EXPERIMENTAL INVESTIGATION OF THE MECHANICAL AND CREEP RUPTURE PROPERTIES OF BASALT FIBER REINFORCED POLYMER (BFRP) BARS

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EXPERIMENTAL INVESTIGATION OF THE MECHANICAL AND CREEP RUPTURE PROPERTIES OF BASALT FIBER REINFORCED POLYMER (BFRP) BARS

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ABSTRACT

A wide range of experimental tests was performed to determine short-term and long-term characteristics of Basalt Fiber Reinforced Polymer (BFRP) bars. Mechanical properties of BFRP bars such as tensile strength, rupture strain and modulus of elasticity were determined from short-term experimental tests. The mechanical properties then were used to calculate guaranteed properties of BFRP bars according to requirements of American and Canadian design codes. Also, a set of durability tests were performed to study creep rupture behavior of BFRP bars and to find creep rupture coefficient to be used for creep design based on the concrete design codes. After knowing the behavior of the bars, they were used as internal reinforcement of concrete beams. The beams were designed according to available design codes applying the properties obtained from primary tests. Therefore, flexural behavior of concrete beams reinforced with BFRP bars were studied experimentally and then were compared with ACI design code and strain compatibility methods. Regarding the fact that deflection dominates the design of FRP reinforced concrete beams, a new equation was derived for BFRP-RC flexural members to be used for deflection prediction of this type of beams, which could be useful adding to design codes. Finally, BFRP bars were used in design of a seawall structure in order to study the feasibility implementation of BFRP reinforced concrete seawall system instead of currently existing steel reinforce concrete seawalls.
DEDICATION

I dedicate my dissertation work to my wonderful family. A special feeling of gratitude to my loving parents Ezatollah Banibayat and Sadigheh Momen whose words of encouragement and push for tenacity rings in my ears. To my beautiful wife, Elham Morovvati, who has changed my life with her pure heart and never stopped encouraging me. Also, to my older brother, Saman Banibayat, who has never left my side and his wise words was always the most relaxing medicine in the toughest times of my life.
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INTRODUCTION

1.1. Introduction

Premature degradation of concrete structures due to corrosion of the embedded steel is a well-known and well-documented problem, particularly in coastal areas and where de-icing salts are routinely used [1.1]. The use of fiber reinforced polymer (FRP) composite materials in severe adverse environment is gaining acceptance because of the non-corrosive nature, higher specific strength, higher specific stiffness, and lower specific weight of FRP materials relative to conventional steel reinforcing bars. In recent years, the development of insight into the performance of FRP materials has been at a rate comparable to similar developments within the iron and steel industry in 19th century [1.2]. The defense and aerospace industries with performance-critical applications, generally adopted FRP materials in the last forty years more readily than the civil engineering construction industry which typically involves cost-sensitive applications.
However, understanding of the performance of FRP composite materials has been adequately developed in the last 20 years so as to consider these materials to be economically viable and structurally acceptable construction material for buildings and bridges in certain special cases [1.3, 1.4]. Such special cases can be applications where dead loads (or self-weight), space or time constraints govern the design [1.5]. FRP materials are also proving to be suitable for substitution of conventional metallic materials in civil engineering applications where corrosion resistance is critical or if the need is based on magnetic properties.

Commercially available FRP composites for structural applications at the present time are made from carbon, glass or aramid fibers. FRP composite bars that are commonly made from these three types of fibers for internal reinforcement of concrete are referred to as CFRP, GFRP and AFRP bars, respectively. While minor variations exist, most manufacturers of FRP bars use the pultrusion process for manufacturing such bars. Guidelines for the design and construction of structural concrete reinforced with FRP bars are specified by ACI Committee 440.1R [1.6]. Unidirectional FRP bars can be utilized similar to steel reinforcing bars as internal reinforcement in concrete structures.

Two developments have occurred in the composites industry in the recent years. Firstly, fiber made from basalt rock is currently available for making basalt FRP bars (hereafter called BFRP bars). Secondly, an automated wet-lay process was successfully used to manufacture FRP bars [1.7] demonstrating that it is somewhat cheaper to produce FRP bars by this new method than by pultrusion method. This study outlines recent research
that was conducted to investigate if the mechanical properties obtained for BFRP bars made using the new process are suitable for reinforced concrete applications.

1.2. Basalt Fiber

Basalt (solidified volcanic lava) is an igneous rock. Fiber can be extruded from molten basalt rock at diameters between 13 to 20 µ in a single stage process. The fiber production process is similar to the process used for the production of glass fiber [1.8]. The primary composition of basalt rock is in the form of various oxides, with silica-oxide being most abundant. The percentage of silica oxide is generally between 51 and 58 percent by mass. Basalt primarily comprises minerals plagioclase, pyroxene and olivine. When heated at high temperature, basalt is capable of producing a natural nucleating agent, which plays a major role in the thermal stability of the material. This leads to apparent increased volumetric integrity of basalt compared to other materials [1.9].

Basalt fiber has excellent resistance to high temperatures, and has high strength and good durability. The technology to extrude fiber from basalt rock was originally developed in the former Soviet Union for defense applications and was classified until recently. Basalt fiber production technology is currently commercialized, and basalt fiber products are available in commercial quantities from various sources around the world. When there is adequate demand for high volume production, basalt fiber is likely to be cost effective, and will certainly be much cheaper than carbon fiber (estimated to be a tenfold difference).
Basalt rock being a natural substance, the fiber made from basalt is environmentally and ecologically harmless. The fiber is free from carcinogens and other health hazards. The properties that are superior in basalt fiber are: good range of thermal performance, -435°F to 1760°F (-260°C to +960°C), high tensile strength (360 ksi or 2500 MPa), modulus of elasticity of about 12,900 ksi (89 GPa), rupture strain of over 3%, high resistance to acids, superior electro-magnetic properties, inertness, resistance to corrosion, resistance to radiation and UV light, and good resistance to vibration. The use of BFRP fabric was successfully demonstrated already for strengthening concrete beams and concrete compression elements in the form of external wraps previously [1.10].

1.3. Automated Wet Lay-up Process

Currently available FRP bars are mostly made using pultrusion process, which is considered to be a low-cost process that results in continuous manufacturing of FRP bars with a constant cross-section. In a pultrusion process, fibers are continuously wetted (impregnated) with the desired resin in a resin bath and are pulled through a die of the desired diameter and suitably cured to harden to the required final shape and diameter. A tight dimensional control of FRP bars is possible by pultrusion process because of the fixed cross-section of dies. This is a relatively simple process for the manufacture of FRP bars particularly for straight lengths. For small diameter FRP bars, coiling is also possible by this method.

Wet lay-up is a common manufacturing process that is used to make simple composite parts. A new automated wet lay-up manufacturing process was recently developed in
Europe to manufacture FRP bars [1.7]. The process essentially consists of laying up the fibers that are impregnated with polymeric resin such that it yields usable composite bars when cured. FRP bars are made using a programmable arm with controlled movement in the three orthogonal directions to manufacture the desired lengths to the required shape. Cost of production of FRP bars by this method is believed to be reduced because the production method is simple and designed to reduce human involvement.

A schematic of the new FRP bar production process is shown in Fig. 1.1. Reels (spools) of fiber are placed on the top fixture that moves with the programmable arm. Depending on the intended diameter of FRP bars, the fiber is drawn from a number of spools to the
required thread count. The fiber is guided through a funnel like resin bath where thorough wetting and impregnation of the fiber take place. The wet fiber is then pulled through another guide closer to a steel working platform. In order to start the wet lay-up, one end of the impregnated fiber is knotted to a steel rod at one corner of the working platform over which the lay-up process is to be carried out. The impregnated fiber is then pulled with a small tension while being spirally wound around the wet fiber bundle at a fixed pitch with a twine (string). The robotic arm is traversed from one end to the other end in parallel patterns to produce a one dimensional FRP mesh. This mesh can be cut to the desired lengths as needed. Continuous closed loops of smaller dimensions can also be manufactured by this process with some minor modifications to the steel fixtures. The arm can be programmed to manufacture closed loops, rectangular spirals, circular spirals or other shapes as needed. A two dimensional mesh can be produced when the arm is traversed perpendicular to the previously laid up mesh as shown in Fig. 1.1. Several layers of loops are laid up one over the other for mass production.

The new process has some advantages and some disadvantages. Once programmed, the process is an automated process. It can produce one dimensional or two dimensional meshes of FRP bars. The cross-sectional shape of wet laid-up FRP bars is not necessarily perfectly uniform unlike pultruded FRP bars. Some fiber waviness can be expected due to non-uniform pull on the fiber threads. The surface of the FRP bars produced by this method can also be uneven because of the lack of a well-defined die to form the required shape. However, this waviness can help in improving pull-out (bond) strength of bars within concrete. The waviness of fiber however is likely to reduce the efficiency in terms
Another advantage is that a two dimensional mesh can be produced with uniform curvature in the third direction. Very large meshes can be made using this production method with the only limitation of dimensions being shipping lengths and practicality. Bar diameters of 0.2 in (5 mm) to 0.6 in (16 mm) were successfully produced using this process. Trials are underway to produce larger diameter bars. Cost effectiveness can be achieved if the production needs are repetitive so that the fixtures and the programs can be used repeatedly without modifications. The BFRP bars included in the current research project were made using the described automated wet lay-up method.

1.4. Serviceability

FRP bars have mostly higher tensile strength and lower elastic modulus compared to steel rebar. Relatively low elastic modulus of FRP bars leads to large deformation of flexural members reinforced with them. Although this behavior does not affect their flexural performance, it severely affects the serviceability criteria. Therefore, deflection prediction, which is a function of flexural stiffness, plays an important role in FRP reinforced concrete flexural members design. So, it is important to have a reliable set of equations to be able to predict deflections of beams under transverse loading. Deflections can be calculated using classic equations for different boundary conditions and loadings. An important factor in deflection equations is moment of inertia, which is a section property. Regardless of material properties, boundary condition and loads, the moment of inertia is constant for a solid section and it could be calculated by a simple equation. On the other hand, a concrete beam should be considered in two stages: before and after
cracking. Before cracking, the section is intact and the moment of inertia of the gross cross-section can be calculated by simple and currently existing equations for gross moment of inertia, $I_g$. While, after cracking the entire concrete section is not fully effective. Therefore, it is important to know the moment of inertia of the section at each load level. The effective moment of inertia $I_{eff}$ provides as a solution for deflection calculations after concrete cracking, which can lead to a good prediction of deflection at each load level. Effective moment of inertia varies with change in internal reinforcement material. It is important to have suitable equations of effective moment of inertia for the new reinforcement BFRP. Therefore, in the current research, studies on the effective moment of inertia is derived for the BFRP reinforced beams.

