in comparison to the same stirrup on the uncorroded control (virgin) specimen, 0-0L. Both Figures 4.13a and 4.13b show that the stirrups remain inactive until about 200-300 kN load when shear cracks began to form. Afterwards, strain in the stirrup increases approximately linearly with increasing load. In each of the 20% and 50% corrosion specimens, the stirrup in the damaged (corroded control) specimens (20-0L and 50-0L) became engaged at the lowest load, followed by the uncorroded control (virgin) specimen (0-0L). The rehabilitated specimen stirrups became engaged at or above the load at which the uncorroded control (virgin) specimen did. This suggests that the BFRP composite assisted in carrying the shear.

![Stirrup load-strain plots](image)

(a) 20% corrosion specimens  
(b) 50% corrosion specimens

**Figure 4.13**: Stirrup load-strain plots

4.3.5 Load Capacity

Table 4.4 shows the service, yield, and ultimate loads, as well as an analysis dividing the shear capacity of the specimens amongst the concrete, steel stirrup, and FRP contributions. The experimental values (E) are compared to the theoretical code predictions.
(T) according to CSA S806-12 [7]. All theoretical calculations were completed by setting any material resistance factors equal to one. As can be seen in the table, the service loads for all specimens are relatively similar. The service load was calculated assuming a service strain in flexural tension steel of 0.0012 [14]. The yield loads are also similar for all specimens, with the exception of 100-0L, which did not achieve its yield load. The ultimate loads are also all comparable except for specimen 100-0L, which failed in shear instead of flexural compression like all the other specimens.

The analysis of shear capacity of the beams shown in Table 4.4 reveals the FRP contribution to shear for the 100-3L specimen, since 100-0L failed in shear. No other experimental comparisons of $V_c$ can be made since the other specimens did not fail in shear. The CSA S806-12 [7] code predictions of $V_c$, $V_s$, and $V_{FRP}$ are all shown alongside the experimental values, where available (see Equations 4.2-4.5). As can be seen in Table 4.4, the theoretical and experimental values of $V_c$ for specimen 100-0L compare well. No comparisons can be drawn between experimental values and theoretical values of $V_s$ since no specimens that contained steel stirrups failed in shear. Also, since specimen 100-3L did not exhibit debonding or rupture failure of the BFRP composite, the theoretical code prediction cannot be compared to the experimental value of $V_{FRP}$. It can be noted, however, that approximately one third of the theoretical code capacity of the BFRP composite was achieved for specimen 100-3L. The CSA S806-12 [7] code specifies that $V_{FRP}$ be calculated assuming a strain in BFRP of 0.004 (Equation 4.5). This is probably to control crack width and ensure adequate aggregate interlock.

The shear capacity is calculated from Equation 4.2 according to S806-12 [7].

$$V_r = V_c + V_s + V_{FRP}$$  \hfill (4.2)
The concrete resistance, $V_c$, is calculated according to Equation 4.3 as follows.

$$V_c = 0.2 \lambda \phi_c \sqrt{f_c'} b_y d$$  \hspace{1cm} (4.3)

The steel stirrup contribution to shear, $V_s$, is calculated according to Equation 4.4.

$$V_s = \frac{\phi_s f_s A_d}{s}$$  \hspace{1cm} (4.4)

The FRP contribution to the shear capacity is calculated according to Equation 4.5.

$$V_{FRP} = \frac{\phi_{FRP} E_{FRP} \varepsilon_{FRPe} A_{FRP} d_{FRP}}{s_{FRP}}$$  \hspace{1cm} (4.5)

