Flexural strengthening of reinforced concrete beams with basalt fibre reinforced polymers

Eric William Hughes
University of Windsor

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FLEXURAL STRENGTHENING OF REINFORCED CONCRETE BEAMS WITH BASALT FIBRE REINFORCED POLYMERS

By
Eric Hughes

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

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DECLARATION OF ORIGINALITY

I hereby certify that I am the sole author of this thesis and that no part of this thesis has been published or submitted for publication.

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ABSTRACT

This thesis presents the experimental results of laboratory testing conducted on full-scale concrete beams which were strengthened with Basalt Fibre Reinforced Polymer (BFRP) fabrics. The goal was to determine the viability of using external BFRP fabric reinforcement to strengthen flexurally controlled concrete members in-situ. The use of BFRP as an external strengthening material is compared to other materials such as glass (GFRP) and carbon (CFRP) fabrics which are currently widely accepted strengthening materials. Two parameters were varied during the research: the internal steel reinforcement ratio, and the external BFRP layers, to study the interaction between the two. Using BFRP showed excellent results as a flexural strengthening method. The moment capacity of the strengthened beams was found to increase by up to 79% over the control beam for the yield strength, and by up to 120% over the control for the ultimate strength. The yield deflection of the strengthened beams remained similar to the control beam without much reduction or increase, and the ultimate load deflection was increased by up to 140% over the control specimen. This is a key finding as previous tested discussed in the literature review found that both the yield and ultimate deflections of strengthened beams was greatly reduced when using GFRP and CFRP fabrics. When compared to the applicable Canadian and American FRP design guidelines, it was found that the Canadian code needs to be updated to reflect the same process used to determine the FRP design strain used in the American code. With this update, both codes can accurately predict the strength increase found in these specimens. When strengthening flexural members with BFRP fabrics, the beams exhibit increased load-deflection stiffness. It is recommended to also strengthen the beams shear capacity when flexurally strengthening a concrete member to maintain beam integrity and ductility.
I dedicate this thesis to all the people who helped encourage me to pursue my passions and assisted me in getting this far. Without this assistance from my family and friends it would not have been possible to come this far. It is my hope to help push the engineering profession forwards, and that this research can be taken even further by future generations.

♦♦♦

*Science can amuse and fascinate us all, but it is engineering that changes the world.*

~ Isaac Asimov
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The following abbreviations and symbols are used in this thesis:

- $a$ = Depth of Equivalent Rectangular Stress Block
- $A_{f,\text{anchor}}$ = Area of FRP Anchorage
- $AFRP$ = Aramid Fibre Reinforced Polymer
- $A_{FRP}$ = Cross-Sectional Area of FRP
- $A_s$ = Area of Tensile Steel
- $A's$ = Area of Compressive steel
- $A_v$ = Shear Stirrup Area
- $b$ = Beam Web
- $BFRP$ = Basalt Fibre reinforced Polymer
- $c$ = Depth of Neutral Axis
- $C_c$ = Compressive Force in Concrete
- $CFRP$ = Carbon Fibre Reinforced Polymer
- $C_s$ = Force in Compression Steel
- $C_v$ = Coefficient of Variation
- $d$ = Depth of Tensile Rebar
- $d'$ = Depth of Compressive Rebar
- $d_c$ = Distance of Extreme Tensile Fibres to Center of Closest Longitudinal Wire
- $DIC$ = Digital Imaging Correlation
- $d_v$ = Effective Shear Depth
- $E_c$ = Concrete Modulus of Elasticity
- $E_{FRP}$ = FRP Modulus of Elasticity
- $E_s$ = Steel Modulus of Elasticity
- $E_{\text{service}}$ = Service Energy
- $E_u$ = Ultimate Energy
- $f'_c$ = Concrete Compressive Strength
- $FRP$ = Fibre Reinforced Polymer
- $f_s$ = Service Load
\( f_y \) = Yield Strength of Steel  
\( GFRP \) = Glass Fibre Reinforced Polymer  
\( h \) = Height of Beam  
\( HFRP \) = Hybrid Fibre Reinforced Polymer  
\( I \) = Inertia  
\( k_v \) = Bond Reduction Coefficient  
\( l \) = Arc Length  
\( l_a \) = Lap Length  
\( LVDT \) = Linear Variable Distance Transducer  
\( Mr \) = Moment Resistance  
\( Mr_{service} \) = Service Moment of Resistance  
\( Mr_{ultimate} \) = Ultimate Moment of Resistance  
\( n_f \) = Number of FRP Layers  
\( PF \) = Performance Factor  
\( S \) = Stirrup Spacing  
\( t_f \) = Fibre Layer Thickness  
\( T_{FRP} \) = Force in FRP  
\( T_s \) = Force in Tensile Steel  
\( V_c \) = Shear Concrete Resistance  
\( V_r \) = Shear Resistance  
\( V_s \) = Shear Steel Resistance  
\( z \) = Quantity Limiting Distribution of Flexural Reinforcement  
\( \alpha_1 \) = Average Stress Ratio in Rectangular Compressive Block  
\( \beta \) = Shear Resistance of Cracked Concrete  
\( \Delta_{service} \) = Service Midspan Deflection  
\( \Delta_{ultimate} \) = Ultimate Midspan Deflection  
\( \varepsilon_{cu} \) = Concrete Ultimate Strain  
\( \varepsilon_i \) = Initial Structural Member Strain  
\( \varepsilon_{FRP} \) = FRP Strain  
\( \varepsilon_s \) = Tensile Steel Strain
\( \varepsilon_s \) = Compressive Steel Strain

\( \varepsilon_y \) = Yield Strain in Steel

\( \theta \) = Angle of inclination of Diagonal Stresses in Member

\( \lambda \) = Concrete Density Factor

\( \mu \) = Mean

\( \mu_\Delta \) = Deflection Curvature

\( \mu_E \) = Energy Curvature

\( \mu_\phi \) = Curvature Ductility

\( \rho \) = Reinforcement Ratio

\( \sigma \) = Standard Deviation

\( \sigma_c \) = Stress in Concrete

\( \sigma_s \) = Stress in Steel

\( \sigma_{FRP} \) = Stress in FRP

\( \phi_c \) = Concrete Resistance Factor

\( \phi_{FRP} \) = FRP Resistance Factor

\( \phi_s \) = Steel Resistance Factor

\( \phi_u \) = Ultimate Curvature

\( \psi \) = Curvature
CHAPTER 1

INTRODUCTION

1.1 OVERVIEW

The prevalent use and structural effectiveness of steel reinforced concrete as structural elements is widely known and accepted throughout the structural engineering field across the world. Reinforced concrete structures, along with wood, masonry, and steel, are among the most commonly used in the civil and structural engineering field. This is observed from their use in heavy civil construction projects like the Hoover Dam in Nevada, USA, to the transportation field and such projects like the Herb Grey Parkways project in Windsor, Ontario. Reinforced concrete has been used and counted on for its incredible strength and durability, along with its low cost of production. Although concrete itself is very strong in compression, without the internal steel reinforcement it would not be nearly as strong and popular as it is today. This reliance on the use of this internal tensile reinforcement is a major drawback. For years the use of steel rebar reinforcement has been the most common way to compensate for concrete’s lack of tensile strength, but the steel inside the concrete is vulnerable to oxidation causing it to rust. This rusting process can compromise both the steel and concrete as it can reduce the effective cross-sectional area of the steel weakening the member. Furthermore, the expansive forces of this process can cause cracking and spalling in the concrete as the oxidation causes the steel to expand. This process can end up compromising structural members, and if left unchecked, it can compromise whole structures as well, leading to costly repairs or replacements. This problem is especially prevalent in cold or corrosive environments such as Canada, northern part of the USA. and northern Europe, where the winter seasons can create freeze and thaw cycles, causing water in the concrete to expand.
and contract as it transitions through its solid and liquid state, expanding and cracking the concrete.

In some areas within these climates, corrosive materials are also used to melt dangerous ice on these structures which can further accelerate the corrosion and spalling in the steel and concrete. While this can be considered by some to be the natural life span of the structure which is considered during the design phase, if the life span can be extended, it would be more sustainable and reduce infrastructure costs.

One promising alternative would be to use corrosive-resistant tensile reinforcement in reinforced concrete structures. Although tensile reinforcement has predominantly been steel, it does not necessarily have to be. There are two main reasons to use tensile reinforcement in concrete structures: to allow the member to fail in a ductile manner, and to compensate for concrete’s lack of tensile strength. Steel has been popular for many years mainly due to its high elastic modulus, allowing it to carry a high capacity load before failing, and its ductility or ability to deform visibly and experience strain hardening before its ultimate rupture. Many current researches have focused on finding alternative non-corrosive material that can provide tensile strength and retain as much of the ductility as possible. Some of the most promising advances in this research has been on the use of fibre reinforced polymers (FRPs), as internal reinforcement bars, and externally bonded fabrics.

1.2 BACKGROUND ON FIBRE REINFORCED POLYMERS (FRPs)

FRP composite material can be made of many different fibres such as glass, carbon, basalt. However, any FRP material is composed of continuous unidirectional or bidirectional fibres, impregnated with an epoxy resin matrix. These composite materials generally exhibit much better corrosive resistance properties than steel. These materials have been found to have many improved durability aspects over steel, including excellent resistance to weather, alkalinity resistance, and
high resistance to acidic and corrosive environments [1]. The corrosive resistance and durability of these FRP laminates is discussed in more detail in the literature review, as it was not a part of this research, but is a widely researched field.

One of the main fields which is now being researched is the application of these FRPs as a flexural strengthening method to either rehabilitate or strengthen existing and new steel reinforced concrete members. When using this method for flexural strengthening the main limitations which are considered is whether the same characteristics and behaviour can be obtained from beams strengthened or rehabilitated with an FRP, as are observed from the same beams with only internal steel reinforcement.

1.2.1 FIBRE TYPES

There are a variety of materials which can be combined with epoxies to form FRP, with the most commonly used materials being comprised of Carbon fibres or E-Glass or S-Glass fibres. Other FRP can be comprised of Aramid fibres which can cover a variety of synthetic fibres such as Kevlar and Technora, each having their own unique mechanical properties [2], and Basalt, which is a new fibre introduced in civil engineering applications.

Figure 1.1 [3] shows the stress-strain of some commonly used fibres and their relative elastic modulus. The exact modulus of the FRP used will depend on the manufacturing process of the fibres. Figure 1.2 shows ranges of mechanical properties of the constituent materials used in FRP [4]. As can be found in this figure, fibres can hold a very high stress, and are the primary contributor to the composite’s tensile strength. The ultimate strain varies greatly in the matrix based on the material used. The fibres fail in a linear elastic fashion, and this trend is found also in the FRP when the fibres and matrix are combined.
Figure 1.1: Stress-Strain Curves of Typical Reinforcing Fibres: a) Carbon (High Modulus); b) Carbon (High Strength); c) Aramid (Kevlar 49); d) S-Glass; e) E-Glass; f) Basalt [3]

Figure 1.2 Stress-Strain Relationship for Fibrous Reinforcement and Matrix [4]
1.2.2 MANUFACTURING PROCESS

For Basalt Fibre Reinforced Polymers (BFRP), the base material comes from magma which has been forced to the earth’s surface where they solidify and is a very abundant material. Fiore et al. [1] stated that its melting point of basalt fibre varies between 1500–1700°C, and when it is melted in a furnace it consumes less energy than carbon and glass. It also has no additional additives making it cheaper than both carbon and glass to produce [1]. The process which is used to draw raw basalt out into basalt fibres is called the continuous spinning method, or the spinneret method, which is very similar to how glass fibre is produced which greatly decreases the startup cost of mass-producing basalt fibres as the infrastructure is already present. A simplified version of the manufacturing process using continuous spinning method is shown in Figure 1.3.

Figure 1.3: A Simplified Scheme of a Basalt Fibreization Processing Line [1]
When processing raw Basalt rocks into this fabric the rocks are first crushed and loaded into a silo (1). The material is then transferred to a loading station (2), where it is then transported to processing plant (3). From there it goes through batching stations (4) and into the initial melt process to heat the raw material (5). Once the material has been heated up, it travels to the secondary heat zone (6), which has precise temperature control, to ensure the quality of the post-processed material and the crystallization which can form from the quenching process. After the Basalt has reached the correct temperature, it is then drawn out into filaments (7), sized for the correct diameter (8), and combined into strands of fibre (9). After this occurs the fibres and wound into rolls which are ready to be distributed (10 & 11).

1.2.3 MATRIX

The matrix is used to attach the fibres and transfer the stress and strains between them. The matrix then transfers the stresses in the fabric, through the matrix into the structural substrate, through in-plane shear stresses. There are two types of matrices; thermosetting and thermoplastic, and although thermoplastic can be reheated in order to reshape them, this comes at the expense of a reduction of mechanical properties [5]. Hence, the most commonly used for structural applications is the thermosetting matrix. Inside this category, three resins are used to make the FRP matrix; epoxy, polyester, or vinyl ester matrices. Epoxy resins exhibit the best mechanical properties, as well as having a high moisture absorption resistance, and excellent resistance to corrosive liquids and environments along with great durability. This combination of characteristics has caused epoxy resins to be the most commonly used matrix [1], and was used in this study.

1.2.4 MECHANICAL PROPERTIES

The mechanical properties of FRP can vary greatly between the different fibre types. When the modulus of FRP fabrics, such as Glass Fibre Reinforced Polymer (GFRP), Carbon Fibre
Reinforced Polymer (CFRP), and Aramid Fibre Reinforced Polymer (AFRP), are compared with the modulus of common reinforcing steel as in Figure 1.4, the major differences become apparent. While the stress strain curves vary between the fibres, none of the FRP material has a higher elastic modulus than steel. Table 1.1 shows the comparison of average strength and modulus values for the commonly used fabrics, and this difference in modulus becomes apparent. However, while the initial modulus is less than that of steel, due to the linear stress-strain which these FRP materials experience and the lack of a yield point, these fabrics can reach much higher stresses before failure. This can be a great advantage over steel when rehabilitating and strengthening reinforced concrete beams, if the proper precautions are taken to ensure there is no brittle failure.

![Figure 1.4: FRP and Steel Modulus Relationships](image)

**Table 1.1: Mechanical Properties of Reinforcing Steel and FRP**

<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>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>
Other researchers have also studied combining certain fibres together to create Hybrid Fibre Reinforced Polymers (HFRP). The most popular fibres to combine have been glass and carbon fibres to try and retain the high strength of carbon fibres, and the good ductility of glass fibres, as shown in Attari et al. [6].