1.5. Creep-rupture property

Along with mechanical properties of materials, there are some other important long-term properties, which will affect the performance of structures in different situations. One of these properties is known as creep rupture. Creep is defined as plastic deformation under sustained load over an extended period of time. Therefore, in concrete structures it is important to consider the creep for the design. American Concrete Institute ACI design code [1.6] recommends different creep coefficients for different reinforcement materials in creep design. The current research includes several of creep rupture on BFRP reinforcing bars to determine the required creep rupture coefficient for design of BFRP reinforced concrete beams as per ACI 440 [1.6].
1.6. FRP Reinforcement Bars Applications

Vulnerability of steel to moisture and aggressive environments has been causing serious problem for steel reinforced concrete structures, which increases costs for maintenance and repair of decayed structures. Substitution of steel with a noncorrosive material is a feasible solution to increase effective life of structures and decrease maintenance costs. Fiber reinforcement polymers (FRPs) have been widely used as internal reinforcement over the past two decades. A large amount of research has been performed in this project to study the behavior of FRP reinforced concrete beams. A commonly used structure in coastal areas is seawall. Seawall is a type of retaining walls that is designed to stabilize the soil from collapsing into the waterway, and to simply protect the buildings and foundations from moving or overturning. Seawalls interact with soil from one side and interfaces with water from other side. Therefore, it is highly exposed to seawater and regardless of the concrete cover thickness, the internal reinforcing bars would be under attack of the salt from seawater, where the presence of chloride ions expedites the rate of degradation. Because of non-corrosive behavior of the FRP reinforcing bars, applying them in coastal concrete structure can be a good solution to increase the design life of structure.

In this project, a practical application in the form of seawalls is also included for the demonstration project undertaken by another agency in the State of Florida.
1.7. Research Significance

Performance evaluation of FRP bars that are made by using a new process is needed in order to determine the usefulness of such bars for civil engineering applications. Basalt fiber is a new fiber that was used to make FRP bars. Research on the evaluation of BFRP bars is beginning to emerge from several sources. The primary objective in this study is to investigate the mechanical properties of BFRP bars made from the new process. Along with the mechanical properties, time dependent behavior in alkaline environmental of BFRP bars was needed to determine if the characteristics of such bars are suitable for reinforced concrete applications. Studies were also needed to demonstrate the usefulness of BFRP bars in practical applications. None of these studies were conducted at the University of Akron or elsewhere prior to the commencement of this project.

The studies in this project address a new type of fiber (basalt fiber) for manufacturing FRP bars for reinforced concrete applications and a new manufacturing process (automated wet lay-up process) for making the FRP bars for concrete applications. Therefore, the research presented in this report is the first of its kind to develop a sound design basis for BFRP reinforced concrete members, particularly for bars made using the automated wet lay-up method.

1.8. Objectives

The overall objectives of the project are summarized as follows:
• To determine the mechanical properties and the characteristics of BFRP reinforcing bars (short-term) under instantaneous loading. The objective is also to establish the guaranteed mechanical properties based on statistical analysis that can be used in practical designs.

• To study the creep behavior of BFRP reinforcing bars in sustained loading under alkaline environment (long-term) with the creep rupture strength and the creep coefficients primarily for using in practical designs.

• To establish the flexural behavior of concrete beams reinforced with BFRP reinforcing bars including the flexural strength, cracking behavior, and failure modes for BFRP reinforced concrete members.

• To establish a reasonably balanced approach for the prediction of beam deflections of FRP-RC flexural members based on the effective moment of inertia approach.

• To provide a path for implementation of BFRP bars in practical applications through a in-depth study on the behavior of concrete seawall systems that are reinforced with BFRP reinforcing bars.

1.9. Organization of the Dissertation

This dissertation is organized into seven chapters. The problem statement is introduced in Chapter I. Each of the next five subsequent chapters has emphasis on a major research aspect of the project. These chapters are mostly self-contained and stand-alone chapters. Literature review and a discussion of the current state-of-the-art on the chapter theme are presented at the beginning of each chapter. The research methodology, results and
discussion are presented on the chapter theme following the literature review and the discussion of the background for that particular aspect presented in each chapter.

The mechanical properties of BFRP bars manufactured using the new process (automated wet lay-up process) are presented in Chapter II. A thorough statistical analysis and treatise of the test results related to the mechanical properties are also included in this chapter. The guaranteed strengths as needed by design engineers to design reinforced concrete structural members reinforced with BFRP bars are derived based on the methodology devised in this chapter.

Chapter III primarily deals with the flexural behavior, cracking behavior and failure modes of BFPR reinforced concrete beams. Load strain curves and theoretical analysis methods based on several theoretical stress-strain curves of concrete are presented in the chapter.

Extensive analysis and investigation of deflection behavior of BFRP reinforced concrete beams is given in Chapter IV. The effective moment of inertia as applicable to BFRP reinforced concrete beams is derived based on statistical analysis and the relevant recommendations are outlined in this chapter. A proposed equation of the effective moment of inertia of beam sections for the predictions of deflections over the entire loading range of interest is given in the chapter.
The creep rupture strength and creep coefficients are determined and the results of the in-depth investigation are presented in Chapter V. Design values that can be used for different life span of structures are included in this chapter.

A case study of a BFRP practical application is presented in Chapter VI. BFRP reinforced seawalls for applications in Florida were studied and documented in this chapter. The conclusions derived from this project are listed in Chapter VII.
Different types of fiber reinforced polymer (FRP) bars are currently being used as internal reinforcement for concrete members. Recent developments in fiber production technology allow fiber to be made from basalt rock. Basalt fiber has many attractive physical and mechanical properties. A new automated wet lay-up process was used to make basalt fiber reinforced polymer (BFRP) bars for potential use in concrete. In this chapter, the mechanical properties of BFRP bars manufactured by the new process was investigated for their suitability in civil engineering applications. BFRP bars of three sizes were tested using the ACI 440.3R-04 test method.
2.1. Current Methods for the Determination of Mechanical Properties of FRP Bars

The methods for the determination of the mechanical properties such as tensile strength, tensile modulus of elasticity, and ultimate elongation of FRP bars are developed by several national committees (e.g., ACI 440.3R-04 test method B2 [2.1] and CSA S806 Annex C [2.2]).

ACI 440.3R-04 test method B2 is intended for use in laboratory tests where the principal variable is the size or type of FRP bars. Guidance on end anchor details and specimen preparation is given in Appendix A of the document. The tensile strength of a test specimen is derived from the failure load and the average area of cross-section of the FRP bar.

\[
f_u = \frac{F_u}{A}
\]  

(2.1)

where,

\[f_u\] = tensile strength (ksi or MPa)

\[F_u\] = tensile capacity or load at failure (kips or N)

\[A\] = cross-sectional area of specimen (inch\(^2\) or mm\(^2\))

The tensile modulus of elasticity is determined based on the slope of the line drawn by linear regression of stress-strain data points between 20% and 50% of the tensile strength.
Obtaining reliable rupture strains of FRP bars from tests is normally difficult when using extensometers. Therefore, ACI 440.3R-04 suggests that the prediction of ultimate strain of FRP bars be made using the following equation:

\[
(2.2)
\]

where

\( \varepsilon_u \) = the ultimate strain of FRP bar

\( E_L \) = axial (longitudinal) modulus of elasticity (ksi or MPa) obtained based on data points between 20% and 50% of the tensile strength.

Similar guidelines are given in CSA S806 [2.2] for determining the mechanical properties of FRP bars.

2.1.1. Analysis of Test Data to Determine Guaranteed Properties

Once the test results are developed, determination of suitable guaranteed properties is based on statistical interpretation of the test data. Equations are given in ACI 440.1R-06 [1.6] to determine guaranteed tensile strength, \( f^{*}_{fu} \), guaranteed rupture strain, \( \varepsilon^{*}_{fu} \), and a guaranteed tensile modulus, \( E_f \) for design purposes. If \( \sigma \) is the standard deviation of the relevant mechanical property, \( f_{u,ave} \) is the mean tensile strength of test specimens, \( \varepsilon_{u,ave} \) is the mean tensile strain at failure of test specimens, and \( E_{f,ave} \) is the mean elastic modulus,
These equations are based on the assumption that the frequency distribution of strengths or strains of the test specimens satisfies normal (Gaussian) distribution. ACI 440.1R-06 approach to the determination of guaranteed values of strengths and strains also assumes a 99.87% probability that the strengths or strains are exceeded by similar FRP bars, provided 25 specimens are tested. The guaranteed strength or strain is to be derived based on statistical analysis if fewer test specimens are tested, or the distribution is not a normal distribution.

Based on Canadian Standard CSA S807-10 [2.3], the guaranteed properties (notation retained from ACI equations) are calculated using the following equations:

\[ (2.6) \]

where,

\[ (2.7) \]
In Eq. (2.7), \( V \) is the coefficient of variation (COV) of the tensile strength (or the modulus in Eq. (2.9)) obtained from tests. COV is the ratio of standard deviation to the mean of test results. The number of successful tests is \( n \).

Similarly, for the modulus of elasticity, the specified value needs to be based on the mean value obtained from tests multiplied by \( F_{e,csa} \) as follows:

\[
(2.8)
\]

where,

\[
(2.9)
\]

The guaranteed tensile strength, strain and modulus obtained from ACI 440.1R-06 are different from the corresponding specified values obtained by using CSA S807-10 equations due to the different philosophies adopted by the two approaches.

Many times, it is cost prohibitive to test at least 25 specimens to establish the normal distribution of test data and to use Eqs. (2.3) to (2.5). Equations (2.6) and (2.8) are more useful if the number of data points is less than 25, as long as it can be established that the data points are normally distributed.

To compare the two approaches, Eqs. (2.3) and (2.8) may be written as follows:
Equations (2.10) and (2.11) may be compared to understand the relative values of guaranteed values by the two approaches. These equations are plotted in Fig. 2.1 and Fig. 2.2. In both figures, the ratio of guaranteed value to mean value is shown on the vertical axis. In Fig. 2.1, Eq. (2.10) and Eq. (2.11) are shown with an assumed number of test specimens of 25. ACI 440.1R equation (Eq. 2.10) varies linearly with the coefficient of variation (V) regardless of number of specimens. The CSA S807 equation (Eq. 2.11) varies linearly with V if the number of specimens is constant. The figure demonstrates that ACI 440.1R equation grossly underutilizes test values compared to CSA S807 approach in determining guaranteed values.
Fig. 2.1 Comparison of Equations for Guaranteed Values based on ACI 440.1R and CSA S807 with Number of Samples $n = 25$

Fig. 2.2 Variation of Guaranteed Values based on CSA S807 with Number of Samples and Comparison with ACI 440.1R
The influence of the number of specimens on the guaranteed test values using the CSA S807 equation is shown in Fig. 2.2. Six different curves are shown corresponding to six values of $V$ ranging from 0.01 to 0.06. The corresponding ACI 440.1R guaranteed values are shown corresponding to each curve (in bold lines) only for number of specimens varying from zero to two. The two pairs of lines intersect in all the cases shown in the figure for the number of specimens less than 2. The figure shows that the CSA S807 approach allows guaranteed test values to be about the same as the corresponding ACI 440.1R values for the number of test specimens less than two. The curves become mostly flat for larger number of test specimens. Much larger test values are allowed by CSA S807 compared to ACI 440.1R for practical range of the number of test specimens (3 to 6).

If the number of test data points for each set of test results is less than 25, it becomes necessary to use statistical methods to obtain minimum guaranteed test values. Some researchers used ANOVA to determine differences between different types of FRP bars in the past [e.g., 2.4]. Data from different populations can be combined if the validity of this combination can be verified by applying the analysis of variance (ANOVA) techniques or $t$-test [2.5]. By these methods, it is required to ensure that the means of individual populations are not significantly different. For both techniques, normal distribution of the data points is required. When the number of groups is more than two, $t$-test increases the chance of occurrence of type I error while ANOVA provides a better result [2.5]. When combining data from different populations, if F-value is smaller than
F-critical meaning that there is no significant difference between mean tensile strength of three groups for 95% significance level ($\alpha = 0.05$). However, a statistical analysis is needed to verify if the test data follows a normal distribution (Gaussian model).