where $\varepsilon_{FRPe} \leq 0.004$
Table 4.4: Load capacity

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Mode of Failure</th>
<th>$P_{service}$ (kN)</th>
<th>$P_y$ (kN)</th>
<th>$P_{ult}$ (kN)</th>
<th>$M_{ult}$ (kN-m)</th>
<th>$V_{ult}$ (kN)</th>
<th>$V_c$ (kN)</th>
<th>$V_s$ (kN)</th>
<th>$V_{FRP}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>T</td>
<td>E</td>
<td>T</td>
</tr>
<tr>
<td>0-0L</td>
<td>FT, FC</td>
<td>192.8</td>
<td>353.3</td>
<td>397.4</td>
<td>198.7</td>
<td>198.7</td>
<td>149.2</td>
<td>n/a</td>
<td>149.6</td>
</tr>
<tr>
<td>20-0L</td>
<td>FT, FC</td>
<td>178.2</td>
<td>366.8</td>
<td>422.2</td>
<td>211.1</td>
<td>211.1</td>
<td>149.2</td>
<td>n/a</td>
<td>119.7</td>
</tr>
<tr>
<td>20-3L</td>
<td>FT, FC</td>
<td>198.4</td>
<td>365.9</td>
<td>409.4</td>
<td>204.7</td>
<td>204.7</td>
<td>149.2</td>
<td>n/a</td>
<td>119.7</td>
</tr>
<tr>
<td>50-0L</td>
<td>FT, FC</td>
<td>175.6</td>
<td>367.8</td>
<td>409.4</td>
<td>204.7</td>
<td>204.7</td>
<td>149.2</td>
<td>n/a</td>
<td>74.8</td>
</tr>
<tr>
<td>50-3L</td>
<td>FT, FC</td>
<td>198.3</td>
<td>360.6</td>
<td>401.2</td>
<td>200.6</td>
<td>200.6</td>
<td>149.2</td>
<td>n/a</td>
<td>74.8</td>
</tr>
<tr>
<td>100-0L</td>
<td>S</td>
<td>195.5</td>
<td>n/a</td>
<td>287.1</td>
<td>143.6</td>
<td>143.6</td>
<td>149.2</td>
<td>143.6</td>
<td>0.0</td>
</tr>
<tr>
<td>100-3L</td>
<td>FT, FC</td>
<td>185.2</td>
<td>348.6</td>
<td>390.0</td>
<td>195.0</td>
<td>195.0</td>
<td>149.2</td>
<td>n/a</td>
<td>0.0</td>
</tr>
</tbody>
</table>

FT = Flexural Tension
FC = Flexural Compression
S = Shear
4.3.6 Mode of Failure

Table 4.4 also shows the mode of failure for each specimen in the order that they occurred. The primary mode of failure for all specimens except 100-0L is flexural tension (yielding of steel). The second mode of failure was flexural compression for all specimens except 100-0L. Figure 4.14a shows an example of the flexural compression mode of failure. Specimen 100-0L failed in shear only, as shown in Figure 4.14b.

![Specimen failure modes](image)

(a) Flexural compression  (b) Shear

**Figure 4.14**: Specimen failure modes

4.4 CONCLUSIONS

Based on the results of this test program, the following conclusions can be drawn. The conclusions may be limited to the specific test specimens studied under the scope of this study.

1. Flexural cracks are wider at yield and service loads for beam specimens rehabilitated in shear. It is recommended that shear rehabilitation be accompanied by flexural rehabilitation to avoid this.

2. The effectiveness of BFRP rehabilitation in changing shear cracking behaviour was significant for specimens with a high percentage of corrosion damage (100%). The BFRP rehabilitation increased the number of shear cracks from one wide shear
crack to three finer shear cracks. However, for low percentages of corrosion damage (20% and 50%), the BFRP rehabilitation did not affect the shear crack patterns.

3. The ductility of specimens which failed in shear is significantly less than those which experienced a flexural failure. The rehabilitated beam with a high corrosion percentage exhibited an approximately four-fold increase in ductility.

4. The BFRP composite was effective in changing the mode of failure from a pre-yielding shear mode in specimen 100-0L to a post-yielding flexural compression mode in specimen 100-3L. The BFRP composite was effective in carrying approximately 50 kN of shear, increasing the section capacity from approximately 140 kN to 190 kN for those specimens, respectively.

4.5 ACKNOWLEDGEMENTS

The authors wish to express their appreciation for MEDA Engineering & Technical Services for providing materials and technical assistance necessary for this investigation as well as Sahan Jayasuriya and Amirreza Bastani for conducting the tension tests on the BFRP composite and Eric Hughes for helping with specimen preparation. The financial assistance in the form of graduate student scholarship was received from the Natural Sciences and Engineering Research Council of Canada.