Although the FRP fabric is a brittle material, when used to strengthen a reinforced concrete beam, the failure of the concrete beam can still ductile as long as it is not over reinforced. These phenomena will be discussed later in the thesis. Alternatively, the stiffer the FRP is the more the global stiffness of the concrete beam will increase. This can be potentially very dangerous as concrete is a very brittle material, and thus, the ductility of concrete members should be preserved as much as possible. This is one of the main benefits of a lower elastic modulus but higher strain fibres for FRPs such as glass and basalt. These fibres help to preserve the ductility of the beam, while still providing a moderate to high increase in the beam’s flexural strength.

1.3 BASALT FIBRE REINFORCED POLYMER (BFRP)

The overall strength and mechanical properties of this material are dependent on the rate at which the material is quenched, which impacts the crystallization of the material [1]. Basalt fibres were researched as early as the 1950s by the Soviet Union [1], but their use in the rehabilitation and strengthening of structures is a new area of research. Basalt is very appealing for this application due to its moderate modulus, high ultimate tensile strain, and cheaper cost of production.
1.3.1 OVERVIEW OF BFRP MATERIALS

Basalt fibre reinforcement can come in many different forms. Figures 1.5 a) and c) show unimpregnated basalt fibres in its fabric and chopped forms, with fabrics available with both unidirectional and bidirectional weaves. The chopped fibres are generally used inside concrete mixes to increase the modulus of rupture, while the fibres are used as strengthening and rehabilitation laminates. Figures 1.5 b) and 5d) show forms of basalt rebar and mesh, which has been impregnated with a resin. BFRP rebars can be used internally in place of steel rebar, while a bidirectional mesh is often used in near surface mounted reinforcement scenarios.

![Figure 1.5: Forms of Basalt Reinforcement](image)

1.3.2 BFRP MECHANICAL PROPERTIES

One of the major reasons that BFRP is being studied for this application, is because it has a moderate modulus of elasticity, and a high strain at rupture when compared to other fibre
composites. The increase in the ultimate rupture strain can give the concrete beam a chance to undergo a higher deflection before the ultimate rupture of the laminate. This is especially critical for concrete structural members since they tend to fail in a more brittle manner, when compared to structural members made of steel. The exact elastic modulus and failure strain will vary however, as it depends on the quality of the individual materials and composite. There is some variation in the literature as to what the exact numbers are, but Table 1.2, shows the material properties of BFRP fabric which have been tested in the laboratory at the University of Windsor. These results are presented in Figure 1.6. These values were taken from five tests which conformed to the ASTM standard D3039/D3039M for the testing method for the tensile properties of polymer matrix composite materials [7]. These values are presented below based on the mean ($\mu$), standard deviation ($\sigma$), and coefficient of variation ($C_v$) in Table 1.2.

**Table 1.2: Experimental BFRP Mechanical Properties**

<table>
<thead>
<tr>
<th>Material Properties</th>
<th>$\mu$</th>
<th>$\sigma$</th>
<th>$C_v$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of Elasticity (GPa)</td>
<td>21</td>
<td>1.37</td>
<td>0.06</td>
</tr>
<tr>
<td>Ultimate Strain (%)</td>
<td>2.35</td>
<td>0.15</td>
<td>0.07</td>
</tr>
<tr>
<td>Ultimate Stress (MPa)</td>
<td>460</td>
<td>28.7</td>
<td>0.06</td>
</tr>
</tbody>
</table>
When comparing Table 1.1 and Table 1.2, the increase in ultimate strain over the CFRP, GFRP and AFRP fabrics is observed, and it is this increase in ultimate strain which will help the concrete beam to capture an increased amount of deflection. Typical ultimate strains for CFRP is 0.9%, and GFRP is 1.75%, while here BFRP can reach up to 2.35% which is a significant increase.

1.3.3 DURABILITY OF BFRP

One of the main reasons for replacing steel with a corrosive resistant material when strengthening reinforced concrete, is due to the unfavorable durability characteristics steel can have, especially with resistance to corrosive environments. This degradation is damaging to the steel and the concrete, and so an alternative is being researched to avoid or impede this structural degradation. Testing and research on the material properties and durability were not a focus for this research. This has however, been a widely studied, and it is discussed in Chapter 2.
1.3.4 COST

Cost is a driving factor for much of the innovation which has happened in the past and it is no different here. With much of the infrastructure in many countries like in Canada aging, massive investments will need to be made to maintain the existing infrastructure. Methods of using Carbon and Glass FRP can be very expensive, due to the manufacturing process or the scarcity of resources. This is not the case with Basalt, since it is a naturally occurring material which is found all over the world due to being a byproduct from volcanic activity. The manufacturing process is very similar to glass, and hence, the cost to manufacture this material is low due to the infrastructure already being in existence [1]. Currently the University of Windsor has been able to purchase BFRP fabric for as low as $8.50/m², while the similar CFRP fabric cost just over $100/m² [5]. This is a massive difference in cost, and it is expected that the cost of BFRP will decrease as it gains more popularity as a strengthening material and more companies start to produce it. If BFRP can prove to be efficient and effective at strengthening and rehabilitating reinforced concrete structures, this can cut infrastructure repair and maintenance costs significantly.

In 2016, an updated Canadian Infrastructure Report Card was released, and this report assesses what state Canadian infrastructure is currently in, as well as giving estimates on present and future costs to maintain and upgrade Canadian infrastructure. In the latest report, the entire value of core Canadian infrastructure was estimated at $1.1 trillion CAD [8]. Of this number, nearly 35% of all these assets already fall into categories which need attention right now, representing approximately $385 billion CAD in costs to solve this problem. Apart from this, the next category of infrastructure which it presents are in need of attention in the next 10 to 15 years, and represent another approximately $247 billion CAD [8]. These values can be observed in Figure 1.7.
Figure 1.7: Summary of Average Physical Conditions Rating of Infrastructure in Canada [8]

1.4 OBJECTIVE

The objective of this research is to study the effect of strengthening reinforced concrete flexural beams with BFRP fabric. The purpose of this study is to determine the effectiveness and efficiency of BFRP fabrics to increase the flexural strength of concrete beams. These materials and repair method were applied to reinforced concrete beam specimens built and tested in the structural engineering laboratory at the University of Windsor. The objective is to determine a quantitative relationship between the internally reinforced concrete beams and the externally bonded FRP fabric. The various structural codes and standards from both Canada and the United States of America was adapted to reflect and predict the accurate strength and ductility capacities of these specimen. This prediction was verified from the data, to ensure the existing codes are reliable and accurate.

To study the effect of strengthening reinforced concrete beams with BFRP fabrics, two parameters were considered. One was the three internal reinforcement ratios of 0.5%, 0.77%, and
1.0\%, and the second was the amount of external BFRP fabrics. The test matrix was set this way to determine the effect these two variables have on each other and the overall beam behavior.
CHAPTER 2

LITERATURE REVIEW

The repair, rehabilitation, and strengthening of concrete structural elements using composite materials is a rapidly expanding area of study within the structural engineering field. These composites can be bonded externally to structural members, which can be done either during the construction phase or later in the members’ structural lifetime to retrofit and repair structures. Using composite fabrics can greatly reduce the cost of repairing aging structures, as well as reduce the section size needed for structural members for the same strength. The two most widely used materials for composite fabrics currently are Carbon Fibre Reinforced Polymers (CFRP), and Glass Fibre Reinforced Polymers (GFRP).

2.1 FIBRE REINFORCED POLYMER (FRP) MATERIAL PROPERTIES

2.1.1 MECHANICAL PROPERTIES

This study focuses on the feasibility of using Basalt Fibre Reinforced Polymers (BFRP) fabric, as an alternative for GFRP and CFRP fabrics, when repairing and strengthening flexurally deficient concrete members. This method could greatly reduce the cost of repairing and strengthening concrete members using externally bonded FRP fabric, as the price of carbon and glass fabrics are much higher than the cost for basalt fabrics [1]. BFRP fabrics have been shown to exhibit tensile strengths of about 30% of carbon, and just over 60% of high strength S-glass fibres [9]. Even with this reduction in strength however, basalt has many advantages which are appealing as a construction material. A study completed by Fiore et al. [1] compared the properties of basalt fabrics to E-glass fabrics and found that basalt fabrics outperformed the glass fabrics in many categories. The study found that the basalt fabrics showed an increase of 15% to 40% in the
elastic modulus, tensile strength, and fracture strain with only a slight increase of 10 % to the fabric density [1]. It is also shown that basalt fabric has a higher ultimate elongation, making it the most ductile of these three materials (basalt, glass, carbon) [1,9]. This will be particularly important when trying to maintain a ductile failure mode in concrete members to maintain the design safety. While there can be a significant reduction in tensile strength when compared with high strength fabrics such carbon fabric, basalt has major advantages in terms of the durability and strain which basalt fabric can sustain before failure.

2.1.2 DURABILITY

Fiore et al. [1] and Sim et al. [9] conducted tests to determine the mechanical and chemical durability properties of dry basalt fibre. The study found that the basalt fabric provided a good weather resistance to ultra-violet light through accelerated exposure. The weather resistance of basalt fibre was found to be slightly better than the glass fibre, but it was slightly less than carbon fibre [1,8]. With regards to alkaline resistance, basalt fibre was found to degrade, providing a reduction in volume at a rate similar to glass [9]. Ramachandran et al found the opposite, that when exposed to alkali environments similar to what would be experienced in concrete, basalt fibre exhibited a good resistance to alkaline environments even at elevated temperatures [10]. When the basalt fibre does degrade in alkali environments, the degeneration is less severe than what is exhibited in similar glass fibres, and the basalt retains more of its strength after degradation [1,9,10]. Through accelerated aging material testing, the basalt fabric was shown to have a better resistance than the glass fibre.

One of the major advantage’s basalt has however is its excellent thermal stability, much greater than both carbon and glass fibres. Hence, basalt fibre products can carry a much higher ultimate load than both alternatives when subjected to fire load. This makes basalt fabric an
excellent material for fire-proofing work [1,9]. After exposing fibres to 600°C for two hours, Sim et al. found that basalt fabric was able to maintain about 90% of its normal temperature strength [9]. However, both carbon and glass fibres were not able to maintain their volumetric integrity. Other studies showed that at temperatures of 200 – 350°C, both glass and basalt experienced mass loss. However, the basalt lost its mass at a slower rate, and retained a higher percentage of its mass at the end of the test [1,9].

Similarly, it has been shown that E-glass could be replaced with basalt fibres even in corrosive environments. Nasir et al. [11] studied the effect of submerging basalt and glass fibres in a sulfuric acid solution for different immersion times to determine the effect corrosive environments would have on the fibres. It was found that both fibres did experience significant reductions in both the strength and modulus. The degradation in the strength of the glass fibres was more severe than in basalt fibres, concluding that basalt degrades in corrosive environments at a lower rate than glass [11].

2.2 FLEXURAL STRENGTHENING USING FRP FABRICS

Use of BFRP fabric is relatively new for repair and strengthening of structural components in comparison to carbon and glass fabric fabrics. Basalt is a much newer construction material, and as such there is limited studies on its feasibility and effectiveness for the flexural strengthening of concrete beams. Hence, the current study was designed and executed.

2.2.1 FLEXURAL STRENGTHENING WITH CFRP FABRICS

The use of CFRP fabric has been well researched in repairing and strengthening of in-situ concrete members since the early 1990’s [6,12–19]. This material gained popularity due to the high strength and stiffness. Kachlakev and McCurry [20] studied application of CFRP fabric for
strengthening existing pre-cracked flexural members which are deemed inadequate for future expansion due to predicted increase in traffic load. The study concluded that this strengthening technique was an economical solution considering the alternative would be to replace the insufficient bridge girders. The study also found that this strengthening method yielded an increase in static capacity of approximately 150% over the un-strengthened control specimens. Along with this increase in strength, the cracking resistance and post cracking stiffness of the beams also increased over the control specimens due to the added flexural strengthening of the CFRP [20].

One of the main limitations to using carbon fabric is the low ductility which it provides. Carbon dry fabric only has an ultimate fabric strain of between 1.2% - 1.7% [6,18,20,21]. When compared to steel which has an ultimate strain of approximately 20%, this is a significant reduction. When combined into the composite fibre and epoxy wrap, CFRP exhibits an elongation of only 0.95% at failure [6]. This results in a reduction in the overall ductility of the flexural member. This is clearly demonstrated by Attari et al. [6], where the ductility of beams which were strengthened flexurally with only CFRP was reduced to 67% of the ductility of the control beam [6].

2.2.2 FLEXURAL STRENGTHENING WITH GFRP AND HFRP FABRICS

Due to the reduction in ductility which comes from using CFRP as a strengthening fabric, more ductile alternatives such as glass fibres have been studied previously [6,18,20,22,23]. Multiple studies have found that the ultimate strain in the glass dry fabric is between 2.0% and 2.8% for E-glass [6,20,21], and the ultimate strain of the GFRP composite wraps is 1.7% [6]. This is a significant increase over carbon of 160% and 170%, respectively. This increase in ductility has made using a hybrid of both glass and carbon FRP fabrics a viable solution to increase the deflection which the flexural fabrics can experience before a brittle rupture or delamination failure.
occurs. Attari et al. [6] tested this theory using two different types of repair. The first consisted of separate unidirectional CFRP and GFRP laminates used either in flexure, or as cross-strapping. The second used a bidirectionally woven glass and carbon blended fabric. Attari et al. found when using a combined interwoven fabric composed of carbon and glass, the ductility and strength observed increased compared to beams strengthened with only CFRP or GFRP fabrics on their own. The beam with both glass and carbon fabrics exhibited a 15% increase in ductility over beams strengthened with only CFRP, and an increase of 10% in strength over the beam with only GFRP, due to the high flexural strength of the CFRP and the elevated ultimate strain of the GFRP. The beams strengthened with only GFRP showed a similar load capacity as the beams with CFRP strengthening. However, the beams with GFRP exhibited an increased ductility of approximately 15% over the beam with only CFRP strengthening [6].

Attari et al. [6] experimented with bidirectional Hybrid Fabric Reinforced Polymers (HFRP). These fabrics consisted of glass and carbon fibres interwoven together into one sheet. Three beams were tested with this blended fabric. Attari et al. tested beams with three layers of HFRP U-wrap, two layers of HFRP U-wrap, and three layers of GFRP flexural strengthening without anchorage. The beam with no anchorage failed suddenly due to fabric delamination, presenting the requirement for proper anchorage to control debonding [6]. Three layers of U-shaped HFRP exhibited similar strength to GFRP strengthening, but with a much-reduced deflection of 65%. The two layers of HFRP held a slightly lower load and deflection than the three layer scheme [6]. Through this study, it becomes apparent that in most cases, GFRP would be preferable to its CFRP as a concrete strengthening alternative. Similar strengths can be accomplished with beams strengthened with GFRP when compared to beams strengthened with CFRP. The beams which are strengthened with only GFRP all experience much higher ductility
when compared to beams strengthened with CFRP. The beam ductility is a critical consideration when strengthening concrete flexural members. Glass fabrics exhibit a higher fracture strain than carbon fibres, allowing them to retain higher beam ductility post-strengthening.