2.1.2. Kolmogorov-Smirnov (K-S) Test to Verify Normal Distribution of Tests Data

A statistical test called Kolmogorov-Smirnov test (K-S test) is performed to verify if the test data is normally distributed. Kolmogorov-Smirnov is a nonparametric test used to compare probability distributions. This test may be applied as a goodness-of-fit test. To check the normality of a distribution, K-S test compares the frequency distribution of the population with a standard normal distribution which has been generated using the given mean and variance of the data [2.6].

2.1.3. Minimum Number of Test Specimens Needed for Analysis

When performing data analysis with fewer test specimens than 25, a sound statistical approach needs to be used. The number of test specimens required for obtaining mean tensile strength of test specimens with various significance levels and maximum error can be specified using the following equation [2.4]:

$$m = \text{number of required test specimens}$$
\( e \) = desired precision (± maximum acceptable error)

\( Z_p \) = the abscissa of the normal curve that cuts off an area at the tails, \( Z_p \) values are provided in Table 2.1.

\( \sigma^2 \) = variance

As an example, to estimate mean tensile strength of BFRP bars with standard deviation of 11.3 ksi (78 MPa), a confidence level of 90% and desired precision of ±5 ksi (34.5 MPa), at least 9 test specimens are required. To satisfy 95% of confidence level at the same error range, a sample size of 14 is needed. A summary of minimum required number of tests for various confidence levels and marginal error is provided in Table 2.2.

<table>
<thead>
<tr>
<th>Confidence Level</th>
<th>( Z_p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>90%</td>
<td>1.285</td>
</tr>
<tr>
<td>95%</td>
<td>1.645</td>
</tr>
<tr>
<td>99%</td>
<td>2.325</td>
</tr>
<tr>
<td>99.875%</td>
<td>3.000</td>
</tr>
</tbody>
</table>

Table 2. 2. Minimum required number of tests for different confidence levels

<table>
<thead>
<tr>
<th>e (ksi)</th>
<th>90</th>
<th>95</th>
<th>99</th>
<th>99.875</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>211</td>
<td>9</td>
<td>346</td>
<td>891</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td>14</td>
<td>28</td>
</tr>
</tbody>
</table>

2.2. Experimental Procedure

BFRP bars that were produced using the new process were tested in this study to establish the tensile strength, modulus of elasticity, rupture strain, and the variability of
these properties. Minimum guaranteed tensile strength, rupture strain and modulus of elasticity for three different bar diameters were determined.

2.2.1. Materials

BFRP bars of three different diameters were produced using basalt fibers and vinyl ester matrix. Vinyl ester resin is a combination of an epoxy and an unsaturated polyester resin [1,4]. Advantage of vinyl ester is that it has the good physical properties of an epoxy and the beneficial processing properties of a polyester resin. Fiber volume fraction was approximately 50% for all bar sizes.

The bar diameters tested in this study are designated as R4, R7 and R10 which relate to the respective diameter of the gross section. The corresponding net fiber diameters of the three sizes of bars are 3, 5 and 7 millimeter respectively. A summary of BFRP bar properties such as the diameters and cross-sectional areas of the three bar sizes is provided in Table 2.3.

Table 2.3. Summary of Geometric Properties of BFRP Bars

<table>
<thead>
<tr>
<th>Bar</th>
<th>No of Tows</th>
<th>Nominal Diameter</th>
<th>Nominal Area (Gross Area)</th>
<th>Net Fiber Area</th>
<th>Volume Fraction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>mm</td>
<td>inch</td>
<td>mm²</td>
<td>inch²</td>
<td>mm²</td>
</tr>
<tr>
<td>R4</td>
<td>4</td>
<td>4.3</td>
<td>0.167</td>
<td>14.5</td>
<td>0.0225</td>
</tr>
<tr>
<td>R7</td>
<td>11</td>
<td>7.1</td>
<td>0.279</td>
<td>39.7</td>
<td>0.0615</td>
</tr>
<tr>
<td>R10</td>
<td>21</td>
<td>9.8</td>
<td>0.386</td>
<td>75.6</td>
<td>0.1172</td>
</tr>
</tbody>
</table>
2.2.2. Test Specimen Preparation

FRP bars are weak against compression in a direction transverse to the fiber direction, which causes them to crush under lateral pressure force within the grip of standard testing machines. This is caused due to the weakness of the matrix in compression. Therefore, FRP bars cannot be mounted directly within the grips of a testing machine. A simple anchorage system made of steel tube filled with a mix of non-shrinkable epoxy and dried sand was used to support bars from crushing within the grips of the testing machine [2.1]. The length of end anchors for the test specimens in this study was based on bar sizes in the project in order to provide effective interlock between bar and epoxy. The dimensional details and the cross sectional details of the tensile test specimens are shown in Fig. 2.3. The left half of the figure shows the elevation and the right half shows the section of a typical specimen. Steel caps were fixed to the inner ends of steel anchor tubes. A central hole was drilled into the caps to allow for the bar specimen to pass through. Rubber caps were used at both far ends. Combination of steel and rubber caps at the two ends of each end anchor was used to align the bars to be concentric with the end anchors. The applied epoxy was a two part hard set epoxy formulated for general purpose use. Mixing ratio of epoxy parts was one to one by volume. To increase interlock behavior and decrease shrinkage of epoxy, dry sand of up to 20% by volume was added to the epoxy mix. The epoxy was allowed to cure for a minimum twenty four hours at ambient room temperature.
ACI 440.3R-03 recommends that the free length of a bar specimen be greater than or equal to $40d_b$ where $d_b$ is nominal bar diameter of the gross section [2.1]. A free length of 12 inch was selected for the three bar sizes to meet this requirement. Two different anchorage lengths, 12 inch (305 mm) for R4 and R7 bars, and 18 inch (457 mm) for R10 were used to prevent de-bonding or failure by slippage. To improve bond between the epoxy and internal surface of the steel tube of the anchors, the tube length was increased, and roughened on the inside surface of the tubes so that the anchors could hold BFRP bars without any slippage. Uni-axial tensile tests were performed on the three sizes of BFRP bars. In total, eleven tests were performed on R4 bars, ten on R7 bars, and fourteen on R10 bars.
2.2.3. Tension Test Set-Up

The tensile tests were performed in a universal testing machine with 300 kip (1,335 kN) capacity. Typical setup for tensile tests is shown in Fig. 2.4. The end anchors of the test specimens were held in serrated V-grips seated in the crosshead and the loading end. The load was applied at a constant rate so as to complete each test within 1 to 10 minutes as specified by ACI 440.3R-04 [2.1]. Strains were measured with an extensometer with 2 inch (50 mm) gage length that was attached to the center of free length of the test specimen. The ultimate tensile failure load was recorded from a data acquisition system. The load-strain data were also recorded in the data acquisition system.
2.3. Test Results and Data Analysis

The test results are tabulated in Tables 2.4 to 2.6 for the three diameters of BFRP bars tested in this project. Results are based on gross diameter of the bars. The modulus of elasticity for each specimen was determined based on the slope of the straight line drawn
by regression of points between stresses of 20% and 50% of the tensile strength of that particular specimen [2.1].

Table 2.4. Tensile Test Results R4 BFRP Bars

<table>
<thead>
<tr>
<th>Test #</th>
<th>Max Load</th>
<th>Strain (Measured)</th>
<th>Stress</th>
<th>Modulus</th>
<th>Strain (Calculated)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>lb</td>
<td>in/in</td>
<td>ksi</td>
<td>ksi</td>
<td>in/in</td>
</tr>
<tr>
<td>1-R4</td>
<td>3,636</td>
<td>0.02417</td>
<td>161.8</td>
<td>7,445</td>
<td>0.02173</td>
</tr>
<tr>
<td>2-R4</td>
<td>3,734</td>
<td>0.03057</td>
<td>166.1</td>
<td>6,025</td>
<td>0.02758</td>
</tr>
<tr>
<td>3-R4</td>
<td>3,615</td>
<td>0.0283</td>
<td>160.8</td>
<td>6,080</td>
<td>0.02645</td>
</tr>
<tr>
<td>4-R4</td>
<td>3,056</td>
<td>0.02201*</td>
<td>136*</td>
<td>7147*</td>
<td>-</td>
</tr>
<tr>
<td>5-R4</td>
<td>3,511</td>
<td>0.0269</td>
<td>156.2</td>
<td>7,997</td>
<td>0.01953</td>
</tr>
<tr>
<td>6-R4</td>
<td>4,037</td>
<td>0.0358</td>
<td>179.6</td>
<td>6,859</td>
<td>0.02619</td>
</tr>
<tr>
<td>7-R4</td>
<td>3,847</td>
<td>0.03108</td>
<td>171.2</td>
<td>5,903</td>
<td>0.02900</td>
</tr>
<tr>
<td>8-R4</td>
<td>3,516</td>
<td>0.03254</td>
<td>156.4</td>
<td>5,795</td>
<td>0.02700</td>
</tr>
<tr>
<td>9-R4</td>
<td>4,092</td>
<td>0.029</td>
<td>182.1</td>
<td>6,155</td>
<td>0.02958</td>
</tr>
<tr>
<td>10-R4</td>
<td>3,552</td>
<td>0.028</td>
<td>158</td>
<td>6,231</td>
<td>0.02536</td>
</tr>
<tr>
<td>11-R4</td>
<td>3,892</td>
<td>0.029</td>
<td>173.2</td>
<td>6,387</td>
<td>0.02711</td>
</tr>
<tr>
<td><strong>Mean</strong></td>
<td><strong>3,681</strong></td>
<td><strong>0.0295</strong></td>
<td><strong>166.5</strong></td>
<td><strong>6,488</strong></td>
<td><strong>0.0260</strong></td>
</tr>
<tr>
<td>Std Dev</td>
<td>290</td>
<td>0.00320</td>
<td>9.5</td>
<td>724</td>
<td>0.00311</td>
</tr>
</tbody>
</table>

Note: *Outlier, not included in the calculation of the mean value
1 lb=4.45 N
1 ksi=6.89 MPa
Table 2. 5. Tensile Test Results for R7 BFRP Bars