4.6 REFERENCES


CHAPTER 5

FIELD APPLICATION OF BFRP REHABILITATION

5.1 INTRODUCTION

The issue of accelerated corrosion and spalling of concrete has long remained a problem for reinforced concrete (RC) structures. Particularly in cold climates, the heavy use of deicing salts makes this problem a significant concern. In the United States alone, it is estimated that approximately 1 in 9 bridges are structurally deficient, requiring approximately $120 billion to fix [1]. Since the 1970s, the use of fibre reinforced polymers (FRPs) have been proposed to solve the issue of corrosion. FRPs are high strength, lightweight, and corrosion resistant. In particular, the use of carbon (CFRP) and glass (GFRP) have been research and tested in the field for use as a rehabilitation material for concrete bridges. Recently, basalt fibres have been introduced as an alternative to other more traditional fibre types. Basalt fibres are made from igneous basalt rock. Those fibres can then be woven into fabrics (Figure 5.1) for use in FRP systems. Sim et al. (2003) [2] showed that basalt fibres have superior durability properties to carbon and glass fibres. Basalt products also have the potential to be cheaper and more ecofriendly than other types of FRP. Sim et al. (2003) [2] and Lihua et al. (2013) [3] also proved the use of basalt fibre reinforced polymer (BFRP) composite as a flexural strengthening material on RC beams. Thus, the use of BFRP is becoming popular in addressing the issue of corrosion of RC structures, and may be suitable as an alternative to other types of FRP.
The purpose of this study is to investigate the long term feasibility of BFRP composite as a rehabilitation material for RC structures. A local candidate structure was identified and rehabilitated for this purpose. The structure will be monitored on a regular basis to observe the weathering and long term durability of the new BFRP composite material in the field.

5.2 IDENTIFICATION OF CANDIDATE STRUCTURE

The County of Essex and MEDA Engineering, along with the University of Windsor worked together to identify a local concrete structure to apply the basalt fibre reinforced polymer (BFRP) composite to. After considering other structures, including a culvert, and looking for local contractors willing to work with the new product, a suitable bridge and local contractor willing to work with the new product were identified. The Merrick Creek Bridge, located on Country Road 8, west of Country Road 9 in Windsor, Ontario (Figure 5.2) was selected as an ideal candidate structure. The Merrick Creek Bridge is estimated to have been constructed in the 1970s. It is made with eleven precast prestressed T-beam girders topped with a concrete deck and spans 12.9 m. Figure 5.3 shows the section view of the bridge. Figure 5.4 shows the plan view of the bridge. In each figure, the girders are labelled A-K from north to south. The plan view in Figure 5.4 shows how
the girders run east/west, and also indicates the diaphragm pieces in between each girder which run north/south.

Figure 5.2: Merrick Creek Bridge

(a) Location in Windsor, ON

(b) Google street view looking east

Figure 5.3: Bridge cross-section looking east
5.2.1 Initial Condition

Figure 5.5 shows the damage to the bridge as it was inspected prior to rehabilitation. Damage to the structure was typical of weathering to local concrete structures. Spalling of concrete combined with signs of corroded reinforcing steel was observed on the bottom and side faces of the girders, as well as splitting of concrete on the diaphragms (shown in Figure 5.5c). Small round areas of corrosion at the end of each stirrup on the bottom face of the girder were also visible, indicating that stirrups were not closed loops and could have possibly been contributing to spalling of concrete due to lack of confinement. Also evident from the location of the stirrup rust stains on the bottom face of the girders was the insufficient concrete cover, also contributing to spalling and corrosion.
5.3 REHABILITATION METHODOLOGY

5.3.1 Lab Simulation

Previously, numerous test specimens rehabilitated with BFRP had been prepared in the laboratory for destructive testing. For the purposes of research, the previous test specimens had been repaired while the beam specimens were inverted. This was done to enhance safety and ergonomics while working in the lab (Figure 5.6a). In order to prove the suitability of this product in a field application, an “in-situ” demonstration was performed for the contractor. An expended beam specimen was spanned between two supports in the upright position and the composite was applied using the same dry lay-up method used for the research specimens. After viewing the demonstration, the contractor was satisfied that the dry lay-up method was applicable in the field.