2.3 ANCHORAGE OF FRP FIBRE STRENGTHENING SYSTEMS

Brittle failure of the composite fibre debonding from the concrete substrate needs to be prevented. Anchoring systems need to be developed to resist the peeling force exerted on the fibre, applied through the beam’s deflection. Obaidat et al. [17] found that the main failure mode for strengthened beams was plate debonding. Obaidat et al. found that this failure occurred due to high shear stresses which developed at the ends of the CFRP laminate [17]. The study suggested that either a lower stiffness or a higher fabric fracture load capacity was needed to prevent this type of brittle failure [17]. One way to increase delamination resistance in the fabric is to implement an anchoring system for the beam which resists the delamination forces exerted by the deflection. Anchorage can be applied in multiple ways with varying degrees of success. Attari et al. used continuous U-shape wrap down the entire length of the beam. This was not an optimal anchorage method however, because strain concentrations developed in the beam’s moment zone. These strain concentrations led to premature fabric rupture [6]. Dong et al. used non-continuous U-shaped anchors along the entire beam [18]. Finally, Lihua et al applied the fabric anchorage as U-shaped wrappings only at the end points of the strengthening fabric [21]. Lihua et al. found that this was the optimal method to anchor the fabrics. This resulted in a higher yield and ultimate strength from the beam when compared to wrapping the entire beam length, as it avoided strain concentrations in the moment span.
2.4 EFFECT OF REINFORCEMENT RATIO ON BEAMS STRENGTHENED WITH FRP FABRICS

Much research has been done on the behaviour of varying amounts of external GFRP and CFRP reinforcement, and its effect on flexural concrete members. Research is sparse however, when comparing this strengthening technique between varying reinforcement ratios. Some research, [13,17,18,22] studied the effect of varying the internal reinforcement ratio. Hawileh et al. [13] tested beams with 0.9% and 1.7% internal steel reinforcement. If the same number of layers of CFRP were used (in this case beams were tested with 2, 3, and 4 layers), the strength of the beams was very similar. The biggest difference between specimens was in the observed ductility. In this test, the beam with only 0.9% steel reinforcement held the same ultimate load as the beam with 1.7% steel reinforcement. However, the ductility in the beam with less rebar was 50% higher than what was found in the higher reinforcement ratios [13]. Hawileh et al. concluded that this could be attributed to the larger moment arm due to a higher neutral axis because of the lower rebar. This allowed the beam to experience a higher tension force in the flexural reinforcement provided by the CFRP before failure. However, in Dong et al. [18], the testing found that the main factor which increased the ultimate load was the longitudinal reinforcement ratio in the beam. The test did verify however that if the rebar increased while the fabric layers remained constant, the specimen still increased in stiffness [18].

A major concern when strengthening flexural concrete members with composite fabrics, is the efficiency of this strengthening method. Barros et al. [24] studied the percent increase in strength of fabric repairs when compared to the control beams, to determine the effectiveness of this method for service and ultimate conditions. Barros et al. [24] used an internal reinforcement ratio of 0.2%, 0.33%, and 0.5%. The beams were strengthened with one, two, and three layers of
CFRP respectively. It was found for the service load, a lower reinforcement ratio of steel and CFRP was the most efficient with an increase of 82% over the control. For the ultimate load, the middle reinforcement ratio and CFRP strengthening was the most efficient with a 64% increase over the control [24]. Barros et al. showed similar trends as other research. An increase in longitudinal reinforcement is the major factor towards an increase in the beam’s ultimate load capacity. This study also showed that there is a point where the repair becomes ineffective. The strength will converge to a similar ultimate point as the internal and external reinforcement is increased, but the stiffness in the beam keeps increasing leading to a major loss of ductility. The mid and high reinforcement ratio had an ultimate capacity of approximately 40% higher than the control. However, the ultimate deflection of the mid ratio beam was 60% lower than the control, while the high ratio beam only accomplished 78% of the control’s deflection [24].

2.5 STUDYING THE EFFECTIVENESS AND EFFICIENCY OF FRP FABRICS

2.5.1 INTRODUCTION OF THE PERFORMANCE FACTOR

To analyze and create an efficient repair strengthening scheme, Spadea et al. [25] suggested using a performance factor. The performance factor can be used to find the best possible balance of the strengthened beam’s ductility and strength [25]. The performance factor relates the ductility and strength of a repaired or strengthened beam directly, at two critical points. Spadea et al. [25] used the performance factor to provide a ratio between the change in strength and ductility at the service and yield points of the beam. These two ratios are multiplied together to determine the performance factor of the strengthened concrete beam as shown in Equation 2.1.
The designer can use this factor to analyze how the beam will react to different strengthening schemes, and if a higher stiffness or ductility value is critical. This factor can also be used to see the efficiency of different fibre materials when repairing or strengthening a beam. This gives the designer a more informative way to determine how to strengthen reinforced concrete members in different situations.

2.5.2 STUDY OF BEAM FRACTURE ENERGY

Another technique to determine the effectiveness of using FRP fabrics for strengthening reinforced concrete members includes analyzing the fractural toughness, or energy, of the beam. Sim et al. [9] found that the more layers the beam was strengthened with, the less deflection it would experience before it failed. With the reduction in deflection, the beam experienced a reduced amount fracture energy absorbed before it failed. Sim et al. [9] hypothesized that this was due to the fact that with the increase in strength, the bottom face of the concrete was more restrained, reducing the amount of fracture energy absorbed [9]. The fracture energy can be accurately calculated by determining the area underneath the load-deflection curve of a concrete beam. Fractural toughness is the amount of energy which is required to open a unit area of crack surface in a concrete beam. This decrease which was reported by Sim et al. [9] was due to the lower rate of increase in strength gained from the beam, compared to the rate of decrease in ductility. This area of study can present a more complete insight on the behaviour and interaction of the fabric and concrete when FRP laminates are used to strengthen reinforced concrete beams.
2.6 FURTHER RESEARCH ON BFRP FABRICS NEEDED

Much of the research which has been conducted in this field relates to the use of CFRP, GFRP, and HFRP laminates as a strengthening system for reinforced concrete elements. Throughout the literature this has shown to be a viable method for rehabilitating and strengthening concrete structures. This method does come with the major drawback of reduction in structural ductility which is observed when these materials are used. One goal of the current research is to develop an alternative which exhibits the same strengthening properties but can retain more deflection in the beam before ultimate.

Research has been extensively conducted into how FRP fabrics react with the shear beam reinforcement ratio, and shear strengthening and repair. There is much less research focusing on the interaction between flexural internal and external reinforcement interaction. This topic of research needs to be further developed and continued. There is little to no research on the topic of the interaction between the flexural steel reinforcement ratio and a higher ultimate strain composite fabric such as E-type glass or basalt fibre fabrics. These GFRP E-type and BFRP fabrics have a much higher ultimate strain than CFRP and could eliminate or at least minimize the drastic decrease in ductility which is found with CFRP rehabilitation method.

While BFRP fabrics do exhibit a lower tensile strength to CFRP and S-type GFRP, the strength is very similar to E-type GFRP which has been widely studied. This, along with the increase in durability, (both mechanically and chemically), and the lower production and material cost, makes BFRP a very enticing repair, rehabilitation, and strengthening material.
CHAPTER 3

METHODOLOGY

3.1 EXPERIMENTAL PROGRAM

3.1.1 TEST MATRIX

The test matrix was built into three smaller test matrices, which can be seen in Table 3.1, based on the specimens’ reinforcement ratio. Inside each of these three groups consists of four concrete specimens of the same reinforcement ratio. One of these beams is reserved to act as a control, to compare the strengthened beams to a reference value. After this, the beams have an increasing number of BFRP fabric layers laminated to the bottom tension face. The goal of this research was to increase the flexural strength of these beams by about 50%. The amount of BFRP layers used was based on the control beam strength and the expected strength increase from the FRP application. The reinforcement ratio was determined by using Equation (3.1). In this equation, \( \rho \) is the reinforcement ratio, \( A_s \) is the cross-sectional steel area, \( b \) is the beam width and \( d \) is the effective depth of the reinforcement. The specimen naming convention indicates both the internal reinforcement ratio and the number of BFRP layers. For example, 0.75PR-B04 stands for 0.75 percent reinforcement, with four BFRP strengthening layers.

\[
\rho = \frac{A_s}{bd} \tag{3.1}
\]
Table 3.1: Experimental Test Matrix

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Tension Rebar</th>
<th>Reinforcement Ratio (%)</th>
<th>Layers of BFRP</th>
</tr>
</thead>
<tbody>
<tr>
<td>1PR-CONTROL</td>
<td>2 x 15M</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>1PR-B04</td>
<td>2 x 15M</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>1PR-B06</td>
<td>2 x 15M</td>
<td>1</td>
<td>6</td>
</tr>
<tr>
<td>1PR-B08</td>
<td>2 x 15M</td>
<td>1</td>
<td>8</td>
</tr>
<tr>
<td>0.75PR-CONTROL</td>
<td>3 x 10M</td>
<td>0.75</td>
<td>0</td>
</tr>
<tr>
<td>0.75PR-B02</td>
<td>3 x 10M</td>
<td>0.75</td>
<td>2</td>
</tr>
<tr>
<td>0.75PR-B04</td>
<td>3 x 10M</td>
<td>0.75</td>
<td>4</td>
</tr>
<tr>
<td>0.75PR-B08</td>
<td>3 x 10M</td>
<td>0.75</td>
<td>8</td>
</tr>
<tr>
<td>0.5PR-CONTROL</td>
<td>2 x 10M</td>
<td>0.5</td>
<td>0</td>
</tr>
<tr>
<td>0.5PR-B02</td>
<td>2 x 10M</td>
<td>0.5</td>
<td>2</td>
</tr>
<tr>
<td>0.5PR-B04</td>
<td>2 x 10M</td>
<td>0.5</td>
<td>4</td>
</tr>
<tr>
<td>0.5PR-B08</td>
<td>2 x 10M</td>
<td>0.5</td>
<td>8</td>
</tr>
</tbody>
</table>

The goal for this research is to study the viability of using BFRP fabrics as a strengthening material, instead of GFRP and CFRP. The target was to see a significant increase in beam flexural capacity, while still maintaining as much of the beam’s deflection as possible prior to beam failure. The target strength increase was an approximately 50% increase in strength over the control beam for each ratio. This increase was estimated using CSA S806-12. After confirmation with CSA S806-12, four layers of BFRP was decided on for the starting point for the testing. The layers were then varied to study the effect which this would have on the beams’ strength and deflection and to find the optimal amount of strengthening reinforcement. Attari et al. [6] experienced reductions in
beam deflections of 67% when compared to the control beam. In the current study, the deflection goal was set for the strengthened beams to experience only a deflection reduction of 30% or less when compared to the control beam.

The increase observed in the number of layers between the lowest reinforcement ratio and the highest is due to the increase in the base flexural strengthening as the $A_s$ value is increased. The number of layers and therefore tensile strength of the BFRP had to be increased to get a noticeable increase in the capacity. This does not affect the ability to compare results between ratios, as the results are presented in terms of internal beam moment ($M_r$) and stress ($\sigma$).

### 3.1.2 THEORETICAL DESIGN CALCULATIONS

All specimen were designed to be under reinforced flexural beams and hence, these beams were expected to fail in the moment span. This research is focused on the affect which external fabric laminations with BFRP has flexurally on reinforced concrete beams, so the beams were designed to be flexurally dominated. All concrete beams were designed using the Canadian Design of Concrete Structures standard CSA A23.3-14 [26].

The total shear capacity, $V_r$, is a function of the concrete shear resistance $V_c$ and steel shear resistance $V_s$, and is defined as:

$$V_r = V_c + V_s$$

(3.2)

The concrete shear resistance ($V_c$) represents a function of the concrete specific material and beam properties. These include $\alpha_c$ for the concrete resistance factor, $\lambda$ represents the concrete density factor, $\beta$ accounts for the shear resistance of cracked concrete, $f'c$ is the concrete compressive strength, $b_w$ is the beam web dimension and $d_v$ is the effective shear depth.
The steel shear resistance is a function of the resistance added by the vertical steel stirrups. This factor is represented through $\phi_s$ for the steel resistance factor, $A_v$ represents the steel shear reinforcement, $f_y$ is the steel yield strength, $\theta$ is the angle of the compressive stress inclination, and $S$ is the spacing in millimeters of the steel stirrups.

$$V_s = \frac{\phi_s A_v f_y d_v \cot \theta}{s}$$  \hfill (3.4)

These shear calculations are compared against the flexural capacity of the beam shown in Equation (3.5), to ensure that the flexural capacity is lower than the shear capacity. This ensures a flexural failure in the beam. The flexural capacity ($M_r$) is calculated as a moment arm due to the tensile force in the reinforcement (assuming steel yielding failure) around the center of the concrete compressive block. In Equation (3.5), $\alpha_1$ represents the average concrete stress in the idealized rectangular block.

$$M_r = \left[ \left( 1 - \frac{\rho \phi_s f_y}{2 \alpha_1 f_c'} \right) \rho \phi_s f_y \right] bd^2$$  \hfill (3.5)

### 3.2 SPECIMEN CONSTRUCTION

#### 3.2.1 FORMWORK

When testing concrete specimens, it is inevitable that there will be variation between individual specimen due to the nonhomogeneous nature of the material. To limit this, the entire test matrix was cast from the same batch. The concrete was mixed professionally by a concrete supplier and delivered in a standard concrete mix truck. The formwork was constructed in such a way that all the beams were cast together at the same time. This was accomplished by creating 5
sets of forms, which were able to cast five beams each. The layout for these forms can be seen in Figure 3.1. The beams were designed to have a length of 2.4 m, with a clear span of 1.9 m. The height of the beams is 250 mm and the width is 200 mm. The dimensions were designed to produce a slender beam, with a length to depth ratio of 9.6. This allows the beam to be considered a pure bending beam, with plane sections remaining plane even after deformation, and analyzed as a Bernoulli beam.

![Figure 3.1: Rebar Cages Inserted into Concrete Formwork](image)

**3.2.2 REBAR CAGES**

Three sets of rebar cages were built based on the flexural reinforcement ratio. These were designed to have very high shear capacity and be under reinforced so that the failure mode controlling all specimen was ductile. The cages were designed to the dimensions show in Figure 3.2. A concrete cover of 35 mm was designed to surround the rebar, based on the exposure conditions provided in CSA A23.3-14. The amount of shear stirrups number 15 10M bars, more than enough to increase the shear strength of the beam and make it flexurally dependent. Once the
rebar cages had been fastened together using standard rebar cage ties, the cages are placed into the oiled formwork, as seen in Figure 3.1.