<table>
<thead>
<tr>
<th>Test #</th>
<th>Max Load</th>
<th>Strain (Measured)</th>
<th>Stress</th>
<th>Modulus</th>
<th>Strain (Calculated)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>lb</td>
<td>in/in</td>
<td>ksi</td>
<td>ksi</td>
<td>in/in</td>
</tr>
<tr>
<td>1-R7</td>
<td>10,132</td>
<td>0.02725</td>
<td>164.7</td>
<td>6,045</td>
<td>0.02724</td>
</tr>
<tr>
<td>2-R7</td>
<td>10,805</td>
<td>0.03154</td>
<td>175.6</td>
<td>5,830</td>
<td>0.03012</td>
</tr>
<tr>
<td>3-R7</td>
<td>8,764</td>
<td>0.02635</td>
<td>142.4</td>
<td>5,981</td>
<td>0.02381</td>
</tr>
<tr>
<td>4-R7</td>
<td>9,064</td>
<td>0.02579</td>
<td>147.3</td>
<td>5,860</td>
<td>0.02514</td>
</tr>
<tr>
<td>5-R7</td>
<td>9,462</td>
<td>0.0263</td>
<td>148.9</td>
<td>5,875</td>
<td>0.02617</td>
</tr>
<tr>
<td>6-R7</td>
<td>10,208</td>
<td>0.02725</td>
<td>153.8</td>
<td>5,772</td>
<td>0.02874</td>
</tr>
<tr>
<td>7-R7</td>
<td>9,819</td>
<td>0.03097</td>
<td>165.9</td>
<td>7,114</td>
<td>0.02243</td>
</tr>
<tr>
<td>8-R7</td>
<td>9,815</td>
<td>0.02577</td>
<td>159.6</td>
<td>6,682</td>
<td>0.02387</td>
</tr>
<tr>
<td>9-R7</td>
<td>10,421</td>
<td>0.02369</td>
<td>159.5</td>
<td>6,518</td>
<td>0.02598</td>
</tr>
<tr>
<td>10-R7</td>
<td>10,421</td>
<td>-</td>
<td>169.4</td>
<td>6,261</td>
<td>0.02705</td>
</tr>
<tr>
<td><strong>Mean</strong></td>
<td><strong>9,891</strong></td>
<td><strong>0.0272</strong></td>
<td><strong>158.7</strong></td>
<td><strong>6,194</strong></td>
<td><strong>0.0261</strong></td>
</tr>
<tr>
<td><strong>Std Dev</strong></td>
<td>642</td>
<td>0.00252</td>
<td>10.6</td>
<td>445</td>
<td>0.00235</td>
</tr>
</tbody>
</table>

Note:*Outlier, not included in the calculation of the mean value

1 lb=4.45 N
1 ksi=6.89 MPa
Table 2.6. Tensile Test Results for R10 BFRP Bars

<table>
<thead>
<tr>
<th>Test #</th>
<th>Max Load, lb</th>
<th>Strain, in/in (Measured)</th>
<th>Stress, Ksi</th>
<th>Modulus, Ksi</th>
<th>Strain, in/in (Calculated)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-R10</td>
<td>21,380</td>
<td>0.02991</td>
<td>182.5</td>
<td>6,587</td>
<td>0.02770</td>
</tr>
<tr>
<td>2-R10</td>
<td>19,974</td>
<td>0.02867</td>
<td>170.5</td>
<td>6,629</td>
<td>0.02571</td>
</tr>
<tr>
<td>3-R10</td>
<td>17,542</td>
<td>0.02424</td>
<td>149.7</td>
<td>6,544</td>
<td>0.02288</td>
</tr>
<tr>
<td>4-R10</td>
<td>18,145</td>
<td>0.01801</td>
<td>154.8</td>
<td>6,814</td>
<td>0.02272</td>
</tr>
<tr>
<td>5-R10</td>
<td>17,767</td>
<td>#</td>
<td>151.6</td>
<td>6,207</td>
<td>0.02443</td>
</tr>
<tr>
<td>6-R10</td>
<td>18,194</td>
<td>#</td>
<td>155.3</td>
<td>6,162</td>
<td>0.02520</td>
</tr>
<tr>
<td>7-R10</td>
<td>16,689</td>
<td>0.02351</td>
<td>142.4</td>
<td>6,715</td>
<td>0.02121</td>
</tr>
<tr>
<td>8-R10</td>
<td>17,928</td>
<td>0.02701</td>
<td>153</td>
<td>6,740</td>
<td>0.02270</td>
</tr>
<tr>
<td>9-R10</td>
<td>16,968</td>
<td>0.02372</td>
<td>144.8</td>
<td>6,435</td>
<td>0.02250</td>
</tr>
<tr>
<td>10-R10</td>
<td>17,031</td>
<td>0.02417</td>
<td>145.3</td>
<td>6,450</td>
<td>0.02253</td>
</tr>
<tr>
<td>11-R10</td>
<td>18,300</td>
<td>0.02311</td>
<td>156.2</td>
<td>6,686</td>
<td>0.02336</td>
</tr>
<tr>
<td>12-R10</td>
<td>17,807</td>
<td>0.02365</td>
<td>152</td>
<td>6,400</td>
<td>0.02374</td>
</tr>
<tr>
<td>13-R10</td>
<td>20,464</td>
<td>0.0255</td>
<td>174.6</td>
<td>6,317</td>
<td>0.02765</td>
</tr>
<tr>
<td>14-R10</td>
<td>18,677</td>
<td>0.02236</td>
<td>159.4</td>
<td>6,149</td>
<td>0.02592</td>
</tr>
</tbody>
</table>

**Mean**

<table>
<thead>
<tr>
<th>Max Load, lb</th>
<th>Strain, in/in</th>
<th>Stress, Ksi</th>
<th>Modulus, Ksi</th>
<th>Strain, in/in</th>
</tr>
</thead>
<tbody>
<tr>
<td>18,348</td>
<td>0.0250</td>
<td>156.6</td>
<td>6,488</td>
<td>0.0242</td>
</tr>
</tbody>
</table>

**Std Dev**

<table>
<thead>
<tr>
<th>Max Load, lb</th>
<th>Strain, in/in</th>
<th>Stress, Ksi</th>
<th>Modulus, Ksi</th>
<th>Strain, in/in</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,368</td>
<td>0.00309</td>
<td>11.7</td>
<td>221</td>
<td>0.00200</td>
</tr>
</tbody>
</table>

Note: *Outlier, not included in the calculation of the mean value
#Could not record
1 lb=4.45 N
1 ksi=6.89 MPa

The R4 series test specimens failed in tension except specimen number 4-R4 which failed prematurely by splitting at one end. The test data for specimen 4-R4 was considered as an outlier. For R4 BFRP bars, test results indicate the average tensile strength of 167 ksi (1,148 MPa), the average rupture strain of 0.0295 as recorded from the extensometer, and the modulus of elasticity of 6,488 ksi (44.7 GPa). The average rupture strain based on the ACI 440.3R-04 recommended approach (Eq. 2.2) was 0.026 as shown in Table 2.4. There is a minor difference in the measured average strain at failure and the strain
calculated from Eq. 2.2. A typical stress-strain diagram for R4 bars is shown in Fig. 2.5. The stress-strain curves obtained from the tests were very linear in the range of 20% to 50% after minor non-linearity in the initial part of the curves.

![Stress-Strain Curve for R4 BFRP Bar](image)

**Fig. 2.5 Typical Stress Strain Curve for R4 BFRP Bar**

For R7 BFRP bars, the test results are shown in Table 2.5. In this case, although the testing process was the same as for R4, results show lower tensile strength and rupture strain in comparison to R4 bars. The average tensile strength obtained was 159 ksi (1,094 MPa), the average rupture strain was 0.0272 based on extensometer reading (0.0261 from Eq. 2), and the modulus of elasticity is 6,194 ksi (42.7 GPa). A typical stress-strain curve
for R7 bars is shown in Fig. 2.6 which also demonstrates very good linearity in the range of interest.

![Typical Stress Strain Curve for R7 BFRP Bar](image)

For R10 BFRP bars, there were some specimens that prematurely failed due to slippage in the end anchors. Therefore, a larger number of tests (14 tests) were conducted to increase reliability of test results. By increasing the length of ends anchors, the problem was partly overcome but the failure mode of some R10 bars was still not a perfectly tensile failure. Most of the R10 test specimens failed by splintering at the center of the free length, while the others failed at one of the ends at the anchors by slippage, close to
metal cap or inside it (Fig. 2.8). The tensile strengths and rupture strains were lower than the corresponding values of the companion smaller size bars. These test results as summarized in Table 2.6 are therefore to be considered lower bound strengths. The results show that the average lower bound tensile strength was 157 ksi (1,080 MPa), the average rupture strain of 0.0245 as recorded with the extensometer (0.0242 from Eq. 2.2), and the modulus of elasticity is 6,488 ksi (44.7 GPa). A typical stress-strain diagram for R10 bars is shown in Fig. 2.7. The linearity of the curve is still maintained within the range of interest regardless of the mode of failure.

Larger diameter bars exhibited lower tensile strengths compared to those of smaller diameter bars, which is what other researchers observed in their tests as well. This was explained by shear-lag effect that is common for polymer matrix composites and is well documented elsewhere [1.6] for GFRP and CFRP bars.
Fig. 2. Typical Stress Strain Curve for R10 BFRP Bar

R10-BFRP Bar

Stress-Strain Curve
modulus line

Tensile Modulus = 6,400 ksi (44.1 GPa)
Tensile Strength = 152.0 ksi (1,048 MPa)
Fig. 2. 8 Failure Modes
The mean values and standard deviations of the tensile strengths, rupture strains and modulus values for each bar size are also shown in Tables 2.4, 2.5 and 2.6. These values were applied to determine the minimum guaranteed mechanical properties of the three sizes of BFRP bars based on ACI 440.1R-06 (Eqs. 2.3-2.5) and CSA S807 (Eqs. 2.6-2.9) recommendations. The guaranteed values are summarized in Table 2.7. However, the test data need to be verified to be normally distributed. Therefore, statistical analysis was done to verify if the test data frequency distribution was normal.

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Guaranteed Strain (in/in)</th>
<th>Guaranteed Stress (ksi)</th>
<th>Guaranteed Modulus (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>ACI</td>
<td>CSA</td>
</tr>
<tr>
<td>R4</td>
<td>0.01865</td>
<td>137.9</td>
<td>146.5</td>
</tr>
<tr>
<td>R7</td>
<td>0.01971</td>
<td>126.9</td>
<td>136.6</td>
</tr>
<tr>
<td>R10</td>
<td>0.0152</td>
<td>121.5</td>
<td>133</td>
</tr>
</tbody>
</table>

2.4. Verification of the Test Data for Normal Distribution

To determine that a probability distribution of data is normal, one of the statistical methods needs to be used. Histogram is one of the visual methods to determine if a probability distribution is normal. However, histogram is useful when number of samples is large. In case of midsize and small size samples, a more reliable method is probability plot, which is a graphical approach. Normal probability plot will show if a population is normally distributed. If the plotted points are falling along a line (evenly on either side of
the line), the data are normally distributed. If the points are distributed with large fluctuations, then the data cannot be described as having normal distribution. This visual test is very subjective and may result in unreliable interpretation. Therefore, the K-S test was also performed to check normality of the population. A large sample size is always preferable to determine a statistical distribution for a data set. In this study, limited number of tests (less than 25) was done for each bar size. Therefore, the test results from three bar sizes were merged into one group to analyze a set of 34 data points in total. The validation of this combination was verified using analysis of variance (ANOVA) techniques. This analysis demonstrated that the F-value is smaller than F-critical (Table 2.8) meaning that there is no significant difference between mean tensile strength of three groups for 95% significance level ($\alpha = 0.05$).