Figure 5.5: Bridge condition before rehabilitation
5.3.2 Field Work

The field work for this project was completed between October to November of 2016 and is summarized in Table 5.1. The field work started with chipping of the spalled concrete areas and sandblasting of the reinforcing steel as shown in Figure 5.7. The concrete was first chipped by hand and exposed reinforcing steel was sandblasted. Afterwards, the edges of the patch were cut to produce a sharp 90° corner, instead of smooth transition (as seen in Figure 5.7a). The chipped areas were then patched as shown in Figure 5.8. For deep patches, the area was formed for patching. For less deep patches, the material was simply troweled onto the surface. After allowing the patch material to cure and achieve a certain maximum moisture content, the BFRP composite was applied as shown in Figure 5.9. The BFRP composite was applied using a Sika system. Two epoxy systems were tested: a wet lay-up and dry lay-up. In Figure 5.9b, the rehabilitated girder on the right (girder I) was repaired using the wet lay-up method. The rehabilitated girder on the left (Girder H) was repaired using the dry lay-up method. In addition to the two systems used to study on the individual girders, three sample patches of different FRP composite and different application systems were applied for further study (Figure 5.10a
and 5.10b). The three sample patches consist of one with CFRP dry lay-up, one with BFRP dry lay-up, and one with BFRP wet lay-up. Half of each patch was also painted with a UV resistant paint. These patches were applied to the southwest wingwall (shown in Figure 5.4) where the most sun exposure occurs. The purpose of the patches is to study the long term durability of the different systems. UV paint was not applied to the BFRP composite underneath the bridge since it is not common practice.

<table>
<thead>
<tr>
<th>Day No.</th>
<th>Activities</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Chipping of concrete and sandblasting of rebar (Figure 5.7)</td>
</tr>
<tr>
<td>1-3</td>
<td>Forming and patching of concrete (Figure 5.8)</td>
</tr>
<tr>
<td>21</td>
<td>Application of BFRP composite (Figure 5.9)</td>
</tr>
</tbody>
</table>

(a) Damage on girders after chipping  
(b) Damage on diaphragm after chipping

**Figure 5.7**: Spalled concrete after chipping and sandblasting

(a) Forming girder for patching  
(b) Girder patch after repair

**Figure 5.8**: Major patchwork
The following labour costs were incurred in this project as follows:

<table>
<thead>
<tr>
<th>Description</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial concrete repair</td>
<td>$7,790.00</td>
</tr>
<tr>
<td>Fabric wrap; both wet lay-up application and dry application</td>
<td>$4,656.00</td>
</tr>
<tr>
<td>Working platform</td>
<td>$4,666.00</td>
</tr>
<tr>
<td><strong>Total labour for concrete repairs and fabric wrap:</strong></td>
<td><strong>$17,112.00</strong></td>
</tr>
</tbody>
</table>

The cost of the basalt fabric is $8.47/m². A total of 8 m² was needed for this project, and thus the cost of the fabric is about $70. The cost of epoxy was approximately $700. As
can be seen, the majority of the cost of the project is labour. The material costs are very low relative to the total project cost.

5.5 CONCLUSIONS

The purpose of this field study was to demonstrate the feasibility of BFRP composite to rehabilitate a real concrete structure with natural damage due to weathering. The bridge structure will be monitored periodically to observe the durability of the BFRP composite in the field. Both the damaged girders and the sample patches will be inspected and comparisons can be drawn to CFRP composite. As the rehabilitation was completed less than one year ago, it may be too soon to see how the BFRP composite stands up to the harsh weathering conditions in southwestern Ontario. Only time will tell how durable the new material is. It is expected that at least 5 to 10 years should be allowed to pass before any conclusions are drawn.

5.6 ACKNOWLEDGEMENTS

The authors would like to acknowledge MEDA Engineering & Technical Services and The County of Essex for assisting with the project.

5.7 REFERENCES


CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

The purpose of this thesis was to investigate the structural performance of basalt fibre reinforced polymers (BFRPs) for reinforcing and rehabilitating concrete beams to solve the corrosion problem affecting reinforced concrete structures. Based on the experimental results of the studies conducted for this thesis, the following conclusions are drawn and recommendations for future work suggested.