![Rebar Cage Dimensions](image)

Figure 3.2: Rebar Cage Dimensions

### 3.2.3 BEAM CONSTRUCTION

The concrete mix which was used for the beams was purchased from a professional concrete supply company. The requested strength and composition was 30 MPa compressive strength, 100mm slump which was obtained, and 19mm maximum aggregate size. The mix was requested to contain no additives such as plasticizers, fly ash, slag or silica fume. The concrete was poured inside the University of Windsor’s structural engineering lab. As the forms were filled the concrete was vibrated sufficiently. The top surface of each beam was finished by hand to a smooth trowel finish, observed in Figure 3.3. This was to ensure a smooth finish for the beam so that the testing would have a smooth, square finish to apply the load to as symmetrically as possible.
Figure 3.3: Concrete Casting

As the concrete was cast into the forms in Figure 3.3, concrete compressive cylinders were cast as well. Cylinders were taken throughout the concrete cast, to provide a representative result for the concrete 28-day strength. While the concrete was cast, a slump test was also taken to measure the field slump, which is shown in Figure 3.4. After the concrete cylinders had reached their 28-day strength for compression, they were tested according to ASTM Standard C39/C39M [27]. A sample of the specimen after the compressive test was completed can be seen in Figure 3.5. All the information for the actual concrete compressive strength values can be seen in Table 3.2.

Figure 3.4: Concrete Slump Test
Figure 3.5: Compressive Cylinder Test
Table 3.2: Concrete Strength Properties

<table>
<thead>
<tr>
<th>Value</th>
<th>Compressive Strength (MPa)</th>
<th>Coefficient of Variation (%)</th>
<th>Number of Samples</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>32.5</td>
<td>7.3</td>
<td>7</td>
</tr>
</tbody>
</table>

3.2.4 BFRP APPLICATION

When applying the BFRP fabric to the tension face of the concrete, a great emphasis was placed on the preparation and application of BFRP to the specimen. This is because all the tensile forces are going to be transferred to the concrete, in the same manner as internally cast steel rebar, through the in-plane shear stresses. Before the BFRP fabric was applied, the beams need to be clean of any dust and debris. Then the beams were primed with MasterBrace P 3500 primer, to ensure the epoxy resin adhered correctly to the concrete surface. This is illustrated in Figure 3.6, and Figure 3.7 shows the beam in the correct orientation and with the BFRP applied to the tension face. The epoxy used in this research was a two-part epoxy; MasterBrace SAT 4500. This two-part epoxy was mixed and then impregnated into the fabric as it was applied to the concrete beam.

![Figure 3.6: Concrete Beams with Epoxy Primer](image1)

![Figure 3.7: Concrete Beams with BFRP Applied](image2)

![Figure 3.8: Layout of BFRP Application](image3)
The BFRP was applied in layers with cross-strapping running orthogonal to the direction of flexural strengthening at the ends of the span. This anchorage was to ensure that the fabric did not de-bond prematurely. The delamination failure would not have resulted in a safe failure mode, as this can happen rapidly and is dangerous. When repairing or strengthening shear deficient beams the most common practice is to wrap and constrain the shear spans [5,20]. This provides a confinement force to the concrete similar to strengthening columns by confining them with collars. Research has been conducted with flexural reinforcement, where the laminate is anchored through the use of continuous U-wraps [17,18,20,21,25]. This can affect the shear capacity of the beam as it can increase the confinement in the shear zone inadvertently. To avoid this, cross strapping can be in the form of U-Wraps which run along the entire span of the beam, but only partially up the vertical face, as recommended and used by Duic et al [28], so as not to interfere with the beam’s un-strengthened shear capacity. Literature and past tests have used cross strapping that ran the length of the span. It was suggested by Lihua et al. [21] that if the spacing is too close or continuous in the moment zone, a high strain concentration will build up at the interface and cause premature rupture or debonding of the anchorage [21]. It was decided to maximize the strength increase, by only using anchorage at the endpoints. Designing the anchorage in this fashion allowed for the specimen to be strengthened purely in their flexural capacity. The cross-strapping layout can be seen in Figure 3.8. Cross strapping was applied on each side of the span, every two layers of tensile reinforcement; i.e. for each 4 layers of laminate there is 2 layers of cross-strapping on either side. All fabric had to be applied at the same time within a 30-minute period. The BFRP was applied to the beam using a dry lay-up method, meaning that the fabric was not pre-impregnated with an epoxy.
3.3 EXPERIMENTAL TEST SET-UP

3.3.1 SPECIMEN SET-UP

The tests were conducted using a four-point bending load to accomplish a constant moment zone in the mid-span. This is illustrated in Figure 3.9, where the comprehensive test setup is observed. The load was applied from a stiff loading frame, through a computer-controlled load-displacement actuator, showing in Figure 3.9. The load was applied monotonically through the actuator to the specimen using a displacement-controlled loading method and was applied at a rate of 2 mm/min. The sampling rate chosen for data collection was one second intervals, ensuring sufficient data was collected during testing. This rate of application and sampling was chosen to allow for ample data both within the beam elastic and plastic ranges. The load is then applied to a stiffened loading beam, which applies the load down into the pin and roller supports on the top of the beam. This ensures that there is a point load, creating a constant moment zone of 800 mm wide.

At the ends of the beams, is a load cell on either end to verify the load coming from the actuator controller. This is also used to ensure that the load is being applied symmetrically along the length of the beam. At the ends of the beams are pin and roller supports, to ensure a simply supported beam. Underneath the beam at the center and quarter points of the beam span are three Linear Variable Differential Transformers (LVDT), which capture the profile of the beam as it deflects. This was used for the analysis which will be presented further, of the deflection, curvature and deformability of the beam. For the specimen strengthened with BFRP, it was ensured that the BFRP cross-strapping did not overlap with the support plates to avoid stress concentrations and premature rupture at the support.
3.3.2 DATA ACQUISITION SYSTEM

A Data Acquisition System (DAQ) was used throughout all the tests to capture important data during the experiments. This included the LVDT, loadcell, and strain data from the beams. As illustrated in Figure 3.10, strain gauges were installed on the rebar cage prior to the cage being cast into the concrete. There was a total of eight to ten strain gauges installed on the rebar, with redundant gauges in the case that some may be damaged during casting, prepping or testing the specimen. Some beams have ten strain gauges is due to the 0.75PR beams having an extra middle flexural bar when compared to the other ratios. The rebar was flattened slightly and smoothed with a small grinder where the strain gauges were applied to ensure that they have a good adhesion to
the metal. The gauges were then soldered to wires which were run down the length of the cage and out the ends of the forms. These wires were attached to the cage using zip ties to prevent any damage. The strain gauges were also protected using electrical tape, along with the wire to gauge connections, to keep them safe. Examples of this can be observed in Figure 3.11, Figure 3.12, and Figure 3.13.
3.3.3 DIGITAL IMAGING CORRELATION

Another tool which was used to capture and analyze data for this research project was the use of Digital Image Correlation (DIC). DIC has proven to be an accurate method to capturing strain profiles, among other data, for both brittle and ductile material [29,30]. It is especially effective for brittle materials such as concrete and masonry, because it can catch strain concentrations and cracks which strain gauges could miss due to the material’s brittle nature. It works by taking pictures over a time interval during the test, (in this case every five seconds), and then comparing the pictures to previous ones while tracking the pixels. This is accomplished through analytical software, such as GOM Correlate [5]. The pictures are loaded into the software and pixels are analyzed to measure the distance which they travel in relation to each other. If they travel closer together it creates a compression value, and away from each other creates a tension value. The pixels which it tracks are arranged into blocks based on a size and number of pixels which the researcher decides on. The smaller the pixel block size, the finer the 2-D strain profile becomes. This essentially allows the researcher to analyze the beam as if there are hundreds or even thousands of strain gauges active on the concrete.

![DIC Images]

Figure 3.14: DIC at Center: a) Before Loading b) At Yield c) At Ultimate
In Figure 3.14 a), b), and c) is an example for one of the beams’ mid span. The pictures show the general strain contour which is provided when GOM Correlate analyzes the DIC pictures during the tests. These can be used to analyze how the overall beam is behaving during the test, and at localized points, depending on what data is being studied during the test. All this analysis work happens post-testing.
CHAPTER 4
RESULTS AND DISCUSSION

4.1 STRENGTHENED BEAM PERFORMANCE

4.1.1 LOAD-DEFLECTION DATA

The load-deformation behaviour of all the beams in this study are shown in Figure 4.1, Figure 4.2, and Figure 4.3. These figures show global strength and deflection of all specimens and are grouped according to their internal reinforcement ratios. This helps to illustrate the changes each set of beams undergo as the amount of BFRP layers increase. The increase in stiffness becomes apparent through these figures, as the slope of each beam’s load-deflection curve increases with the increase in BFRP layers. This increase in BFRP provides an increase in yield and ultimate capacity of the beam when compared to the control beams. The deflection of the beam at yield load and ultimate load increases, as opposed to the findings in past literature. An increase in deflection of up to 26% at the yield points and 142% at the ultimate points of these beams were observed in these specimens, as can be found in Table 4.2. As more BFRP fabric is added, the flexural strength of the beams increases until the failure mode changes. Over reinforcement can be seen in 0.5PR-B08 and 0.75PR-B08 specimens from Figure 4.1 and Figure 4.2. The amount of reinforcement changed the failure mode from rupture in the BFRP to shear-tension failure.

Equation (4.1) depicts how the failure mode changes based on the ratio between compressive resistance and tensile resistance of the beam section (CSA S806-12). Equation (4.1) shows the internal compression resistance (left hand side of equation) which needs to be higher than the tension resistance (right hand side of the equation), for the beam to fail due to FRP tensile rupture. If the amount of tensile steel and/or FRP fabric is increased, however, the failure mode
changes to concrete crushing. In this equation, $\phi$ represents the resistance factor for each material, $f'_c$ is the compressive strength of the concrete, $A'_s$ and $A_s$ is the cross-sectional area for the compressive and tensile steel respectively, $f_y$ is the yield strength of the steel, $A_{FRP}$ is the cross-sectional FRP area, $\varepsilon_{FRP}$ is the strain in the FRP, $\beta_1$ is the ratio of the depth of the neutral axis to the compression block, and $E_{FRP}$ is the modulus of elasticity for the FRP.

$$\alpha_1 \phi f'_c c b \beta_1 c + \phi s f y A'_s > \phi s f y A_s + \phi_{FRP} E_{FRP} \varepsilon_{FRP} A_{FRP}$$

All beams strengthened with BFRP fabrics exhibited increase in strength as compared to the control beam (un-strengthened beam). However, percent increase in beams with the lowest amount of steel (for example, 0.5PR) was much higher than the beams with a higher amount of steel (0.75PR). It does not translate to as great an increase in global strength due to the control beam’s higher internal reinforcement. The behaviour of the beam also changes before and after the rupture of the fabric, illustrated in Figures 4.2 and 4.3. After the failure of the BFRP occurred, some of the beams continued to be loaded to determine whether the beams could maintain the same strength exhibited in the control. The strengthened beams were not able to recover to the same strength of the control beams. This is due to the increase in beam and rebar damage before the BFRP failed. With the strengthened beams able to reach higher $M_r$ capacities, the tensile and compression rebar experienced higher strains before failure when compared with the control beam. This increase in strain contributes to the reduction in the post-failure strength of the beam when compared to the unstrengthened control. The control specimens’ rebar experienced less severe damage and was able to resist a higher moment than the strengthened beams at post-ultimate strength. This behaviour is covered more extensively in Section 4.2.2 on the strains found in the
internal rebar and illustrated again in Section 4.3.2 with the crack patterns. This reduction in post-failure beam strength is exhibited in 0.75PR-B02, 0.75PR-B04, 1PR-B04, and 1PR-B06.

Figures 4.1 – 4.3 display the load-deformation behaviour of the twelve specimens. The specimens with the failure mode of concrete crushing is represented by C, basalt rupture is R, and shear-tension failure is S.
Figure 4.2: 0.75PR Load-Deflection Curves

Figure 4.3: 1PR Load-Deflection Curves
Table 4.1 represents the numerical summary of what is presented in Figures 4.1 – 4.3. The load and mid-span deflection are taken at the same service, yield, and ultimate load states. The service moment was taken as 60% of the yield load, as per clause 10.6.1 in A23.3-14. The yield load was taken as the load resisted by the concrete beams at the onset of inelastic behaviour. The equation to calculate the service load, shown in Equation (4.2), determines the highest acceptable service load based on the crack width and exposure conditions. In Equation (4.2), $z$ is the allowable force for both interior or exterior exposure, $d_c$ is the depth of the concrete cover, $f_s$ is the stress in the reinforcement at the service load, and $A$ is the twice the area from the centroid of the internal reinforcement to the bottom face of the beam.

$$z = f_s^3 \sqrt[3]{d_c A}$$  \hspace{1cm} (4.2)
Table 4.1: Moment and Deflection of Strengthened Beams at Service, Yield, and Ultimate State

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Moment (kN·m)</th>
<th></th>
<th></th>
<th>Displacement (mm)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Service</td>
<td>Yield</td>
<td>Ultimate</td>
<td>Service</td>
<td>Yield</td>
<td>Ultimate</td>
</tr>
<tr>
<td>1PR-Control</td>
<td>18.1</td>
<td>30.3</td>
<td>30.6</td>
<td>7.1</td>
<td>12.1</td>
<td>19.0</td>
</tr>
<tr>
<td>1PR-B04</td>
<td>19.8</td>
<td>34.1</td>
<td>43.9</td>
<td>6.7</td>
<td>12.9</td>
<td>46.1</td>
</tr>
<tr>
<td>1PR-B06</td>
<td>23.0</td>
<td>36.7</td>
<td>53.5</td>
<td>3.7</td>
<td>12.4</td>
<td>34.0</td>
</tr>
<tr>
<td>1PR-B08</td>
<td>26.0</td>
<td>44.2</td>
<td>56.3</td>
<td>6.2</td>
<td>11.4</td>
<td>30.9</td>
</tr>
<tr>
<td>0.75PR-Control</td>
<td>15.6</td>
<td>26.8</td>
<td>30.8</td>
<td>6.0</td>
<td>12.0</td>
<td>31.2</td>
</tr>
<tr>
<td>0.75PR-B02</td>
<td>16.2</td>
<td>27.4</td>
<td>40.1</td>
<td>4.9</td>
<td>9.3</td>
<td>41.3</td>
</tr>
<tr>
<td>0.75PR-B04</td>
<td>17.1</td>
<td>29.3</td>
<td>41.1</td>
<td>5.2</td>
<td>10.6</td>
<td>48.2</td>
</tr>
<tr>
<td>0.75PR-B08</td>
<td>19.8</td>
<td>36.7</td>
<td>59.0</td>
<td>4.3</td>
<td>9.6</td>
<td>29.1</td>
</tr>
<tr>
<td>0.5PR-Control</td>
<td>10.2</td>
<td>16.3</td>
<td>22.6</td>
<td>2.8</td>
<td>6.8</td>
<td>31.3</td>
</tr>
<tr>
<td>0.5PR-B02</td>
<td>12.5</td>
<td>20.8</td>
<td>33.6</td>
<td>3.4</td>
<td>8.5</td>
<td>40.5</td>
</tr>
<tr>
<td>0.5PR-B04</td>
<td>13.6</td>
<td>23.0</td>
<td>44.2</td>
<td>3.3</td>
<td>8.0</td>
<td>52.9</td>
</tr>
<tr>
<td>0.5PR-B08</td>
<td>16.7</td>
<td>29.1</td>
<td>49.7</td>
<td>4.3</td>
<td>8.5</td>
<td>26.1</td>
</tr>
</tbody>
</table>
Table 4.2 presents the percent change in moment and deflection the beams experience as the BFRP strengthening is increased.