<table>
<thead>
<tr>
<th>Source of Variation</th>
<th>Degree of freedom</th>
<th>Sum of squares</th>
<th>Mean square</th>
<th>F-value</th>
<th>P-value</th>
<th>F-critical</th>
</tr>
</thead>
<tbody>
<tr>
<td>Between Groups</td>
<td>2</td>
<td>28885</td>
<td>14442</td>
<td>2.62</td>
<td>0.0891</td>
<td>3.31</td>
</tr>
<tr>
<td>Within Groups</td>
<td>31</td>
<td>171058</td>
<td>5518</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>33</td>
<td>199942</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Furthermore, the results from K-S test of the test data developed in this project showed a $P$-value of 0.791 which is greater than 0.05. Therefore, the assumption of normal distribution of this data for the limited sample size is considered valid. From a total 34 tests, an overall mean tensile strength of 160 ksi (1,104 MPa) was obtained with standard deviation of 11.3 ksi (78 MPa). The frequency diagram of tensile strength is plotted in Fig. 2.9. Normal probability density function (pdf) corresponding to the mean and the standard deviation of the dataset is also shown in the same figure. From the plot, the
Gaussian behavior of the test data is verified. Mean and standard deviation were used from Tables 4, 5 and 6 to plot the corresponding pdf curves in Fig. 2.10 for tensile strength, and Fig. 2.11 for modulus of elasticity.

Probability plots for three bar sizes are plotted for tensile strength are shown in Fig. 2.10. The data points (particularly for R7) are well distributed on both sides of the straight lines drawn using the test data indicating normal distribution. Similar graphical representation of test data for modulus of elasticity in Fig. 2.11 indicates reasonable normal distribution. However, this approach is not reliable.
Fig. 2. 10  Tensile Strength Probability Plot (up). Frequency Diagram (down)
Fig. 2.11 Tensile Strength Probability Plot (up). Frequency Diagram (down) (Continued)

\[ y = 0.0927x - 14.716 \]
Fig. 2.11 Tensile Strength Probability Plot (up). Frequency Diagram (down) (Continued)
Fig. 2. 11 Modulus of Elasticity Probability Plot (up). Frequency Diagram (down)
Fig. 2.12 Modulus of Elasticity Probability Plot (up). Frequency Diagram (down) (Continued)

\[ y = 0.0021x - 12.804 \]

Normalized Frequency Curve - Modulus of R7 Bars

Modulus of Elasticity, ksi

Normalized Frequency

Frequency

Generated
Fig. 2.12 Modulus of Elasticity Probability Plot (up). Frequency Diagram (down) (Continued)
2.5. Discussion

The tensile strength, strain at rupture, and the modulus of elasticity of BFRP reinforcing bars are established in this section for all the three sizes of bars. For R10 bars, the failure of the bars occurred by slipping or de-bonding at the anchors for some of the test specimens. Therefore, the strengths determined for R10 bars may only be considered as a lower bound. The stress-strain behavior of BFRP bars was observed to be linear up to the point of rupture, which is in agreement with the work by previous researchers [2.5].

The shear lag effect arises because all the fibers will not be stressed equally when axial load is applied. The outer fibers will be stressed more than the core fibers thus decreasing the overall load carrying capacity of the FRP bar. Therefore, in the case of FRP bars, the strength decreases as the size increases and hence the tensile strength should always be specified for the particular size of the bar [2. 2]. The shear-lag phenomenon is associated with the curing related problem of pultruded FRP bars [2.7]. The BFRP bars tested in this study also showed the shear lag effect. Therefore, there appears to be shear lag effect of FRP bars made by automated wet layup process as well.

2.6. Summary

Based on the test results developed in this study for basalt FRP bars, and the corresponding statistical analysis of test data, the following conclusions are drawn:
1. BFRP bars manufactured by an automated wet lay-up process and investigated in this study had average tensile strengths ranging from 167 ksi for smaller diameter bars to 157 ksi for larger diameter bars. The modulus of elasticity of BFRP bars was found to be 6200 ksi to 6500 ksi. The rupture strain was found to be over 0.025 inch per inch.

2. The test results from a data set smaller than ACI 440.1R-06 required 25 specimens were found to be normally distributed based on K-S test Gaussian (normal) distribution for the BFRP bars manufactured using an automated wet lay-up process and tested in this study.

3. The described automated wet lay-up process for the manufacture of BFPR bars was found to be a satisfactory method producing BFRP bars that have an acceptable normal distribution of test results.
2.7. Notation

\[ A = \text{cross-sectional area of specimen (mm}^2\text{ or inch}^2) \]

\[ e = \text{desired precision (± maximum acceptable error)} \]

\[ \varepsilon_u = \text{the ultimate strain of FRP bar} \]

\[ E_f = \text{guaranteed modulus of elasticity} \]

\[ E_{f,ave} = \text{average modulus of elasticity} \]

\[ E_L = \text{axial (longitudinal) modulus of elasticity, MPa} \]

\[ F = \text{reduction factor for guaranteed tensile strength and modulus of elasticity} \]

\[ f'_{fu} = \text{guaranteed tensile strength} \]

\[ f_u = \text{tensile strength (MPa or ksi)} \]

\[ F_u = \text{tensile capacity or load at failure (N or kips)} \]

\[ f_{u,ave} = \text{average tensile strength} \]

\[ m = \text{number of required test specimen} \]

\[ n = \text{number of samples} \]

\[ V = \text{coefficient of variance (COV)} \]

\[ V_f = \text{volume fraction} \]

\[ Z_p = \text{the abscissa of the normal curve that cuts off an area at the tails} \]

\[ \varepsilon_{u,ave} = \text{average rupture strain} \]

\[ \varepsilon^{*}_{fu} = \text{guaranteed rupture strain} \]

\[ \sigma = \text{standard deviation} \]
3.1. Introduction

This part of the research program comprises studies on the performance evaluation of concrete beams reinforced with BFRP bars. Concrete reinforcing bars made from Carbon, Glass and Aramid fibers have been on market for at least 15 years. There are also several recommended design guidelines available including those compiled by ACI 440.1 committee in the United States entitled, “Guidelines for the design and construction of structural concrete reinforced with FRP bars – ACI 440.1R-06” [3.1]. BFRP bars are used as internal reinforcement in concrete structures. Therefore, the primary objective of this research program in this chapter is to evaluate the structural performance of concrete beams reinforced with BFRP reinforcing bars. On the basis of the research conducted by A. Nanni for the comparative study of the flexural behavior of the Aramid FRP reinforced
beams and conventional steel reinforced beams, some important observations can be made. Based on the moment-curvature analysis, it was revealed that the FRP-reinforced section exhibits the same maximum moment and curvature as in the case of the counterpart steel reinforced beam with a slightly smaller reinforcement ratio, however, the flexural rigidity of the FRP section is only 38% of the steel reinforced beam. This will lead us to the fact that deflection criteria may be as important as flexural strength in the case of FRP reinforced beams [3.2]. Since deflection varies inversely with the flexural stiffness of the element, even an over-reinforced FRP beam may exhibit considerable deformation under service load condition [3.3]. As deflection is deemed to be a governing criterion for the serviceability limit state of FRP reinforced concrete beams, this issue was also seriously investigated during the research.

A test program was designed to systematically study the flexural performance of concrete beams reinforced with BFRP reinforcing bars. This chapter outlines the test program, test results and the conclusions drawn from the tests. Fifteen test beams (beams B1 to B15) were made. Test beams B1 to B13 were reinforced with BFRP bars. The remaining two beams (B14 and B15) were reinforced with steel reinforcement. The beams were designed to include a range of areas of BFRP bar reinforcement varying from 0.045 inch² (29 mm²) to 0.3516 inch² (227 mm²) as shown in Table 3.1. The table also shows balanced BFRP reinforcement ratio when the concrete strength, the BFRP bar strength and the modulus of elasticity corresponding to each beam in this test program are used.
### Table 3.1. Summary of Test Beams

<table>
<thead>
<tr>
<th>Sample no.</th>
<th>Reinforcement</th>
<th>b, in</th>
<th>d, in</th>
<th>ρ/λb</th>
<th>UR/OR</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Group 1 f’c=4.90 ksi; E_c=3,990 ksi</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B1</td>
<td>2 R4</td>
<td>8</td>
<td>6</td>
<td>0.36</td>
<td>UR</td>
</tr>
<tr>
<td>B2</td>
<td>2 R7</td>
<td>8</td>
<td>6</td>
<td>0.99</td>
<td>UR</td>
</tr>
<tr>
<td>B3</td>
<td>2 R10</td>
<td>8</td>
<td>6</td>
<td>1.88</td>
<td>OR</td>
</tr>
<tr>
<td><strong>Group 2 f’c=3.70 ksi; E_c=3,470 ksi</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B4</td>
<td>3 R4</td>
<td>8</td>
<td>6</td>
<td>0.64</td>
<td>UR</td>
</tr>
<tr>
<td>B5</td>
<td>2 R4 + 1 R7</td>
<td>8</td>
<td>6</td>
<td>1.01</td>
<td>OR</td>
</tr>
<tr>
<td>B6</td>
<td>2 R4 + 1 R10</td>
<td>8</td>
<td>6</td>
<td>1.54</td>
<td>OR</td>
</tr>
<tr>
<td>B7</td>
<td>1 R4 + 2 R7</td>
<td>8</td>
<td>6</td>
<td>1.38</td>
<td>OR</td>
</tr>
<tr>
<td>B8</td>
<td>3 R7</td>
<td>8</td>
<td>6</td>
<td>1.75</td>
<td>OR</td>
</tr>
<tr>
<td>B9</td>
<td>3 R10</td>
<td>8</td>
<td>6</td>
<td>3.33</td>
<td>OR</td>
</tr>
<tr>
<td><strong>Group 3 f’c=5.90 ksi; E_c=4,380 ksi</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B10</td>
<td>3 - R4</td>
<td>8</td>
<td>6</td>
<td>0.54</td>
<td>UR</td>
</tr>
<tr>
<td>B11</td>
<td>2 R4 + 1 R7</td>
<td>8</td>
<td>6</td>
<td>0.85</td>
<td>UR</td>
</tr>
<tr>
<td>B12</td>
<td>2 R7</td>
<td>8</td>
<td>6</td>
<td>0.99</td>
<td>UR</td>
</tr>
<tr>
<td>B13</td>
<td>2 R7 + 1 R4</td>
<td>8</td>
<td>6</td>
<td>1.17</td>
<td>OR</td>
</tr>
</tbody>
</table>

Note: 1 in=25.4 mm; 1 ksi=6.9 Mpa

UR: Under-Reinforced
OR: Over-Reinforced

The test program was designed to capture the flexural performance of concrete beams reinforced with BFRP bars over a wide range of reinforcement ratios. This approach establishes the flexural performance of such beams over the practical ranges of reinforcements. Therefore, reinforcement ratios were selected between 35% and 350% of the balanced BFRP reinforcement ratio for the beam size and the properties of the materials used in this test program. This range of reinforcement ratios is believed to establish a good insight into the flexural performance of BFRP bar reinforced concrete beams.
3.2. Experimental Investigation

3.2.1. Test Program

3.2.1.1. Description of Test Beams

Fifteen beams were made in three groups each to a length of 7 feet (2,135 mm). The elevation of the test setup is shown in Fig. 3.1. The test beams were designed to be tested over a span of 5 feet (1,525 mm). Two ends of the test beams were supported at 12 inches (305 mm) distance from the beam ends. Twelve inches of over-hang was considered to account for the uncertainty of the bond between BFRP bars and concrete, and the possibility of bars slipping. The load spreading beam had a span of 6 inches (152.4 mm), as it is shown in the Fig 3.1. According to uniform shear stress along the beam, two-legged #3 steel stirrups were provided at equal distance of 6.75 inches. To hold stirrups tight in place and make a solid cage, two #2 plain steel rebar were used at top of all beams.