6.1 BFRP REINFORCED CONCRETE

This study proved the practical viability of using BFRP as a replacement for traditional reinforcing steel and also showed that current design standards may be used to predict the shear capacity of BFRP reinforced beams with varying accuracy. However, this study showed the importance of providing stirrups with bends or hooks to prevent shear failure. Thus, appropriate BFRP stirrups need to be developed to avoid shear failure. Additionally, flexure and shear should be studied separately, and thus it is recommended that beams be designed with either steel or proper BFRP stirrups to study the flexural behaviour of BFRP reinforced beams. A comparison of the flexural behaviour of similar capacity BFRP and steel reinforced beams should be studied, where the beams are designed to carry equivalent loads as per a design standard. This will most likely result in the need to design the BFRP section for deflection rather than ultimate limit states. As shown in Chapters 3 and 4 of this thesis, a detailed comparison of crack widths using digital image correlation for BFRP and steel reinforced beams is also recommended. It may also be of interest to compare the behaviour of BFRP to other FRP types such as GFRP and CFRP.
6.2 BFRP FLEXURAL REHABILITATION AND STRENGTHENING

The use of BFRP composite for flexural rehabilitation and strengthening of reinforced concrete beams was successfully demonstrated with this study. It was shown that BFRP composite can increase the ultimate and yield loads for strengthened beams and restore the service and yield loads for rehabilitated beams. The use of BFRP in flexure also resulted in significantly reduced crack widths, further proving the ability of BFRP to enhance the durability of reinforced concrete structures by hindering further ingress of salt solution in concrete flexural elements in cold climates. However, it was also found that if insufficient cross-strapping is provided, beams can fail by sudden interfacial debonding, which should be avoided. It was shown that a scheme where the flexural composite is cross-strapped along its entire length is most effective. Future work should focus on studying the strengthening effect of BFRP with varying flexural reinforcement ratios and comparisons should be made to equivalent strengthened beams with other common FRP materials such as GFRP and CFRP.

6.3 BFRP SHEAR REHABILITATION

In this study, BFRP composite was effectively used to rehabilitate shear deficient reinforced concrete beams. It was found that, for the specimen with a high percentage of corrosion damage, the BFRP was effective in increasing the shear capacity of the beam and changing the mode of failure from shear to flexural compression, thus significantly increasing the ductility of the beams. It was also found that the application of the BFRP significantly changed the shear crack pattern in the specimen with a high percentage of corrosion damage. However, it was found that flexural cracks are wider in specimens with shear rehabilitation, and thus it is recommended that shear rehabilitation always be
accompanied by flexural rehabilitation, since it was found in Chapter 3 that the application of BFRP composite in flexure significantly decreases crack widths. Further study should focus on the application of BFRP composite to *shear critical* beam specimens and should be designed in such a way as to fully utilize the capacity of the composite and cause either a debonding or rupture failure of the composite. It is also recommended that a study be conducted that compares the use of BFRP to other common FRP types such as GFRP and CFRP.
APPENDIX A: LABORATORY METHODS

I. Steel Rebar Tension Tests

In this thesis, where it is noted that ASTM A370-14 was followed to determine the mechanical properties of steel rebar, the following test procedure was used. Figure A.1 shows the setup for the test. Specimens were cut to approximately 150 mm in length and tested in a 300 kN universal tension machine. A 50 mm extensometer was used to measure the strain. The specimens were loaded at a rate of approximately 2.5 mm/min until rupture.

![Figure A.1: Steel rebar tension test setup](image)

II. BFRP Rebar Tension Tests

(a) Specimen Preparation

Where it is noted in Chapter 2 that ASTM D7205-11 was followed to determine the mechanical properties of the BFRP rebars, the following procedure was used to prepare the test specimens. The rebars were cut to length and duct tape was wound around each point of the bar where the tube would end (four points in total). The tape was wound until it matched the inside diameter of the tube and the bar was slid inside the tube. The bars were then partially pulled out from the tube in order to pour in grout and stored horizontally until the grout had set up. A concrete demolition grout (called ECOBUST) was used, which expands considerably to induce compression between the bar and tube. An epoxy resin
could have been used in place of the expanding demolition grout. Figure A.2 shows the specimens before and after grouting. After the specimens were grouted, they were painted with white primer and a speckle pattern paint (Krylon Make It Stone!®) for DIC.