**Table 4.2: Percent Change in Moment and Deflection of Strengthened Specimens relative to Control Specimens**

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Δ Moment (%)</th>
<th>Δ Deflection (%)</th>
<th>Service</th>
<th>Yield</th>
<th>Ultimate</th>
<th>Service</th>
<th>Yield</th>
<th>Ultimate</th>
</tr>
</thead>
<tbody>
<tr>
<td>1PR-B04</td>
<td>12.6</td>
<td>43.7</td>
<td>7.1</td>
<td>142.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1PR-B06</td>
<td>21.1</td>
<td>75.1</td>
<td>2.2</td>
<td>78.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1PR-B08</td>
<td>45.9</td>
<td>84.2</td>
<td>-5.4</td>
<td>62.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.75PR-B02</td>
<td>2.4</td>
<td>30.2</td>
<td>-22.9</td>
<td>32.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.75PR-B04</td>
<td>9.4</td>
<td>33.5</td>
<td>-12.0</td>
<td>54.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.75PR-B08</td>
<td>37.1</td>
<td>91.8</td>
<td>-20.5</td>
<td>-6.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5PR-B02</td>
<td>27.6</td>
<td>48.7</td>
<td>25.9</td>
<td>29.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5PR-B04</td>
<td>41.5</td>
<td>95.2</td>
<td>18.8</td>
<td>68.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5PR-B08</td>
<td>79.1</td>
<td>119.7</td>
<td>21.5</td>
<td>-16.8</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Percent change is same for moment and deflection at both service and yield point, because beam is still elastic*

**4.1.2 SPECIMEN MOMENT-CURVATURE**

Calculating the moment-curvature is an effective technique to display the behaviour of flexural beams and was analyzed for all beams in the test matrix. Through the elastic-beam theorem, it can be assumed that Equation (4.3) describes the curvature of a beam element when it experiences pure bending. In this equation, \( \rho \) is the radius of curvature, \( M \) is the internal moment, \( E \) is the modulus of elasticity and \( I \) is the beam’s moment of inertia.

\[
\frac{1}{\rho} = \frac{M}{EI}
\]  

(4.3)
If the beam only displaces vertically and not laterally, this relationship can be expressed as a second order differential equation. Based on the assumption that the beam only displaces vertically, the curvature can be found at any point. The second order equation is shown in its simplified form in Equation (4.4), where $y$ is the vertical displacement.

$$\frac{d^2 y}{dx^2} = \frac{M}{EI} \quad (4.4)$$

Using the LVDTs that were placed under the beam, the beam’s profile can be expressed on an $x$ and $y$ coordinate system for any displacement at a corresponding load, where $x$ is the position of a point along the beam and $y$ is the displacement. There are five known $x$-$y$ data points; the two supports, and the three LVDT. With this data and the relationship between vertical displacement and curvature in Equation (4.4), the curvature of all the beams can be calculated. With five data points known, a fourth order polynomial was fit to the profile curve, and the polynomial equation ($f(x)$) was calculated as a function of its $x$ and $y$ coordinates. This curve was then differentiated twice using Equation (4.5) to find the curvature for every load point, where $\psi$ represents the beam’s curvature.

$$\psi = \int \int f(x) = \int \left( \frac{M}{EI} \right) dx \quad (4.5)$$

The moment-curvature graphs can be found in Figure 4.4, Figure 4.5, and Figure 4.6. The graphs are grouped and compared for beams with the same reinforcement ratios. These graphs illustrate the increase in moment-curvature stiffness as the layers of BFRP fabric increase. It can be seen in Figure 4.4, Figure 4.5, and Figure 4.6, similarly to the load-deflection curves, the beams failed in a ductile manner. There was still a significant curvature at ultimate load, and usually a reduction in load before tensile rupture. This is important when designing concrete flexural
members, as the beam needs to fail in a ductile a manner. Apart from Specimens 0.5PR-B08 and 0.75PR-B08, all other beams still exhibit a ductile failure. When looking at 0.5PR-B08 and 0.75PR-B08, these beams were over-reinforced with BFRP fabrics, which caused them to be pushed into a more dangerous brittle combination of shear-tension failure. This failure is due to the insufficient lap length in the cross-strapping at this force, discussed further in Section 4.1.3. This change in failure mode can provide a ceiling for the maximum amount of reinforcement that can be applied before the failure mode becomes dangerous. Even so, all the beams still experienced tensile steel yield before failure. Both sets of graphs show that the best performance boost comes from the mid-set of fabric reinforcement; 0.5PR-B04, 0.75PR-B04, and 1PR-B04.

![0.5PR Moment-Curvature Curves](image)

*Figure 4.4: 0.5PR Moment-Curvature Curves*
4.1.3 BEAM FAILURE MODE

According to the CSA codes S6-14 [31] and S806-12 [32] regarding the design and application of FRP to structural members, the failure mode of the beam falls into two categories:
concrete crushing failure and failure in the FRP. Concrete crushing failure can happen either before or after the tensile steel yields, but always before the FRP ruptures. Tensile failure of these structures through the rupture in the FRP always occurring after tensile steel yielding. Due to steel having a much higher ultimate strain capacity than FRP materials, the yielding in the tensile steel is generally not considered a failure mode. Steel yielding, however, is always desired in order to add ductility to the beams failure [4].

Table 4.3 shows the failure modes throughout the testing for all the specimens in the test matrix. All specimens, apart from 0.75PR-B08 and 0.5PR-B08, experienced an optimal failure path, which is steel yielding, followed by concrete crushing in the compressive zone and then the tensile BFRP rupture as the ultimate failure mode. This is the optimal failure path as it allows all three failure modes to occur, confirmed through Figure 4.1 to Figure 4.6, and in Table 4.2. The only beams which did not follow the optimal failure path was Specimens 0.75PR-B08 and 0.5PR-B08. These beams experienced shear-tension failure as the ultimate failure mode, which is very dangerous due to its brittle nature.

This shear-tension failure is a form of BFRP debonding where the concrete substrate does not debond from the BFRP. Shear-tension failure in externally strengthened beams occurs when the BFRP causes the concrete cover to debond from the internal rebar due to the peeling force experienced by the BFRP. This phenomenon can lead to early debonding and failure in the shear and flexural zones, causing early failure. This failure is what occurred in Specimens 0.5PR-B08 and 0.75PR-B08, and which can be observed in Figure 4.7.

Due to the strengthening scheme of Specimens 0.5PR-B08 and 0.75PR-B08, these specimens experienced an elevated load when compared with other specimen in this study. As the layers of external reinforcement are increased, this causes higher concentrations of forces at the
end of the flexural and cross-strapping reinforcement. These forces exceed the rupture strength of
the concrete cover, and cause it to delaminate suddenly, reducing the effective section. This is what
caused Specimens 0.5PR-B08 and 0.75PR-B08 to fail in shear-tension before the specimens
reached the shear capacity. The propagation of cracks moving from the flexural span and into the
shear span also encourages this process, as the cracks can damage the cross-strapping as they are
formed. As more external BFRP reinforcement is added, the cracks propagate more densely from
the flexural zone and into the shear span, which is discussed in more detail in Section 4.3.2.

![Figure 4.7: Shear-Tension Failure Mode in Over Strengthened Beams](image)

The increase in stiffness and load capacity, as seen in the load-deflection behavior due to
reinforcing these beams with eight layers of BFRP, is what caused these beams to fail in a brittle
manner. Even with this sub-optimal failure path, the tensile steel still yielded prior to shear-tension failure, retaining some ductility to the failure.

**Table 4.3: Specimen Failure Modes by Stage Through Testing**

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Failure Modes Through Test</th>
<th>First Failure</th>
<th>Second Failure</th>
<th>Third Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1PR-Control</td>
<td>Concrete Compression Crushing</td>
<td>-</td>
<td>BFRP Rupture</td>
<td></td>
</tr>
<tr>
<td>1PR-B04</td>
<td>Concrete Compression Crushing</td>
<td>Concrete Compression Crushing</td>
<td>BFRP Rupture</td>
<td></td>
</tr>
<tr>
<td>1PR-B06</td>
<td>Concrete Compression Crushing</td>
<td>Concrete Compression Crushing</td>
<td>BFRP Rupture</td>
<td></td>
</tr>
<tr>
<td>1PR-B08</td>
<td>Concrete Compression Crushing</td>
<td>Concrete Compression Crushing</td>
<td>BFRP Rupture</td>
<td></td>
</tr>
<tr>
<td>0.75PR-Control</td>
<td>Concrete Compression Crushing</td>
<td>-</td>
<td>BFRP Rupture</td>
<td></td>
</tr>
<tr>
<td>0.75PR-B02</td>
<td>Concrete Compression Crushing</td>
<td>Concrete Compression Crushing</td>
<td>BFRP Rupture</td>
<td></td>
</tr>
<tr>
<td>0.75PR-B04</td>
<td>Concrete Compression Crushing</td>
<td>Concrete Compression Crushing</td>
<td>BFRP Rupture</td>
<td></td>
</tr>
<tr>
<td>0.75PR-B08</td>
<td>Concrete Shear Crushing</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5PR-Control</td>
<td>Concrete Compression Crushing</td>
<td>-</td>
<td>BFRP Rupture</td>
<td></td>
</tr>
<tr>
<td>0.5PR-B02</td>
<td>Concrete Compression Crushing</td>
<td>Concrete Compression Crushing</td>
<td>BFRP Rupture</td>
<td></td>
</tr>
<tr>
<td>0.5PR-B04</td>
<td>Concrete Compression Crushing</td>
<td>Concrete Compression Crushing</td>
<td>BFRP Rupture</td>
<td></td>
</tr>
<tr>
<td>0.5PR-B08</td>
<td>Concrete Shear Crushing</td>
<td>-</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The best failure path is the one which allows all three stages of failure to develop. The best failure result was obtained from the beams which were strengthened with four layers of BFRP. While the load resisted by the beams increased, concrete crushing and BFRP rupture gradually followed. This gradual failure of both the concrete and the basalt is a ductile failure. Conversely, for the beams in Figure 4.1 – 4.3, which show a sudden drop in load, have failed in a brittle manner. This is due to greatly increased stresses in both the BFRP and concrete, which fail very quickly.
releasing a large amount of energy. It is this sudden and violent failure which needs to be avoided. Using BFRP as a strengthening material has shown to exhibit ductile failure when strengthening these concrete beams, if the beams are not over-strengthened.

**4.1.4 LOAD AND DEFLECTION OF STRENGTHENED SPECIMEN**

The load-deflection behaviour of a beam is an effective way to determine the strength and ductility change from one beam to another. Table 4.1 shows the recorded results for all beams. As expected, as more BFRP strengthening reinforcement is added the moment resistance of the beam increases. It was found in literature that the increase in strength provided by composite materials such as CFRP and GFRP, comes with a decrease in ultimate load capacity ductility [6]. That was generally not found to be the case with these beams. Flexural specimen in this research experienced increased deflections at yield and ultimate load, of up to 26% and 142%, respectively, relative to the control specimens (Table 4.2).

The global stiffness in terms of load-deflection behavior of the beam also increases from the application of the BFRP. When fabric is applied to the tension surface of a beam, the beam transfers the stress it experiences through in-plane shear stresses, similar to internal rebar. However, when significant damage has occurred in the concrete substrate (such as extensive cracking), the interaction changes. After significant damage has occurred, it is considered that the BFRP is no longer attached to the concrete substrate at the midspan and is only attached to the beam at the ends. As the beam deflects further, the BFRP experiences increased tensile forces due to the vertical displacement of the beam, exerting a strain on the BFRP as the fabric expands to match the beam deflection. As the beam deflects, the BFRP resists the deformation of the concrete substrate, effectively causing the bottom tension face to be confined by the BFRP. This
is what is causing the increase in the beam’s elastic stiffness with regards to load-deflection behaviour, and the increase in strength in the inelastic behaviour.

As expected, increasing the number of BFRP fabric layers increases the yield and ultimate moment capacity. However, it does not decrease the resulting deflection at these points. This study shows that there will be an increase in beam stiffness, and the percentage increase in the stiffness will depend on how many layers of BFRP fabric are added. Since the BFRP material can sustain a higher strain (up to 2.35%) than commonly used FRP materials (such as carbon or glass), it does not reduce the beam’s ductility below that of the control specimen, as opposed to carbon and glass FRP materials, unless over-strengthened.

Table 4.2 presents the percent increase in strength and midspan deflection. It can be observed that the efficacy with which the BFRP works varies with different levels of internal reinforcement. The percent change in moment capacity is higher for the beam with the lowest reinforcement ratio than the beam with the highest ratio.

When comparing the 1PR series and 0.5PR series beams, there is a higher percent increase in moment capacity at all load levels in the 0.5PR beams than there is in the 1PR beams when the number of BFRP layers are not changed. This could be attributed to the effect the internal steel reinforcements have on the position of the neutral axis, which can be seen in Equations (4.6) and (4.7) Based on the amount of steel present, the neutral axis will move vertically in the cross section. Decreasing the amount of steel in the tension zone causes the neutral axis to move upwards. This increases the moment arm between compressive resistance and tensile resistance offered by BFRP, which is illustrated in Equation (4.8). The higher the neutral axis, the more stress that will develop in the BFRP, increasing the composite’s efficacy at strengthening the member. This was presented as one of the findings in the literature [13], however, the variation in reinforcement ratio in this
presented research does not affect the neutral axis significantly. The increase in overall deflection ductility was the main factor which effected the efficiency of this strengthening technique. The 0.5PR series beams were more significantly affected when the BFRP was applied, due to providing the largest control beam deflection. These beams had the most ductility before strengthening and so were able to retain the most deflection after strengthening.