Arrangement of BFRP bar reinforcement, and typical cross-sectional details of test beams are shown in Fig. 3.1. The cross-section of the beams had a uniform width of 8 inches (203 mm) and a uniform depth of 7 inches (178 mm) for all the test beams. The sizes and the corresponding number of bars along with the total area of the reinforcement are summarized in Table 3.1.
3.2.1.2. Materials

Concrete was supplied by a local ready mix concrete supplier. Three concrete batches were used on different dates, which resulted in various 28 days strength $f'_{c}$ for each group. BFRP bar reinforcement was supplied by the sponsor in three sizes. BFRP bars used for the study were manufactured for the current project. The net fiber area relates to the bar diameters that were approximately 3, 5 and 7 millimeters, with a fiber volume fraction of approximately 50%. The corresponding gross diameters of the three bars were 4.3 mm, 7.1 mm and 9.8 mm. Therefore, these bars were designated as R4, R7 and R10 bars by the manufacturer sponsor. To ensure better bond behavior of BFRP bars with concrete, all the bars were sand coated. The mechanical properties that are of interest in
structural design are guaranteed tensile strength, modulus, and rupture strain. In the previous chapter, the guaranteed properties of BFRP bars were calculated. As per the pertinent standard, the guaranteed properties are suggested to be used for design purpose, however, the mean values of tensile strength and modulus of elasticity, which are greater than guaranteed ones, are used in the analysis for determining normal strength. A summary of the mechanical properties needed for beam analysis are given in Table 3.2.

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Diameter (mm)</th>
<th>Area (in²)</th>
<th>$f_{\text{f,Ave}}$ (ksi)</th>
<th>$E_{\text{f,Ave}}$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R4</td>
<td>0.17 (4.3)</td>
<td>0.0225 (14.5)</td>
<td>160.9 (1,110)</td>
<td>5,950 (41.1)</td>
</tr>
<tr>
<td>R7</td>
<td>0.28 (7.1)</td>
<td>0.0615 (39.7)</td>
<td>157.2 (1,084)</td>
<td>6,030 (41.4)</td>
</tr>
<tr>
<td>R10</td>
<td>0.39 (9.8)</td>
<td>0.1172 (75.6)</td>
<td>154.7 (1,067)</td>
<td>6,530 (45.1)</td>
</tr>
</tbody>
</table>

Note: $f_{\text{f,Ave}}$: Average tensile strength
$E_{\text{f,Ave}}$: Average modulus

3.2.1.3. Beam Preparation

The formwork for the beams was fabricated in the laboratory. Test beams were made in a set that accommodated three beams in one set (Fig. 3.2). Each time of the pour, concrete was poured into two sets of forms giving six beams in total for a single pour (Fig. 3.3) except for the first set where three beams were made (B1, B2, B3). The concrete for the beams was carefully placed in the forms, consolidated with a needle vibrator, and finished manually. The wet concrete was allowed to harden for 24 hours within the forms.
and then forms were disassembled. Soon after disassembly of formwork, the beams were wrapped with wet burlap and plastic sheets to prevent moisture loss of the test specimens.

Fig. 3. 2 Formwork for Test Beams
3.2.1.4. Test Set-up and Instrumentation

The beam tests for beams B1 to B9 were performed in a universal testing machine UTM with 300 kip (1,335 kN) capacity (Fig 3.4). Strain gages were attached to the reinforcing bars prior to the placement of concrete, and then sealed with a wax coat to prevent any short circuit while concrete is wet. These strain gages recorded the strains developed in the reinforcement bars during loading. Two strain gages were also attached to the concrete surface at the top of test beams to record the concrete strains at the top surface of the beams. Deflections were measured at the mid-span of the test beams during the initial 60 to 75 percent of the loading with help of two digital dial gages. From the bottom of beam a U-shape frame was attached to the mid-span. The frame was provided with an arrangement which allowed placing a dial gage on each side. The deflection gage was
later removed to prevent any possible damage to the deflection gage due to the failure of the beam.

The beam tests for specimens B10 to B15 were performed in a different test setup. This time a MTS machine with a hydraulic actuator and digital controller was used (Fig. 3.5). Bar’s strain gages were placed on the same location as the first and second groups of test specimens. Deflections were measured at the mid-span of the test beams during most of the loading with two devices, a LVDT attached to the bottom center of beams which was removed before any failure occurs, plus the actuator itself which was measuring deflection till failure. Unlike groups one and two, for this group deflection data were available for entire loading process.

The rate of loading was controlled by a closed loop servo-hydraulic system at an approximate rate of 2,400 pounds per minute initially for the first half of the test, and 1,200 lb in the latter half of each beam test (Fig. 3.6). The deflection and strain readings were recorded at an interval of one second.
Fig. 3. 4 Set-up 1 for the Beams B1 to B9

Fig. 3. 5 Set-up 2 for the Beams B10 to B15
3.2.2. Beam Tests and Typical Failure Modes

3.2.2.1. Test Beams B1 to B9

These nine beams were loaded gradually and uniformly at a rate of about 1200 lbs per minute. A digital data acquisition system was used to record the load and strain readings. The deflections were recorded constantly. The development of cracks in the beams was tracked and noted. The failure mode was documented after the failure occurred in each test beam. Failure in this test program was assumed to have reached when a beam no longer continues to carry increasing load, and the load falls soon after reaching the peak load, which is defined as the failure load.
The failure of the test specimens typical of beams B1 to B9 is shown in Fig. 3.7 and Fig. 3.8. The beams failed in a ductile manner with large mid-span deflections, with the maximum deflections at failure reaching values over 2 inches (50 mm). The beams failed either by tensile rupture of the BFRP bars or concrete crushing depending on the relative reinforcement ratios. Cracks formed at nearly uniform spacing close to the mid-span of each beam. The crack pattern basically comprised of evenly spaced multiple narrow cracks rather than sparsely spaced wide cracks. In the case of GFRP reinforced beams, it was reported that flexural cracks formed within the mid-span and became considerably wider and deeper as the load increased due to low modulus of elasticity to tensile strength ratio of GFRP bars [3.4]. Similar observations were also made in the case of BFRP reinforced beams too.
Fig. 3. 7 Typical Failure Mode of Test Beams (Beam B1)

Fig. 3. 8 Typical Failure Mode of Test Beams (Beam B6)
3.2.2.2. Test Beams B10 to B15

The failure mode that was typical of beams B10 to B15 is as shown in Figs. 3.9 and 3.10. The deflection of each test beam developed gradually with loading along with uniformly spaced cracks. The failure mode can be characterized as ductile resulting in large deflections and uniform crack patterns. The large deflection close to failure is shown in Fig. 3.9. The cracks were initially narrow, but increased in width, depth and number closer to failure load. The beams were able to sustain the load even after sustaining large deflections for considerable amount of time.

Fig. 3.9 Nearly Uniform Flexural Cracking (Beam B11)
3.3. Experimental Results and Discussion

3.3.1. Test Results

The failure loads for test beams are shown in Table 3.3. The table also shows the load predicted by using the ACI 440.1 committee recommended method for the determination of nominal moment strength for concrete beams reinforced with FRP bars. The tensile strength and modulus used in the calculations for the determination of ACI predicted failure loads are 155 ksi (1070 MPa) and 6,000 ksi (41.37 GPa), respectively. The method for the calculation of the moment-strength of steel-reinforced-section is specified in ACI 318-08 and ACI 440.01R-06 for FRP reinforced concrete sections. They both are based on similar principle of replacing the nonlinear-distribution of the compressive
stress across the section in the concrete by an equivalent ACI rectangular stress block for the calculation of the compressive force acting on the section. For the rectangular stress-block, the strains are assumed to be varying linearly with the depth of the stress-block, the proportionality factor depending on the maximum concrete compressive strain. In case of the steel-reinforced sections, the stress-strain relationship is assumed to be bilinear to simplify the calculation of the tensile force being carried by the steel bars. Whereas in the case of FRP reinforced section, the actual linear stress-strain relationship is incorporated in the analysis. The ACI stress values listed in the table correspond to the stresses determined by solving the quadratic equation given in the ACI report. These values provide insight into the level of utilization of the strength of BFRP bars. The percentage difference listed in the table shows the difference between the prediction and the actual load obtained in the tests relative to the ACI predictions. This is expressed as the percent by which the actual load deviated from the ACI prediction. A negative value indicates that the load obtained from tests is greater than the predicted value as shown in the column corresponding to the comments.
### Table 3. Summary of Test Results

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>(A_t), in(^2)</th>
<th>(P_u), lb</th>
<th>ACI Load, lb</th>
<th>ACI Stress, psi</th>
<th>Difference %</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Group 1, (f'_c=4.90) ksi; (E_c=3,990) ksi</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B1</td>
<td>0.045</td>
<td>3,784</td>
<td>3,034</td>
<td>155</td>
<td>-24.7</td>
<td>Test &gt; ACI</td>
</tr>
<tr>
<td>B2</td>
<td>0.1231</td>
<td>7,444</td>
<td>7,814</td>
<td>146.1</td>
<td>4.7</td>
<td>Test &lt; ACI</td>
</tr>
<tr>
<td>B3</td>
<td>0.2344</td>
<td>11,140</td>
<td>10,748</td>
<td>106.5</td>
<td>-3.6</td>
<td>Test &gt; ACI</td>
</tr>
<tr>
<td><strong>Group 2, (f'_c=3.70) ksi; (E_c=3,470) ksi</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B4</td>
<td>0.0674</td>
<td>4,506</td>
<td>4,543</td>
<td>155</td>
<td>0.8</td>
<td>Test &lt; ACI</td>
</tr>
<tr>
<td>B5</td>
<td>0.1065</td>
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<td>6,543</td>
<td>142.3</td>
<td>-12</td>
<td>Test &gt; ACI</td>
</tr>
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<td>7,903</td>
<td>113.7</td>
<td>-21.2</td>
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</tr>
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<td>8,410</td>
<td>106.6</td>
<td>1.4</td>
<td>Test &lt; ACI</td>
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<td>B9</td>
<td>0.3515</td>
<td>11,191</td>
<td>10,875</td>
<td>73.8</td>
<td>-2.9</td>
<td>Test &gt; ACI</td>
</tr>
<tr>
<td><strong>Group 3, BFRP Reinforcement, (f'_c=5.90) ksi; (E_c=4,380) ksi</strong></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B10</td>
<td>0.0674</td>
<td>4,435</td>
<td>4,755</td>
<td>155</td>
<td>6.7</td>
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<td>0.1065</td>
<td>7,962</td>
<td>7,506</td>
<td>155</td>
<td>-6.1</td>
<td>Test &gt; ACI</td>
</tr>
<tr>
<td>B12</td>
<td>0.1231</td>
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<td>8,682</td>
<td>155</td>
<td>-2.5</td>
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</tr>
<tr>
<td>B13</td>
<td>0.1455</td>
<td>8,439</td>
<td>9,731</td>
<td>147.3</td>
<td>13.3</td>
<td>Test &lt; ACI</td>
</tr>
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<td><strong>Group 3, Steel Reinforcement, (f'_c=5.90) ksi; (E_c=4,380) ksi</strong></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B14</td>
<td>0.22</td>
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<td>7,010</td>
<td>68</td>
<td>-43.9</td>
<td>Test &gt; ACI</td>
</tr>
<tr>
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<td>15,280</td>
<td>12,470</td>
<td>68</td>
<td>-22.5</td>
<td>Test &gt; ACI</td>
</tr>
</tbody>
</table>

Note: 1in=25.4 mm; 1000 lb = 4.45 kN; 1 ksi=6.9 Mpa

A comparison of the test results with the predicted failure loads reveals that beams B2, B10 and B13 failed at loads less than predicted. All other beams exhibited strength greater than the corresponding predicted strength. A comparison of test results with the predictions is shown in Fig. 3.11 with test results on the Y-axis and the ACI predicted failure loads on the X-Axis. A line of equality is also shown in the figure. The predictions match perfectly with the test data if the data points fall exactly on the line of equality. The test results indicate better strength than predicted if the data points fall above the line of equality. The figure shows that the test results for most beams are very close to the line...
of equality demonstrating that the test results are close to the predicted values. The failure loads obtained from tests for three beams B2, B10 and B13 fell short of the predicted failure loads by 4.7, 6.7, and 13.3 percent, respectively. However, the distribution of the test data about the line of equality shown in Fig. 3.11 is a normal distribution expected for reinforced concrete beams and is generally acceptable. The data collected during the tests also include deflection data, strain data for the bars and the concrete compressive strains. A typical load-deflection curve is shown in Fig. 3.12. A typical load-strain curve is shown in Fig. 3.13.