![Figure A.2: BFRP rebar specimen preparation](image)

(a) Rebars before grouting  
(b) Rebars after grouting

(b) Testing

Most details of the test procedure are given in Chapter 2. The intact and ruptured specimens are shown in Figure A.3 below. A board was placed behind the specimens with a scale so that an area of interest of any size could be selected after with DIC analysis. The camera was aimed at the specimen so that at least 100 mm was visible. Appropriate lighting was also placed behind the specimen, careful to avoid producing any shadows. The loading rate was approximately 7 mm/min. The specimens were tested in a 600 kN universal testing machine with hydraulic grips that could accommodate a 50 mm diameter specimen. Photos were captured every three seconds for DIC using a Cannon T3i camera.
III. BFRP Composite Tension Tests

(a) Specimen Preparation

Where it is noted that ASTM D3039-14 was followed to determine the mechanical properties of the BFRP composite, the following procedure was used to prepare the specimens. The fabric was first cut to length and immersed in epoxy (Figure A.4a). Tabs made of circuit board material were also epoxied at the ends to avoid stress concentrations. The specimens were allowed to cure for 7 days and then cut into 15 mm wide strips. The specimens were painted with white primer and speckle paint for DIC (Figure A.4b).
Most details of the testing are covered in Chapters 3 and 4. Photos were captured every three seconds for DIC using a Cannon T3i camera. Figure A.5 shows a typical ruptured specimen.

IV. Concrete Compressive Strength

Where it is noted that ASTM C39-15 was followed to determine the compressive strength of concrete, the following procedure was followed. The concrete cylinders were
cast at the same time as the beam specimens. A minimum of eight cylinders were cast for each set of beams. The cylinders were rodded and allowed to cure for a few days before being demoulded. The cylinders were then capped and tested in compression on day 28. Figure A.6 shows the test setup for determining the concrete compressive strength. Each specimen was loaded at approximately 0.25 MPa/sec until failure.

![Figure A.6: Concrete compressive strength test setup](image)

V. Beam Specimen Preparation

(a) Formwork

The formwork for beam specimen casting was made from standard 2×4 lumber and ¾” concrete forming board (as shown in Figure A.7). Ties were place on the top in three locations to prevent excessive deflection during casting.
(b) Simulation of Spalling

For the flexure and shear rehabilitation studies presented in Chapters 4 and 5, spalling of concrete was simulated by using inserts made of rigid insulation foam board that was CNC machined from an AutoCAD template. Figure A.8 shows the process of machining and the finished foam insert.

![CNC machining foam insert](image1)
![Foam insert](image2)

**Figure A.8:** Machining of foam insert

(c) Simulation of Corrosion

Corrosion of reinforcing steel was simulated by machining using a vertical mill. Figure A.9 shows the process of machining of longitudinal reinforcement and stirrups.
(d) Cage Construction

Rebar cages were tied using traditional steel rebar ties for steel cages and plastic cable ties for BFRP rebar cages. Plastic rebar chairs were then placed on the bottom and sides of each cage to secure it within each form and provide appropriate cover as shown in Figure A.10.

(e) Strain Gauge Placement

To apply the strain gauges, areas were first sanded with 60 and 120 grit sand paper. The gauges were then applied with appropriate adhesive and covered with electrical tape.
The strain gauges used had a gauge length of 5 mm. The strain gauge wire was soldered to the ends and covered with electrical tape. The wires were labeled and routed through the cages to avoid damage during casting. The gauges were then covered with a few layers of duct tape to protect them during casting.

![Strain gauges after placement](image)

**Figure A.11**: Strain gauges after placement

(f) Casting

Beams were cast in a laboratory setting. The concrete truck was reversed into the lab and concrete was placed in the forms and vibrated extensively to remove any air voids. Strain gauge wires were strung outside the forms and placed in a plastic bag to prevent damage during casting. After casting, the top of each beam was troweled smooth and a stamp was applied to identify each beam specimen. Lifting hooks were also placed in each beam specimen during casting. Figure A.12 shows the casting process at various stages.
Concrete Slump Test

The concrete slump test was performed for each set of beams during casting. A slump of approximately 50 mm was targeted during casting. Figure A.13 shows the test.

Curing

Beams were cured for a minimum of 3 days in the forms. The beams were covered with damp burlap and then with a plastic sheet (Figure A.14). Water was applied to the burlap twice on the 2nd day of curing since the burlap tends to dry up from the heat of hydration.

Figure A.12: Casting beams specimens

Figure A.13: Concrete slump test
(i) Patchwork

Patchwork was completed while the beam specimens were inverted (flexure) or on their side (shear). The patch was cleaned with a wire brush to remove any loose concrete and blasted with compressed air. A small piece of wood was placed on the sides while forming the patch. The mortar was then mixed and placed in the patch and vibrated to consolidate it. King Super-Top repair mortar was used for all patchwork. Figure A.15 shows the process of patching.