Equation (4.6) is used if concrete crushing is the beam failure mode and uses the internal moment equilibrium to calculate the neutral axis. In this formula, $A'_s$ represents the compressive rebar area, $\phi_{FRP}$ is the BFRP resistance factor, $E_{FRP}$ is the modulus of elasticity of the BFRP, $\varepsilon_{cu}$ is the concrete’s ultimate strain, $\varepsilon_{fi}$ is the initial strain in the beam before strengthening, $c$ is the neutral axis depth, and $A_{FRP}$ is the cross-sectional area of the BFRP.

$$
\alpha_1 \phi_{c} f' c \beta_1 b c^2 + \left[ \phi_s f_y (A'_s - A_s) + \phi_{FRP} E_{FRP} (\varepsilon_{cu} + \varepsilon_{fi}) A_{FRP} \right] c
- \phi_{FRP} E_{FRP} \varepsilon_{cu} A_{FRP} h = 0
$$

(4.6)

If the failure mode for the specimen is the tensile rupture of the BFRP, then Equation (4.7) is used to determine the depth of the centroid of the concrete compressive block.

$$
c = \frac{\phi_s f_y (A_s - A'_s) + \phi_{FRP} E_{FRP} \varepsilon_{FRP} A_{FRP}}{\alpha_1 \phi_{c} f' c \beta_1 b}
$$

(4.7)

Once the depth of the concrete compressive block is determined, the flexural moment resistance, $M_r$, can be determined from Equation (4.8). This is accomplished by calculating the moment of the forces around the concrete compressive block, in the steel and BFRP. $C_s$ is the force in the compression steel, $T_s$ is the force in the tensile steel, $T_{FRP}$ is the force in the BFRP, $a$ is the
equivalent concrete stress block, \( d' \) is the depth of the compressive rebar, and \( h \) is the height of the member.

\[
M_r = C_s \left( \frac{a}{2} - d' \right) + T_s \left( d - \frac{a}{2} \right) + T_{FRP} \left( h - \frac{a}{2} \right)
\]  

(4.8)

Overall, the increase in strength is significant if the amount of BFRP fabric is proportionate to the amount of steel reinforcement that is present. The deflection retention values from Table 4.2 of the strengthened beams are also more efficient at lower levels of steel reinforcement due to the increase in deflection a beam undergoes, as its tension reinforcement is decreased. This is also applicable when strengthening the beam with BFRP layers. As more tensile steel and BFRP reinforcement is added to the beam, the load-deflection behaviour of the beam becomes stiffer, and less efficient.

4.1.5 PERFORMANCE OF STRENGTHENED BEAMS

To analyze these beams in a way which can be applied and compared to beams with different cross-sections and reinforcement ratios, the concept of a performance factor was applied. Spada et al. [25] first suggested this method when studying the overall effectiveness of using FRP fabrics to strengthen reinforced concrete beams. The concept is to compare the increase on initial strength at a known critical point to the maximum strength at a second known critical point and comparing the change in the deflection at these two points as shown in Equation (4.9). In this equation, the first ratio (term) is the deflection factor and the second ratio (term) is the strength factor. This equation interprets the beam strengthening process into two key variables for the designer: the strength factor and the deflection factor, allowing the designer to place emphasis on one of these variables depending on the design needs. Depending on which is more critical, the
strengthening scheme can be adjusted accordingly [25]. For the presented research, the two critical points which were considered to study the strength and deflections factors, were the service and ultimate loads of these beams. The service load was considered as 60% of the yield load (CSA A23.3-14). Equation (4.9) was used to determine the performance factors for all beams and can be found in Table 4.4.

\[
P_F = \left(\frac{\Delta_{\text{ultimate}}}{\Delta_{\text{service}}}\right) \times \left(\frac{M_{r, \text{ultimate}}}{M_{r, \text{service}}}\right)
\]

Table 4.4: Strength, Deflection and Performance Factors of Strengthened Beams

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Deflection Factor</th>
<th>Strength Factor</th>
<th>Performance Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1PR-B04</td>
<td>6.9</td>
<td>2.2</td>
<td>15.2</td>
</tr>
<tr>
<td>1PR-B06</td>
<td>5.3</td>
<td>2.3</td>
<td>12.3</td>
</tr>
<tr>
<td>1PR-B08</td>
<td>5.0</td>
<td>2.2</td>
<td>10.7</td>
</tr>
<tr>
<td>0.75PR-B02</td>
<td>8.4</td>
<td>2.5</td>
<td>20.8</td>
</tr>
<tr>
<td>0.75PR-B04</td>
<td>9.3</td>
<td>2.4</td>
<td>22.3</td>
</tr>
<tr>
<td>0.75PR-B08</td>
<td>6.8</td>
<td>3.0</td>
<td>20.3</td>
</tr>
<tr>
<td>0.5PR-B02</td>
<td>11.8</td>
<td>2.7</td>
<td>31.7</td>
</tr>
<tr>
<td>0.5PR-B04</td>
<td>15.9</td>
<td>3.3</td>
<td>51.8</td>
</tr>
<tr>
<td>0.5PR-B08</td>
<td>6.1</td>
<td>3.0</td>
<td>18.1</td>
</tr>
</tbody>
</table>

Studying the performance factors in this table, the beams with the best performance are the ones with the lower internal reinforcement area. These beams were able to maintain the most deflection until failure, while still gaining the greatest increase to their moment capacity (strength factor). The beams reinforced with two and four layers of BFRP were the most efficient. The low
performance value for the eight-layer beams is due to over-reinforcing, which excessively increased the stiffness in terms of load-deflection behavior in the elastic range. Thus, these strengthened beams exhibited a very high strength at a relatively low mid-span deflection (Figures 4.1-4.3). This caused the beams to fail in a brittle manner, either due to shear-tensile failure in concrete (0.5PR-B08 and 0.75PR-B08) or due to sudden and severe concrete crushing at the compression zone (1PR-B08). Thus, it can be concluded that the number of BFRP layers needs to be limited such that such brittle failure modes are avoided.

For all the beams tested, the best ratio of strength and deflection increase comes from the beams reinforced with four layers of BFRP. The most efficient strengthened beam is 0.5PR-B04, experiencing the best strength increase, and the highest ductility. This concludes that there are limits in efficiently strengthening beams with different internal reinforcements. The first is the depth of the neutral axis, and the second is the beam’s load-deflection stiffness. While the strength can keep increasing, the deflection will reach a point where it starts decreasing at a pace which reduces the effectiveness of applying more BFRP reinforcement.

4.1.6 SPECIMEN ENERGY ABSORPTION

Another method used to study the specimen was the calculation of the beam’s energy absorption. This is the amount of energy the beam can absorb before failure in the concrete beam. This value was found using the load-displacement graphs in Figure 4.1, Figure 4.2, and Figure 4.3. The fracture energy was calculated for each beam up to the ultimate strength. This ensured that the energy absorption was compared at the same behaviour point on each beam, and ensured it was
not affected by where the test was concluded. The energy for all specimens were calculated using Riemann Sums, seen in Equation (4.10), by calculating the area under the load-deflection curve.

\[
S = \sum_{i=1}^{n} \left[ (\Delta_{i+n} - \Delta_i) \ast \left( \frac{f_i + f_{i+n}}{2} \right) \right]
\]  

(4.10)

The results for the specimen energy for all the beams can be observed in Table 4.5 and in Figure 4.8, which shows the variation in energy absorption between the different layers and specimens.

![Figure 4.8: Fractural Beam Energy](image)

Figure 4.8: Fractural Beam Energy
Table 4.5: Energy Absorption, and Change in Energy Absorption for Specimens

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Fracture Energy (J)</th>
<th>Δ Fracture Energy (J)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1PR-Control</td>
<td>1150</td>
<td>-</td>
</tr>
<tr>
<td>1PR-B04</td>
<td>4600</td>
<td>3450</td>
</tr>
<tr>
<td>1PR-B06</td>
<td>3730</td>
<td>2580</td>
</tr>
<tr>
<td>1PR-B08</td>
<td>3715</td>
<td>2565</td>
</tr>
<tr>
<td>0.75PR-Control</td>
<td>2170</td>
<td>-</td>
</tr>
<tr>
<td>0.75PR-B02</td>
<td>3670</td>
<td>1500</td>
</tr>
<tr>
<td>0.75PR-B04</td>
<td>4620</td>
<td>2450</td>
</tr>
<tr>
<td>0.75PR-B08</td>
<td>3350</td>
<td>1180</td>
</tr>
<tr>
<td>0.5PR-Control</td>
<td>1670</td>
<td>-</td>
</tr>
<tr>
<td>0.5PR-B02</td>
<td>2930</td>
<td>1260</td>
</tr>
<tr>
<td>0.5PR-B04</td>
<td>5100</td>
<td>3430</td>
</tr>
<tr>
<td>0.5PR-B08</td>
<td>2480</td>
<td>810</td>
</tr>
</tbody>
</table>

It can be observed above, this relationship between the strengthened and unstrengthened beams is the inverse of the performance factor. As the internal reinforcement of the strengthened beam increases, it can absorb more energy until ultimate load. The specimen energy is proportional to the load and deflection of the beam. This will in turn be influenced by the cross section and internal reinforcement.

Figure 4.8 shows how much energy can be absorbed by each specimen as the layers of BFRP increases. It shows that the most absorption energy is captured when four layers of BFRP is used, and this increase is slightly higher at lower internal reinforcement ratios. This trend compliments what is observed in Table 4.4. The beams reinforced with four layers of BFRP exhibit
the highest energy absorption. The explanation is because the energy capacity is a function of how much strength the beam exhibits, along with how far it can be displaced. The reason that the four layers of BFRP have the highest energy absorption is because they have the best strength-to-displacement ratio of all reinforcement schemes tested, making Specimen 0.5PR-B04 the most efficient. The energy absorption and its increase from 1PR beams to 0.5PR beams when looking at the B04 beams is due again to the higher control beam ductility. This increased control beam ductility allows the strengthened beams to capture more strengthened ductility, increase the effectiveness of the BFRP.

4.1.7 SPECIMEN DUCTILITY

While analyzing the ductility of the beams, three ductility expressions were calculated to quantify the ductility seen in Figure 4.1 to Figure 4.6. These expressions are the curvature, deflection and energy ductility values for each beam, which are shown in Equation (4.11), (4.12), and (4.13) respectively. For the following equations, \( \phi_u \), \( \Delta_u \), and \( E_u \) represent the values of the beams’ curvature, deflection and energy respectively at the maximum flexural moment. \( \phi_s \), \( \Delta_s \), and \( E_s \) represent the beams’ curvature, deflection and energy at the service load for each beam as determined by clause 10.6.1 in A23.3-14.

\[
\mu_\phi = \frac{\phi_u}{\phi_s} \quad (4.11)
\]
\[
\mu_\Delta = \frac{\Delta_u}{\Delta_s} \quad (4.12)
\]
\[
\mu_E = \frac{E_u}{E_s} \quad (4.13)
\]

These values, shown in Table 4.6, give an indication of the beam’s ductility, and how it changes as the layers of BFRP increase. The change in the ductility of these beams, in terms of
Deflection, curvature, and energy, will vary depending on the amount of external fabric reinforcement. The change in ductility for these three characteristics can be seen in Figure 4.9 for the 1PR beams, in Figure 4.9 for the 0.75PR beams, and in Figure 4.10 for the 0.5PR beams. These figures show how the beams ductility changes as more layers of BFRP are applied for the same reinforcement ratio. The values presented in these figures represent the ratios of the beam ductility value compared to the control value. Out of all the beams, the most ductile repairs when considering the Equations (4.11), (4.12), and (4.13) resulted from the beams strengthened with four layers of BFRP. This confirms that this is the most efficient strengthening scheme in terms of preserving the strengthened beam’s ductility.

Figure 4.9: 1PR Beams’ Normalized Ductility Values
Figure 4.10: 0.75PR Beams’ Normalized Ductility Values

Figure 4.11: 0.5PR Beams’ Normalized Ductility Values
The data presented in Table 4.6 consists of the non-normalized ductility values, which show the same trends that have been discussed. The ductility for all three of these criteria always increases from the control up to the beam which is strengthened with four layers of BFRP. After this optimal point, there is regression in the calculated beam ductility value. The worse values come from the beams which were pushed into a shear failure due to critically increased stiffness.

4.1.8 DISCUSSION

Through this section, the overall strength, ductility and performance of the test specimens was analyzed. The following conclusions have been drawn from the data that have been presented.
and analyzed. While the results and test data are dependent on each specimen, the results which have been presented fall in line with similar tests presented in the literature review.

A significant increase in flexural resistance ($M_r$) can be obtained using BFRP flexural fabrics, both to the yield and the ultimate capacity. Increases of up to 120% in flexural capacity were seen from the beams strengthened with BFRP fabrics over the un-strengthened control beams. This is a significant increase in capacity and shows that BFRP is effective as a strengthening material.

Through using BFRP fabrics, this study found that a beam’s ultimate ductility can be increased over the control beams ultimate ductility. For a specimen’s yield ductility, the values were very similar for both the strengthened and control beams. No significant reduction in ductility occurs, and in some cases the yield point develops at a higher deflection. This is due to the high tensile strain basalt can reach, before ultimate failure. This is an excellent result, as previous studies had found that beams become very brittle when strengthened with CFRP or GFRP fabrics. The BFRP helps to extend the ultimate ductility, allowing the flexural beam to fail in a gradual manner even after strengthening. There is a limit when the beam becomes over strengthened, and so care needs to be taken when deciding the amount of fabric to add.

The optimal number of BFRP layers through this test matrix was found to be four layers of BFRP fabric. For all internal reinforcement ratios, the beams strengthened with four layers of BFRP had the best performance when considering both the increase in strength and the ultimate deflection. While this strengthening technique still does increase the overall beam stiffness, this can be managed to obtain optimal results for both deflection and increase in capacity. The more ductile nature of basalt when compared to the glass and carbon alternatives allows the beams to experience more deflection before failure than CFRP and GFRP would allow. This enables the beams to fail in a safe flexural manner, if the amount of reinforcement added is reasonable.
4.2 SPECIMEN STRAIN DATA

4.2.1 CONCRETE STRAIN PROFILES

Concrete strain profiles give an accurate map of all the strain concentrations in the concrete, and crack formations. Historically, the mapping of strain profiles has been accomplished using 60 mm strain gauges placed on the concrete surface. This can be problematic due to the brittle nature of concrete, as discussed in Section 3.3.3. To provide an accurate strain composition of the concrete beam surface, Digital Image Correlation (DIC) was used. With the strain data shown in Figure 4.12, Figure 4.13, and Figure 4.14, all the cracks and strain concentrations in the concrete surface, and along the interface between the BFRP and the concrete can be observed. Visualization of the strain concentrations help show the crack formation behaviour in the beam.