Fig. 3. 11 Comparison of Test Results with ACI Predicted Failure loads
Fig. 3.12 Typical Load Deflection Curve

Fig. 3.13 Typical Load Strain Curve
Crack maps were plotted for each beam to record the crack development for each case. Crack maps for all tested beams are shown in Fig. 3.14. These curves resemble crack patterns that are as expected of any steel-reinforced concrete beam.
Fig. 3. 14 Crack Map of Test Beams
Fig. 3.14 Crack Map of Test Beams (Continued)
3.4. Load-Deflection Analysis

From the serviceability point of view, load-deflection analysis constitutes a very important part of the analysis for which importance cannot be underestimated. Owing to the lower flexural stiffness of the FRP bars compared to steel, deflection is a governing parameter for the design of the FRP reinforced concrete beams. It can be inferred from the previous research and investigations that load-deflection analysis of the reinforced concrete beams hinges on the reasonable prediction of the effective moment of inertia after the concrete in the tension zone is cracked. Once the concrete in the tension zone is cracked, the flexural stiffness of the section reduces significantly due to the considerable reduction in the moment of inertia of the section.

Since concrete is a nonlinear material and the composite action of the reinforced concrete section cannot be analyzed from the elastic method once the concrete is cracked, the prediction of effective moment of inertia of the cracked section is always a difficult task. The classic effective moment of inertia is Branson’s equation (Eq. 3.1) which was introduced and been used in design offices for steel reinforced concrete beams. Since this equation underestimates FRP reinforced concrete beams deflection, several researchers added correction factors to make it applicable to FRP reinforced concrete flexural members. However it should be noted that due to the lower stiffness of the FRP reinforcement, the nonlinear transition from cracking moment to effective moment of inertia of the cracked section can be much faster than much accepted cubical models. It was observed by Yost et. al [3.5] that in the case of GFRP beams, the nonlinear transition
from the cracked moment of inertia to intended effective moment of inertia happens much faster as compared to the similar steel beams.

\[(Eq. \ 3.1)\]

The next chapter is dedicated to deflection prediction of FRP reinforced concrete members. Extensive evaluation was done on earlier research work by others and a new equation is proposed for Basalt FRP-RC reinforced concrete members, which may be used in the corresponding design guidelines.

3.5. Different Approaches of RC Beam Design

3.5.1. Strain Compatibility Methods

In this section, the beam flexural strengths are calculated with applying actual stress-strain curve of concrete instead of simplified rectangular compressive block which ACI recommends. Therefore, three well-known methods were considered for which there is definition, details and relationships of each one and finally, their results were compared with ACI approach and experimental test results. It is well accepted that strain compatibility methods are more efficient in term of using maximum capacity of reinforced concrete beams.
3.5.1.1. Concepts

The ACI 440 report approach for strength design of FRP-RC flexural members’ section is the same as ACI 318 requirements for steel reinforced concrete, which factored nominal strength of section, should not be smaller than ultimate strength.

\[ (Eq. \ 3.2) \]

The nominal flexural strength \( M_n \) directly relates to the section dimensions and reinforcement ratio. The section could be under-reinforced (UR) or over-reinforced (OR) based on the ratio of reinforcement ratio \( (\rho_f) \) to the balanced reinforcement ratio \( (\rho_{fb}) \). These values could be calculated by the following equations

\[ \text{Equation 3.3} \]

\[ \text{Equation 3.4} \]

Where;

\( A_f = \) area of FRP reinforcing bar
\( b = \) beam width
\( d = \) effective depth
\( \varepsilon_{cu} = \) ultimate strain in out fiber of concrete section
\( E_f = \) modulus of elasticity of FRP reinforcing bar
\( f'_{ce} = 28^{th} \) day compressive strength of concrete
\( f_{fu} = \) ultimate strength of FRP reinforcing bar
The reinforcement ratio governs the failure mode. The under-reinforced section is tension controlled, that is the rupture of FRP rebar governs the capacity. So here, because the member fails before the compressive concrete reaches its ultimate strain, $\varepsilon_{cu} = 0.003$, the compressive stress block may not be considered and nominal moment could be calculated by Eq. 3.5. However, ACI 440 recommends a conservative expression by adding a reduction factor of 0.8 to the Eq. 3.5. to come up with Eq. 3.6 as following:

$$ (\text{Eq. 3.5}) $$

$$ (\text{Eq. 3.6}) $$

The best way to have more accurate results is compatibility method which means to define relationship between concrete stress and strain at any level to be a substitute for the stress block. In this section three models were considered. The analysis is to determine concrete’s strain at failure, depth to neutral axis and $\alpha_{1E}$ and $\beta_{1E}$, which are equivalent stress block parameters [3.17].

3.5.2. Methods

3.5.2.1. Effective Stress Block Parameters

Let’s consider a rectangular beam section. To determine the moment capacity, compressive force in concrete $C$, tensile force in reinforcement $T$ and the moment arm are required. Therefore, for $C$, we need to calculate area under the compressive stress
block curve. In addition, for moment arm we have to find the centroid of the compressive block.

The actual stress block does not have a linear form from the extreme compressive fiber (top of flexural member) to the neutral axis. So, two coefficients have been defined to generalize stress block formula as it is demonstrated in Fig.3.13. Coefficient $\alpha$ is stress intensity and $\beta$ is resultant location coefficient, which could be calculated as following:

\[
\text{total force}
\]

(Eq. 3.7)

On the other hand, compressive force on a small area is

\[
\text{(Eq. 3.8)}
\]

which by integration we have compressive force as

\[
\text{(Eq. 3.9)}
\]

which results in

\[
\text{(Eq. 3.10)}
\]
for simplicity, take $c=1$ to come up with

\[ \beta \]

(Eq. 3.10)

calculation of $\beta$, resultant location coefficient

With unit depth of neutral axis, $c=1$, then $\beta c = \beta$ so if we take moment about neutral axis

(Eq. 3.11)

therefore,

\[ \beta = \frac{1}{c} \]

(Eq. 3.12)

Where

(Eq. 3.13)

Rearrange the equation for $\beta$, we have
However, there is relationship between the generalized stress block parameters ($\alpha$ and $\beta$) and simplified rectangular stress block factors ($\alpha_{1E}$ and $\beta_{1E}$), which could be derived using geometry and equating the area under each stress block (which is total compressive force). The equations below show their relationship [3.17]:

(Eq. 3.16)
3.5.3. Stress-strain models [3.17]

To calculate the mentioned parameters, it is necessary to have a relationship between stress and strain in concrete. Numerous models were proposed during past 30 years, which some of them were for normal strength concrete and some for high performance concrete. In this chapter, three common constitutive stress-strain models are considered to compute the stress block parameters.

3.5.3.1. Model 1: Desayi and Krishnan (1964) [173.]

Desayi and Krishnan proposed their model as Eq. 3.18

\[
\frac{f_c}{f'_c} = \frac{\varepsilon_c}{\varepsilon'_c}
\]

where

\( f_c \) is stress in concrete at any strain level \( \varepsilon_c \)

\( f'_c \) is maximum stress in concrete (peak of the stress-strain curve)

\( \varepsilon'_c \) is concrete strain at \( f'_c \) level, which for normal strength concrete may be assumed as 0.002

Fig 3.14 represents the stress-strain relationship in concrete with this mode for different concrete strengths. Simplicity of this model makes is easy to evaluate the closed form...
integration to calculate stress block parameters $\alpha$ and $\beta$. Fig 3.14 and Fig 3.15 show the curve, which is resulted from the equation, for different concrete strength levels varying between 2.9 ksi (20 MPa) and 14.5 ksi (100 MPa). The most advantage of this model is its simple form. A lot of researchers found this simplicity very interesting and applied it as general concrete stress-strain model. Also, Fig 3.20 and Fig 3.21 show the diagrams related to related parameters $\alpha_{1E}$ and $\beta_{1E}$.

It is important to mention that Desayi and Krishnan model were developed for normal strength concrete NSC, which is obvious from the range of strength on the curves, and it does not match results of high performance concrete HPC.