(j) Application of BFRP Composites

i. Priming
Priming of the areas to be patched was always performed 24 in advance of epoxy application. BASF MasterBrace P 3500® was used as primer. The areas to be primed were first blasted with compressed air to remove any loose material. Figure A.16 shows the process of priming. The priming was completed using a paint roller. Working time for the primer was relatively short (~20 min), so it was important to work fast.

![Image](image1.png)
(a) Priming flexure beam  
(b) Priming shear beam

**Figure A.16:** Priming beams specimens

ii. Epoxying

Epoxying was always performed about 24 hours after priming. MasterBrace SAT 4500® epoxy was used. The fabric was first cut to length and laid out on a table. The two-part epoxy was then mixed in a bucket and applied to the beam. First a thick coat of epoxy was applied directly to the primed surface with a paint roller and the fabric was laid on top and rolled into the epoxy. Subsequent layers were applied using the same method. Figure A.17 shows the process of epoxying. Working time for the epoxy was approximately 40-60 min, so it was important to work quickly. After the epoxy was placed, thick plastic (vapour barrier) was pressed firmly into the epoxy to create a smooth surface to which strain gauges could be applied. The epoxy was allowed to cure for 24 hours before removing the plastic, and for 7 days before testing.
In this thesis, where it is noted that the digital image correlation technique was used for strain measurement on beam specimens, the general procedure for painting is shown below. First, the beams were painted with a white primer paint as shown in Figure A.18a. They were then painted with Krylon Make It Stone!® black and white stone speckle paint to produce a random speckle pattern on the surface of the beam as shown in Figure A.18b.
Figure A.18: DIC paint application

(a) Primer application

(b) Speckle paint application
APPENDIX B: ABBREVIATIONS AND NOMENCLATURE

The following abbreviations and symbols are used in this thesis:

AFRP = Aramid Fibre Reinforced Polymer
BFRP = Basalt Fibre Reinforced Polymer
CFRP = Carbon Fibre Reinforced Polymer
D = Debonding
FC = Flexural Compression
FRP = Fibre Reinforced Polymer
FT = Flexural Tension
GFRP = Glass Fibre Reinforced Polymer
LVDT = Linear Variable Differential Transformer
R = Rupture
S = Shear
$E_f$ = modulus of elasticity of FRP
$E_s$ = modulus of elasticity of steel
$E_u$ = energy absorption at ultimate load
$E_y$ = energy absorption at yield load
$f'_c$ = concrete compressive strength
$f_{fu}$ = rupture strength of FRP
$f_u$ = rupture strength of FRP
$f_y$ = yield stress of steel
$M_c$ = moment corresponding to a maximum concrete compressive strain of 0.001
\( M_{cr} \) = cracking moment

\( M_r \) = resisting moment

\( M_{ult} \) = ultimate moment capacity of the section

\( P_{max} \) = maximum sustained load

\( P_{service} \) = load at service strain in steel of 0.0012

\( P_{ult} \) = maximum sustained load

\( P_y \) = yield load

\( V_c \) = concrete contribution to shear resistance

\( V_f \) = FRP stirrup contribution to shear resistance

\( V_r \) = shear resistance

\( V_s \) = steel stirrup contribution to shear resistance

\( \Delta_s \) = mid-span deflection at service load

\( \Delta_u \) = mid-span deflection at ultimate load

\( \Delta_y \) = mid-span deflection at yield load

\( \varepsilon_{FRP} \) = strain in FRP

\( \varepsilon_{fu} \) = ultimate strain of FRP

\( \varepsilon_s \) = strain in stirrup

\( \varepsilon_{service} \) = service strain in steel = 0.0012

\( \varepsilon_T \) = strain in tension steel

\( \mu_\Delta \) = deflection ductility index
$\mu_{tr}$ = energy ductility index

$\rho$ = longitudinal reinforcement ratio

$\rho_b$ = balanced reinforcement ratio

$\rho_{fb}$ = balanced FRP reinforcement ratio

$\psi_c$ = curvature at $M_c$

$\psi_{ult}$ = curvature at $M_{ult}$
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