Figure 4.12: DIC on the Left Side of Specimen 0.5PR-B02

Figure 4.13: DIC on the Center of Specimen 0.5PR-B02
It was observed while analyzing the DIC data that there is an increase in concrete damage, especially near the tensile fibres, as the layers of BFRP are increased. This is likely due to the increase in confinement which the layers provide, which can effectively add a crushing force to the tensile face as the BFRP resists the vertical displacement. The DIC strain data was also used to construct the crack pattern profiles of the beams at ultimate load.

4.2.2 INTERNAL REBAR STRAIN PROFILES

The internal strain was monitored using 5 mm strain gauges, placed on the rebar cages as illustrated in Figure 3.10. Redundant strain gauges were installed in case of damage during the casting process. These strain gauges were monitored during the testing process through the DAQ and recorded in excel workbooks for analysis. This analysis gives insight into the rebar behaviour throughout the testing.

One thing to note is the difference in the compression rebar strain between the control beams and the beams which were strengthened with BFRP fabrics. This is part of the reason for the reduction in beam strength after BFRP rupture, observed in Figure 4.2 and Figure 4.3. Due to the restraining force that the tensile fabric applies on the beam when increasing its load-deflection stiffness, more damage occurs to both the tensile and compressive internal reinforcement. Due to the increased damage, the bars are not able to hold nearly as much load post-rupture. This is the reason for the observed strength reduction of Specimens 0.75PR-B02, 0.75PR-B04, 1PR-B04, and
1PR-B06 when compared with the control beams, which did not experience this elevated damage.

This strain increase between Specimen 0.75PR-Control and Specimen 0.75PR-B04 are shown in Figure 4.15 and Figure 4.16.

Figure 4.15: Flexural Rebar Strain Gauge Data 0.75PR-Control Beam

Figure 4.16: Flexural Rebar Strain Gauge Data 0.75PR-B04 Beam
Figures 4.15 and 4.16 show the increase in strain in the compression rebar. This is due to the increase in internal moment during the specimen testing, and the increased restraint from the bottom layer of BFRP. For Specimen 0.75PR-B04, as seen in Figure 4.15, the strain in the compressive rebars reaches and slightly exceeds 0.2%, the yield strain of steel. This increase in strain was not present in the control beams and became more severe in the strengthened beams as the layers of BFRP reinforcement were increased. This increase in strain is what contributes to the reduction in post-rupture BFRP strengthened beams shown in Figure 4.2 and Figure 4.3 when compared to the control specimens.

4.2.3 EXTERNAL BFRP STRAIN BEHAVIOUR

The strain was calculated globally by fitting a curve to the coordinates which were provided from the two supports and the three LVDTs. A line integral was then taken between these points to determine the length of the curve, using Equation (4.14). This global strain value which was calculated was compared to the strain value obtained from a strain gauge in the center of the BFRP. The center strain gauge values will vary depending on the amount of strain the gauge was able to capture before it failed.

The strain values provided by the center gauges validate the global strain value obtained using the line integral method. Equation (4.14) was used to compare the change in the length of the BFRP fabrics to its starting length in order to determine the strain in the entire system. This result provides a stress value for the entire 1.9 m span. However, because of the excessive damage in the concrete interface when the BFRP ruptures, the fabric was considered unbonded. With this significant damage to the concrete after the steel yields and up to the BFRP ruptures, much of the resistance from the BFRP comes from the arcing resistance to the beam’s displacement. This stress gets transferred as a tensile force into the anchorage at the ends of the span, instead of being
transferred through in-plane stresses. Therefore, the line integration method provides an accurate means to calculate the strains in the BFRP at the mid-span. If the fabric is unbonded, it will have a constant strain across that unbonded length.

\[ l = \int_{0}^{1.9} \sqrt{1 + (f'(x))^2} \, dx \quad (4.14) \]

\[ \sigma_{FRP} = \epsilon_{FRP}E_{FRP} \quad (4.15) \]

The value of \( E_{FRP} \) was calculated through material testing as per ASTM Standard D3039/3039M, at the University of Windsor.

The results from the material testing is presented in Table 1.2. The tensile stresses in the BFRP (at ultimate load), listed in Table 4.7, were calculated using the relationship between stress and strain shown in Equation (4.15).

As can be observed in Table 4.7, lower reinforcement ratio beams are able to withstand higher strains in the BFRP due to larger allowable deflections. Globally, all the beams were able to meet or exceed 2.2% strain in the BFRP, similar to the ultimate strain value of the tensile coupons in the material testing. Specimens 0.75PR-B08 and 0.5PR-B08 both failed at 2.16% strain in the BFRP, due to the beams becoming over-reinforced and experiencing shear-tensile failure as discussed. This failure mode prevented the development of high strain in the BFRP because the specimens failed prematurely.
Table 4.7: External BFRP Strain and Stresses

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Ultimate Strain in BFRP (%)</th>
<th>Ultimate Stress in BFRP Stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1PR-B04</td>
<td>2.23</td>
<td>468</td>
</tr>
<tr>
<td>1PR-B06</td>
<td>2.19</td>
<td>460</td>
</tr>
<tr>
<td>1PR-B08</td>
<td>2.28</td>
<td>479</td>
</tr>
<tr>
<td>0.75PR-B02</td>
<td>2.21</td>
<td>464</td>
</tr>
<tr>
<td>0.75PR-B04</td>
<td>2.23</td>
<td>468</td>
</tr>
<tr>
<td>0.75PR-B08</td>
<td>2.16</td>
<td>454</td>
</tr>
<tr>
<td>0.5PR-B02</td>
<td>2.19</td>
<td>460</td>
</tr>
<tr>
<td>0.5PR-B04</td>
<td>2.28</td>
<td>479</td>
</tr>
<tr>
<td>0.5PR-B08</td>
<td>2.16</td>
<td>454</td>
</tr>
</tbody>
</table>

The stress in the BFRP at rupture was calculated using Equation (4.15). The center values consist of a large percentage of the global strain, suggesting that most of the stress is concentrated in the middle. The stress also gets distributed through the entire length of fabric bonded to the concrete. This elevated strain and stress concentration in the center of the beam is due to the more ductile manner of the beam; allowing it to deflect and build up more stress, before rupture.

4.2.4 DISCUSSION

One trend which presents itself when studying the strain values of the concrete, and rebar is the increased cracking damage in the concrete and increased strain in the rebar which occurs as more strengthening is applied to the beam. This is due to the increased load-deflection stiffness and ultimate load which the beam experiences before the BFRP rupture. This higher load is exposing the rebar and concrete substrate to higher strains and more damage, which is not present.
in the control beams. This is the reason for the lower post failure behaviour of the beams, observed in Figure 4.2 and Figure 4.3, when compared to the control specimens. The control specimen does not experience this damage and so has a higher capacity at the same deflection as the strengthened beams do, once the BFRP fabric has ruptured. When studying the BFRP, all the beams experience BFRP rupture at very similar global strains which is expected. The stresses remain the same, and so the increase in load capacity comes from the increase in cross sectional area, like internal steel reinforcement. Just like the internal steel reinforcement, the load increases as the cross-sectional area increase. This continues until the beam becomes over-reinforced and starts to exhibit shear failure characteristics.
4.3 SPECIMEN FAILURE

The specimen failure is one major advantage of using BFRP as a flexural reinforcement material over both CFRP and GFRP. Basalt, as a fabric, can sustain much higher strains before rupture leading to a more ductile failure of strengthened beams. Table 4.7 shows the ultimate strain values the specimens reached during testing. It is because of these high strains that the specimens experienced more deflection in the load-deformation and moment-curvature graphs. When using any concrete structural member, and especially when strengthening with composite fabrics, great care needs to be taken to not over strengthen the beam. However, when not over strengthened, the use of BFRP as a construction material is a great option to maintain concrete beam ductility.

4.3.1 CONCRETE CRACK PATTERNS

The crack patterns were analyzed to study the damage which had occurred in the beams when they have reached ultimate capacity. Figure 4.19 to Figure 4.29 show the crack patterns for all the beams that have been tested. The cracking damage is much more severe as the amount of fibre layers are increased, especially along the bond interface between the concrete and the BFRP. The crack density also increases, especially in the midspan. The cracks start propagating into the shear spans as well. This is due to the confinement which the BFRP adds to the tensile face, which experiences increasing pressure as the deflection increases. It is because of this increase in pressure, that the BFRP starts to apply a compressive force on the concrete and can crush the bottom face before it ruptures. This cracking pattern is observed forming on two separate beams in Figure 4.17 and Figure 4.18. This is part of the reason that the strengthened beams are not able to maintain the same post-rupture load as the control beams. This damage to the concrete substrate, along with the higher strains in the rebar, shown in Figure 4.15, is the reason for the reduction in load between the strengthened and control beams post BFRP rupture.
Figure 4.17: Cracking Near Concrete-BRFP Interface in Specimen 0.75PR-B06

Figure 4.18: Cracking Near Concrete-BFRP Interface in Specimen 0.75PR-B04

Figure 4.19: Crack Pattern of 0.5PR-Control

Figure 4.20: Crack Pattern of 0.5PR-B02

Figure 4.21: Crack Pattern of 0.5PR-B04
Figure 4.22: Crack Pattern of 0.5PR-B08

Figure 4.23: Crack Pattern of 0.75PR-Control

Figure 4.24: Crack Pattern of 0.75PR-B04

Figure 4.25: Crack Pattern of 0.75PR-B08

Figure 4.26: Crack Pattern of 1PR-Control
It is through the degradation of this bond between the concrete and the BFRP, which enables the use of the line integral to accurately calculate the strain in the BFRP along the beam’s flexural span. It is assumed that due to the damage in the concrete substrate after the beam has yielded, very little in-plane shear stress is transferred between the fabric and the concrete substrate. It is because of this, a great portion of the tensile stresses are assumed to transfer to the end of the beams through the anchor points, to the relatively undamaged concrete sections in the shear zone. Essentially, the tensile stress that is accumulated in the BFRP from resisting the bending of the beam is transferred directly to the u-wrap anchor points.
4.3.2 DISCUSSION

When designing concrete structural members, it is important to ensure ductile failure, to ensure safety. The results of this study show that basalt is effective for ensuring that members strengthened with BFRP can still fail in a safe, ductile manner. The only limitation is over strengthening the beam, similarly to internal steel reinforcement. With internal rebar there is also the risk of over reinforcing the beam to the point of pushing the beam towards a shear failure. This is something that needs to be addressed in the design process. When strengthening with BFRP, the risk of flexural cracks propagating out into the shear zones increases with number of layers of fabric applied. Therefore, there needs to be a limit to how much strengthening can be applied flexurally. After this limit, reinforcement or strengthening may also need to be provided in the shear zone as well to compensate for any deficiencies.
4.4 TEST RESULTS COMPARISON WITH APPLICABLE CODES

4.4.1 CODES CONSULTED, AND ASSUMPTIONS FORMED

During the research many structural design codes and guide books were consulted and compared against the test results to determine their accuracy. These include the Intelligent Sensing for Innovative Structures (ISIS) design manual for FRP Rehabilitation of Reinforced Concrete Structures [4], The Canadian Highway Bridge Design Code CSA S6-14 [31], Design and Construction of Building Structures with Fibre-Reinforced Polymers (CSA S806-12) which provides guidance on the design of buildings with FRPs [32], and CSA S807-10 for FRP specifications [33]. ACI 440.2R guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures [34] was also considered.

For flexural analysis while strengthening and rehabilitating concrete members with FRP fibres, the codes all make some assumptions, which are as follows:

- Internal stresses at a cross-section are in equilibrium with the effects of the applied loads
- Plane sections remain plane
- There is strain compatibility between adjacent materials, implying perfect bonds exist between the steel, concrete and FRP. Strain compatibility also implies that the strain change in the FRP strengthening system is equal to the strain change in the adjacent concrete after the initial strain
- The maximum tensile strain of the FRP is equal to the code specified allowable tensile strength; this varies between codes
- Maximum compressive strain in concrete is per code (0.0035 in Canadian codes)
• The contribution of FRPs in compression and of concrete in tension are negligible

Additional assumptions were made throughout the research, either during the construction and testing of the specimens, or during the analysis of the results. The additional assumptions that were made are listed below:

• Cross-strapping, while located in the shear span, does not increase shear capacity. This is due to it not fully constraining the shear depth, which is needed to increase shear capacity

• After global beam yielding, the BFRP composite transfers the full stress of the fibres to the cross-straps that anchor the endpoints of the composite to the beam.

4.4.2 FRP STRAIN LIMITS

Each code has a different definition for the maximum allowable strain which the FRP strengthening or rehabilitation system can reach. All codes, from both CSA and ACI only take into consideration CFRP and GFRP materials, since BFRP is a much newer construction material. As such, the codes can be at times be too conservative with their strain calculations, due to the previously used materials being much stiffer than basalt. The Canadian Highway Bridge Design Code CSA S6-14 states that the maximum allowed strain is \( \varepsilon_{frpu} \leq 0.006 \). This is acceptable when using CFRP which has a very low rupture strain of 0.9% - 1.0%. It is overly conservative however, for a material like BFRP which can have rupture strains of up to 2.5%. When designing based off CSA S806-12, Equation (4.16) is used to determine the maximum allowable FRP strain.

\[
\varepsilon_{F, max} = 0.41 \frac{f'_c}{\sqrt{n_F E_F t_F}} \leq 0.007
\] (4.16)
CSA S806-12 takes into consideration a range of maximum strain values by considering a ratio with the fibre modulus and the concrete crushing strength. It still however, limits the strain value to a maximum of 0.7%, which is very conservative for BFRP. ACI takes a more rounded approach when calculating their maximum strain in ACI 440.2R, which calculates the maximum design rupture strain in Equation (4.17).

$$\varepsilon_{fd} = 0.41 \sqrt{\frac{f'_c}{n_f E_f t_f}} \leq 0.9 \varepsilon_u$$  \hspace{1cm} (4.17)

This method used by the ACI 440.2R is a more encompassing method for calculating the maximum allowable design strain within the FRP fabrics. This formula encompasses all different materials which are used for external fibre reinforcement, and the strain is determined based on material specific ultimate strain. Table 4.8 shows the calculated maximum strain values as specified by different codes.

**Table 4.8: Ultimate Strain Calculated by Canadian and American Codes**

<table>
<thead>
<tr>
<th>Beams With 4 Layers BFRP</th>
<th>CSA S6-14</th>
<th>CSA S806-12</th>
<th>ACI 440.2R</th>
<th>Experimental Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varepsilon_{frpu}$</td>
<td>0.6%</td>
<td>0.7%</td>
<td>2.0%</td>
<td>2.35%</td>
</tr>
</tbody>
</table>

Especially for a high ultimate strain material such as basalt, the two Canadian codes are far too conservative in the restriction of the $\varepsilon_{frp}$. Conversely, the American code takes a better approach by giving the option of up to 90% of the ultimate strain found through testing. This research suggests adopting a method similar to the ACI 440.2R method [34], which encompasses a wider variety of material when calculating design strain. This is very important, as the strain is ultimately used to determine the stress in the FRP, and strength provided to the beam.
4.4.3 FRP STRENGTH ESTIMATION

The most widely accepted method for calculating the moment resistance of flexural members is through the strain compatibility hypothesis. The following Equations (4.18-4.22) are used to calculate the moment resistance of beams strengthened with BFRP fabrics.

\[ M_r = C_s \left( \frac{a}{2} - d' \right) + T_s \left( d - \frac{a}{2} \right) + T_{\text{FRP}} \left( h - \frac{a}{2} \right) \quad (4.18) \]

\[ C_c = \alpha_1 \phi_c f'_{c} \beta_1 cb \quad (4.19) \]

\[ C_s = \phi_s E_s A'_s \varepsilon'_s \quad \varepsilon'_s \leq \varepsilon_y \quad (4.20) \]

\[ T_s = \phi_s E_s A_s \varepsilon_s \quad \varepsilon_s \leq \varepsilon_y \quad (4.21) \]

\[ T_{\text{FRP}} = \phi_{\text{FRP}} E_{\text{FRP}} A_{\text{FRP}} \varepsilon_{\text{FRP}} \quad (4.22) \]

The calculation of the moment resistance depends on the failure mode which has been assumed for the strengthened beam. The strain values in concrete, steel and FRP, as well as the beam’s neutral axis will all change based on whether it fails due to FRP rupture or concrete crushing. These failure modes will change which strain values will be used in Equations (4.19-4.22) and in Equations (4.23-4.28). If the failure mode is initiated by concrete reaching 0.0035 and crushing, then Equations (4.23-4.25) are used. If failure is initiated due to FRP rupture before the concrete crushes, Equations (4.26-4.28) are used. In the following formulae, \( \varepsilon'_s \) is the strain in the compressive rebar, \( \varepsilon_s \) is the strain in the tensile rebar, \( \varepsilon_{\text{FRP}} \) is the strain in the FRP, \( \varepsilon_{\text{cu}} \) is the ultimate concrete strain, and \( \varepsilon_{\text{fi}} \) is the initial strain in the structure when the FRP is applied, if applicable.

\[ \varepsilon'_s = \varepsilon_{\text{cu}} \left( \frac{c - d'}{c} \right) \quad (4.23) \]

\[ \varepsilon_s = \varepsilon_{\text{cu}} \left( \frac{d - c}{c} \right) \quad (4.24) \]
\[ \varepsilon_{FRP} = \varepsilon_{cu} \left( \frac{h - c}{c} \right) - \varepsilon_{fi} \]  
(4.25)

\[ (\varepsilon_{fi} + \varepsilon_{FRPu}) \left( \frac{c - d'}{h - c} \right) \leq \varepsilon'_{s} = (\varepsilon_{fi} + \varepsilon_{FRPu}) \left( \frac{c - d'}{h - c} \right) \]  
(4.26)

\[ (\varepsilon_{fi} + \varepsilon_{FRPu}) \left( \frac{d - c}{h - c} \right) \leq \varepsilon_{s} = (\varepsilon_{fi} + \varepsilon_{FRPu}) \left( \frac{d - c}{h - c} \right) \]  
(4.27)

\[ (\varepsilon_{fi} + \varepsilon_{FRPu}) \left( \frac{c}{h - c} \right) \leq \varepsilon_{c} = (\varepsilon_{fi} + \varepsilon_{FRPu}) \left( \frac{c}{h - c} \right) \]  
(4.28)

To use these equations, the neutral axis needs to be determined. This can be accomplished by equating the compressive and tensile forces as shown in Equation (4.29).

\[ C_{c} + C_{s} = T_{s} + T_{FRP} \]  
(4.29)

All the above formulae can be substituted into Equation (4.29) and rearranged to find the value for the depth of the concrete compressive block, \( c \). This formula will also be changed again, based on the failure mode, and can be simplified. The equation is simplified to Equation (4.6) if the failure is due to concrete crushing, and Equation (4.7) if the failure is FRP rupture.

Once the concrete compressive block location is determined, the moment resistance for the strengthened beam can be calculated. The above process is what a structural designer would use to predict and design the strength increase for a concrete flexural beam with FRP fabrics. As stated, the Canadian CSA codes were written considering lower ultimate strain materials. This can skew the actual results away from what is observed in this experiment when compared to the results predicted by the structural FRP codes. Table 4.9 compares the yield strength calculated from the codes to the experimental values. These calculations were made using the guidance from ACI
440.2R as described in Section 4.4.2, with regards to increasing the strain design limits for the BFRP fabrics.

Structures are designed to perform at a strength that is between the yield and ultimate strength. This value will fall somewhere along the beginning of the plasticity curve of the structural member. To account for this, designers use resistance factors for the steel, concrete, and FRP to ensure that the design is conservative. Therefore, the moment resistance for each beam was calculated with and without the use of resistance factors. The theoretical values without the resistance factors all fall within the range of the experimental yield and ultimate beam values. When calculating the moment resistance with the resistance factors, these values are very close to the experimental yield strength of these beams, as can be observed in Table 4.9. This is an excellent result as it means that this method of design, which is already widely accepted and used, is still viable for flexural BFRP fabric reinforcement, provided that the correct ultimate allowable strain is changed to match the ACI 440.2R [34] code.
Table 4.9: Comparison of Strength from Code, and Experimental Strength

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Experimental Yield (kNm)</th>
<th>Experimental Ultimate (kNm)</th>
<th>Theoretical $M_r$ (kNm)</th>
<th>Change in Experimental Yield to Theoretical Yield</th>
</tr>
</thead>
<tbody>
<tr>
<td>1PR-B04</td>
<td>33.01</td>
<td>43.79</td>
<td>34.81</td>
<td>42.97</td>
</tr>
<tr>
<td>1PR-B06</td>
<td>38.23</td>
<td>53.52</td>
<td>35.00</td>
<td>43.27</td>
</tr>
<tr>
<td>1PR-B08</td>
<td>43.44</td>
<td>56.30</td>
<td>36.77</td>
<td>45.65</td>
</tr>
<tr>
<td>0.75PR-B02</td>
<td>27.11</td>
<td>39.96</td>
<td>23.43</td>
<td>28.69</td>
</tr>
<tr>
<td>0.75PR-B04</td>
<td>28.50</td>
<td>41.01</td>
<td>29.31</td>
<td>36.54</td>
</tr>
<tr>
<td>0.75PR-B08</td>
<td>33.01</td>
<td>59.08</td>
<td>34.00</td>
<td>42.82</td>
</tr>
<tr>
<td>0.5PR-B02</td>
<td>20.85</td>
<td>33.71</td>
<td>19.88</td>
<td>24.9</td>
</tr>
<tr>
<td>0.5PR-B04</td>
<td>22.59</td>
<td>44.13</td>
<td>23.54</td>
<td>29.78</td>
</tr>
<tr>
<td>0.5PR-B08</td>
<td>27.80</td>
<td>49.69</td>
<td>28.63</td>
<td>36.57</td>
</tr>
</tbody>
</table>

4.4.4 FRP DEVELOPMENT LENGTH

An important factor to consider when flexurally strengthening concrete beams is the development length or anchorage and ensuring a proper length of fabric is utilized. All three of the structural codes state slightly different methods to determine the minimum development length for flexural fabric reinforcement. CSA S6-14 [31] uses Equation (4.30), while CSA S806 uses Equation (4.31).

\[
l_a = 0.5 \sqrt{E_{FRP} t_{FRP}} \geq 300 \text{ mm} \tag{4.30}
\]

\[
l_a = \frac{n_{FRP} E_{FRP} t_{FRP}}{\sqrt{f_c}} \geq 300 \text{ mm} \tag{4.31}
\]
Both codes also state that an anchorage system can be developed if the recommended development length cannot be achieved. This anchorage system for both codes needs to be tested in laboratory environments simulating what will be experienced in the fields and approved by a Professional Engineer. The ACI code 440.2R [34] takes a slightly more encompassing approach. It has some guidelines for an anchorage system, shown in Equation (4.32), along with its development Equation (4.31).

\[
A_{f_{anchor}} = \frac{(A_{ff_{e}})_{longitudinal}}{(E_f \kappa \varepsilon_{fu})_{anchor}}
\]

(4.32)

Where \( \kappa_v \) is a term which considers the bond-reduction coefficient. This uses the concrete strength, type of wrapping scheme, and stiffness of the composite to determine this factor. This term can be determined using Equations (4.33 - 4.36).

\[
\kappa_v = \frac{k_1 k_2 L_e}{11,900 \varepsilon_{fu}}
\]

(4.33)

\[
k_1 = \left( \frac{f'_c}{27} \right)^{2/3}
\]

(4.34)

\[
k_2 = \begin{cases} 
\frac{d_{fv} - L_e}{d_{fv}} & \text{for } U - \text{wraps} \\
\frac{d_{fv} - 2L_e}{d_{fv}} & \text{for } 2 \text{ sides bonded}
\end{cases}
\]

(4.35)

\[
L_e = \frac{23,300}{(nt_{FRP} E_{FRP})^{0.58}}
\]

(4.36)
In conclusion, the ACI 440.2R [34] code takes a more encompassing approach to the problem. The code provides formula guidelines for the suggested anchoring, instead of leaving it to the engineering and testing as seen in the CSA codes. For the development length, both codes use similar guidelines in both the CSA and ACI codes. In this research program, none of the beams experienced premature debonding in the fabric in the flexural span. This is due to the anchoring system which was used. However, the development length provided in this study was 550 mm, which is greater than the suggested 300 mm length from the CSA codes. It appears that these guidelines work for BFRP as well, and the anchoring system that was implemented prevented premature delamination. As it was not the focus of this research, however, more research would need to be conducted to see if the development length formulae are accurate for a more ductile material such as basalt.

4.4.5 DISCUSSION

When comparing the experimental values to the CSA and ACI codes, the method presently utilized in the codes do not do an adequate job at determining the allowable design strain in a fabric with a high ultimate strain, such as BFRP. The design codes are supposed to be created to be technologically neutral, in that there should not be a bias based on the material which is being used in the design. For both CSA S6-14 and CSA S806-12 the allowable strain is far too conservative for a fabric made from material such as basalt, which exhibits a high ultimate strain. Both CSA S6-14 and S806-12 need to be updated to reflect ACI 440.2R which encompasses a wider range of materials.

If the Canadian code is changed to reflect the method used in the American code in calculating the design strain, the method of estimating the design strength of flexural beams strengthened with BFRP using the compatibility theorem becomes accurate. This theorem was
able to accurately predict the beam’s yield strength to within 12% of the experimental value. This method also always predicts an estimated value between the yield point and ultimate strength of the strengthened beam. The values predicted by the American code are always closer to the yield point than the ultimate. When the material safety factors are added, the predicted yield load is spot on or slightly lower than the experimental yield load of the strengthened beams. These predicted values are very accurate considering that there are always variations between concrete beams due to the non-homogeneous nature of the concrete and BFRP material.

Once CSA S6-14 and CSA S806-12 are updated to reflect the design strain calculations found in ACI 440.2R, the Canadian codes will be able to accurately predict the flexural strength of concrete beams strengthened with BFRP fabrics.
CHAPTER 5

CONCLUSION

This study investigated the use of BFRP fabrics to strengthen reinforced concrete beams. Through the research conducted throughout this study, the following conclusions can be drawn. While these results can be highly dependent on the specimen and testing method, they do coincide with similar results found in previous studies and the literature.

1. Throughout this study, basalt fibres have proven to be a viable alternative when using external FRP reinforcement to strengthen reinforced concrete beams. This method shows many improvements over both CFRP and GFRP. BFRP is effective at increasing the yield and ultimate load capacity of flexural beam. It does this without drastic reductions to the beam’s ductility as seen with CFRP and GFRP reinforced beams. When studying the effect on the ultimate strength and deflection, BFRP fabrics were able to increase the deflection a beam could experience before its ultimate flexural capacity.

2. Flexural concrete beams reinforced with BFRP fabrics provide a much safer failure mode that those reinforced with CFRP and GFRP. This is due to the high fracture strain which basalt fabrics exhibit. The failure mode of these beams, unless significantly over-reinforced, can undergo high deflection and ductility before failure. This allows structural engineers to catch and address any issues before catastrophic failure.

3. The guidelines provided in the Canadian structural codes CSA S6-14 and S806-12, as well as the American structural code ACI 440.2R, provide a good estimate of the capacity of BFRP strengthened beams. While they were created and designed for CFRP and GFRP, they work for Basalt fibres if the ultimate allowable strain is updated. In both Canadian
codes the ultimate strain is capped at a constant value and not at a percentage of the ultimate strain as seen in ACI 440.2R. The ACI method is a more efficient and encompassing way for the design code to estimate the repair strength, especially with high strain materials. The CSA codes need to be updated to reflect ACI 440.2R so that the codes can become technologically neutral.

4. When using BFRP fabrics to flexurally strengthen concrete structures, it is recommended to also strengthen the shear span. Structurally strengthening the shear span allows the beam to retain its unstrengthened failure mode, which is critical to ensure that the member is not over-strengthened, and the failure mode changes to a brittle failure. The shear-tensile failure observed in this study would have been prevented if the shear span was fully confined with BFRP.

5. Using BFRP as a strengthening fabric is much cheaper than CFRP and GFRP. The raw material is much more abundant, and the manufacturing method is similar to glass fibres. The infrastructure to manufacture BFRP fabrics is already available. This reduction in cost can be massive and will greatly reduce the cost to maintain aging infrastructure.

More research needs to be conducted on the estimated development lengths which are estimated in CSA S6-14 and S806-12, and in ACI 440.2R. With these promising findings, and basalt materials having a much smaller financial and environmental impact than other widely used alternative, it is expected that BFRP fabrics will become widely accepted and used in construction. Using BFRP fabrics is an efficient and effective way to increase the strength of flexural members in-situ with a very low impact.
REFERENCES


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