![Stress-strain relationship of concrete (Desayi and Krishnan, 1964)](image)

Fig. 3. 16  Stress-strain relationship of concrete (Desayi and Krishnan, 1964) [3.17]
Fig. 3.17 Value $\alpha$ for different concrete strengths (Desayi and Krishnan model, 1964) [3.17]

Fig. 3.18 Value $\beta$ for different concrete strengths (Desayi and Krishnan model, 1964) [3.17]
Fig. 3.19 Value $\alpha_{1E}$ for different concrete strengths (Desayi and Krishnan model, 1964) [3.17]

Fig. 3.20 Value $\beta_{1E}$ for different concrete strengths (Desayi and Krishnan model, 1964) [3.17]
3.5.3.2. Model 2: Kent and Park (1971) [3.17]

Kent and Park tried to extend Hongnestad’s work with focus on stress-strain behavior of concrete after the ultimate strength peak (post peak behavior). They modified the ascending part of the curve to come up with the following relationship

\[ \text{---} \quad \text{---} \quad \text{---} \quad \text{(Eq. 3.19)} \]

when \[ \text{---} \quad \text{---} \quad \text{---} \quad \text{---} \quad \text{---} \quad \text{(Eq. 3.19)} \]

then

![Stress-strain relationship of concrete (Kent and Park model, 1971) [3.17]](image-url)
Fig. 3.22 Value $\alpha$ for different concrete strengths (Kent and Park model, 1971) [3.17]

Fig. 3.23 Value $\beta$ for different concrete strengths (Kent and Park model, 1971) [3.17]
Fig. 3. 24 Value $\alpha_{1E}$ for different concrete strengths (Kent and Park model, 1971) [3.17]

Fig. 3. 25 Value $\beta_{1E}$ for different concrete strengths (Kent and Park model, 1971) [3.17]
Collins, Mitchell and MacGregor proposed the following equation which was developed from Thorenfeldt et al. work, which explains how compressive stress at any moment (any strain level) is related to maximum compressive stress $f'_{c}$. They came up with two factors $n$ and $k$, which are linear factors of maximum stress of concrete; $k$ is post stress decay factor which was applied to describe the descending part of the stress-strain curve.

\[
\frac{\sigma}{n} = k
\]  

(Eq. 3.20)

where

for $\sigma$ in psi when

and for $\sigma$ in MPa when
Fig. 3.26 Stress-strain relationship of concrete (Collins et al. model, 1993) [3.17]

Fig. 3.27 Value $\alpha$ for different concrete strengths (Collins et al. model, 1993) [3.17]
Fig. 3. 28 Value $\beta$ for different concrete strengths (Collins et. al model, 1993) [3.17]

Fig. 3. 29 Value $\alpha_{1E}$ for different concrete strengths (Collins et. al model, 1993) [3.17]
3.5.4. Calculation results

All three methods were applied to calculate each beam’s flexural capacity and their corresponding loads, as per 4-point load set up. As it mentioned previously, there were three groups of beams with different concrete compressive strength, \( f'_c \), which result in having different values of \( \alpha, \beta, \alpha_{1E} \) and \( \beta_{1E} \) for each group. The ultimate loads were calculated using extracted values from graphs for each method and the results are summarized and compared with ACI prediction and actual test results in Table 3.4.
### Table 3.4: Capacity Loads Comparison

<table>
<thead>
<tr>
<th>No.</th>
<th>$A_f$ in$^2$</th>
<th>$P_u$, lb</th>
<th>ACI Load, lb</th>
<th>Desayi &amp; Krishnan, lb</th>
<th>Kent &amp; Park, lb</th>
<th>Collins et. al, lb</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Group 1, $f'_c=4.90$ ksi; $E_c=3,990$ ksi</strong></td>
<td></td>
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<td></td>
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<td></td>
</tr>
<tr>
<td>B1</td>
<td>0.045</td>
<td>3,784</td>
<td>3,034</td>
<td>3,258</td>
<td>3,258</td>
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<td>7,814</td>
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<td>10,748</td>
<td>11,494</td>
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<td><strong>Group 2, $f'_c=3.70$ ksi; $E_c=3,470$ ksi</strong></td>
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<td>7,707</td>
<td>7,702</td>
<td>7,702</td>
</tr>
<tr>
<td>B12</td>
<td>0.1231</td>
<td>8,900</td>
<td>8,682</td>
<td>8,876</td>
<td>8,870</td>
<td>8,870</td>
</tr>
<tr>
<td>B13</td>
<td>0.1455</td>
<td>8,439</td>
<td>9,731</td>
<td>10,168</td>
<td>9,819</td>
<td>9,819</td>
</tr>
</tbody>
</table>

Note: 1 in = 25.4 mm; 1000 lb = 4.45 kN; 1 ksi = 6.9 Mpa

### 3.6. Discussion and Summary

A bar chart showing the comparison among the three strain compatibility method approaches, ACI 440.1 predictions and the loads obtained from tests is shown in Fig. 3.31. From Table 3.4 and Fig. 3.32, it is demonstrated that most of the BFRP bar reinforced concrete beams in this test program (8 out of 13) have outperformed the ACI 440.1 predictions while less than half of them (6 out of 13) surpassed compatibility method approaches prediction. All the strain compatibility method approaches predicted larger capacity compared to ACI 440 approach, and amongst all models, Desayi & Krishnan indicated larger failure load. Test results of beams B2, B4, and B8 indicate less than 5% underperformance, which is within the experimental variation expected for
reinforced concrete beams. However, it appears that test beams reinforced with larger bars (R10 and possibly, R7) may not have fully developed the bond between concrete and the BFRP reinforcing bars.

Fig. 3.31 Bar Chart of Failure Loads of Test Beams, ACI 440.1 Predictions and Strain Compatibility Methods Prediction

Fig. 3.32. shows the development of load carrying capacity with increased area of BFRP reinforcement. This figure gives an indicative insight into the performance of BFRP bars for concrete beam applications, while recognizing that the beams have different concrete compressive strength. Curves fitted through the test results and compared with a curve drawn through the ACI 440.1 predictions are shown in the figure where the curve for test
results is higher than that corresponding to ACI predictions. The curves indicate fairly acceptable flexural performance of reinforced concrete beams using BFRP bars.

![Graph showing test results and ACI predictions for beam failure loads against area of BFRP bar reinforcement.](image)

**Fig. 3.** 32 Failure Loads of Test Beams and ACI 440.1 Predictions relative to the Area of Reinforcement

Evenly spaced crack development revealed that the BFRP bars may have provided the required bond between the bars and concrete. The significant mid-span deflections at failure of the test beams also indicated that the beams performed in a ductile manner.

After the flexural tests on the fifteen beams including the control steel-beams, all the beams were arranged and the crack-distribution on the beams was studied. Due to the unavailability of the instrument for the measurement of the crack-width, the theoretical crack-width predicted for the BFRP beams could not be compared with the crack-width obtained from the four-point bending. Even though the development of crack with time could not be
determined, the overall spatial distribution of the flexural cracks along the length of the beam was visually depicted. This might help us to have a relative understanding of the distribution of flexural cracks in the case of FRP reinforced beams compared to those of steel reinforced beams.
CHAPTER IV

BEAM DEFLECTION AND EFFECTIVE MOMENT OF INERTIA

The flexural behavior of concrete beams reinforced with basalt fiber reinforced polymer BFRP bars is discussed in this chapter in order to establish an expression for the effective moment of inertia, $I_e$, used for the calculation of beam’s deflections. Acknowledging the fact that serviceability dominates FRP-RC beam design, and also BFRP being a new material, lack of such an equation in ACI 440 code is understandable. The experimental results of a total five beams and one slab are discussed to determine a reliable alternative expression for the current ACI 440 equation. Finally, a set of nonlinear multiple regression analysis was performed to propose a new equation especially for Basalt FRP rebar.
4.1. Introduction

In the deflection calculation equation, for any particular cross section regardless of the reinforcement ratio and material properties, the only parameter varying during loading process is the moment of inertia, which decreases significantly after the occurrence of cracks at tensile zone. Considering this fact, many attempts have been made to derive the best relation for calculating the effective moment of inertia at different load levels. In 1965, Branson [4.1] proposed a semi-empirical equation for effective moment of inertia for deflection calculations of flexural members reinforced with steel rebar. Relationships recommended by North American standards, ACI 318 [4.2] and CSA A23.3-94 [4.3] were based on Branson’s formulas. Although, Branson’s classic equation has been showing good level of prediction for steel reinforced concrete beams, it has been inadequate to address the deflection of FRP reinforced beams. The major influencing factor was the difference between steel rebar and FRP bar stiffness. Some of the researchers have found the difference in interaction mechanisms to be one of the reasons for the inapplicability of Branson’s equation in the case of FRP-RC structures [4.4]. Owing to their lower flexural stiffness, the nonlinear transition from cracking moment to effective moment of inertia can be much faster such that the accepted assumption of cubical decay (as in the case of steel reinforced concrete beams) may not be a reasonable assumption in the case of FRP-RC structures.

In the case of GFRP beams, the transition from cracking to the effective moment of inertia has been found to be much faster [4.5]. A fifth order modified Branson’s equation has been proposed by Brown and Bartholomew [4.6] in the case of FRP reinforced beams.
to account for the faster decay. Since FRP bars do not undergo plastic deformation and have lower stiffness, it is a reasonable assumption that the dissipation of energy is much faster in case of FRP reinforced beams as compared to a similar steel reinforced section.

Branson’s equation has a good range of accuracy when the ratio of gross moment of inertia to the cracking moment of inertia is between 1.5 and 4 [4.7] which is generally the case for steel reinforced section. FRP beams have been found to have this ratio greater than five, thus leading to a stiffer response and the consequent under prediction of deflection by original Branson’s equation [4.8].

Several researchers have believed that the general form of effective moment of inertia equation should remain as similar as possible to the Branson’s equation (Benmokrane et al. [4.9], Brown & Batholomew [4.6], Toutanji and Safi [4.10]). They tried to add coefficients, either constant values or function of some parameters, to the original form of Branson’s equation to adjust it for particular cases. On the other hand, Bischoff argued limitations of Branson’s equation, which is not valid for all reinforcement ratios for steel and FRP reinforced concrete flexural members. He has presented an alternative approach, which could cover all reinforcement ratios. Bischoff used a mechanics based approach, considering the concrete contribution of the tensile zone after cracking, termed as tension stiffening. One of the important parameter governing the structural responses of a reinforced beam is the phenomenon called Tension Stiffening. It is one of the basic assumptions of reinforced concrete beam design, that the tensile load is totally carried by the reinforcement once the concrete is cracked. However, the concrete segments between the cracks are able to carry the tension due to the bond with the reinforcement. This
phenomenon is technically termed tension stiffening [4.11]. Tension stiffening plays an important role to control member stiffness, deformation characteristics and crack-width properties. From the rigorous study, it was found that the GFRP reinforced beams exhibit more tension stiffening than the corresponding steel reinforced beams owing to their lower modulus of elasticity [3.10]. ACI model has been found to underestimate the deflection in the case of FRP reinforced concrete beams as reported by previous researchers.

The additional contribution of concrete from the cracked section thus tends to provide stiffer response of the member [4.12]. It was argued by Bischoff that the tension stiffening component in Branson’s equation tends to increase significantly as the ratio of gross-to-cracked moment of inertia increases, specifically for the low modulus bars and lower reinforcement ratio. Steel reinforced beams have this ratio generally between 2 to 3 whereas in the case of GFRP reinforced beams it was found to be varying between 5 to 25 [4.13]. The deflection calculations based on original and modified Branson’s equation were found to be overestimating the stiffness hence the under prediction of deflection. This was observed by Al-Sunna et al.[3.12] as well as by Rafī and Nadjai [3.13] based on their work on GFRP and CFRP reinforced beams respectively. This subsequently led to the ill-modeling of tension stiffening component hence the overestimation of the stiffness of the FRP reinforced beams when used in conjunction with Branson’s equation.

Almost all analytical and experimental studies so far have been performed on three commercially available fibrous bars, CFRP, GFRP and AFRP. In the current study a new
type of FRP is used which is made of basalt fibers and vinyl-ester matrix. To validate the available developed methods of deflection calculation for BFRP-RC beams, mid-span deflections of a set of BFRP-RC beams were measured running 4-point loading beam tests. On the other hand, theoretical load-deflection curves by five selected approaches were selected to compare the actual and theoretical beam behavior, load-deflection curves from test and theoretical approaches were plotted on the same graph for each beam. Moreover, as the serviceability of FRP-RC flexural members dominates their design and relatively large variability of elastic modulus of these bars, a need for reliability study seems inevitable. A set of reliability methods was used to evaluate probability index and probability of failure for mid-span deflection compared with ACI 440.1R [4.14] allowable deflection.

Theoretical approaches of deflection calculation are based on serviceability conditions thus using the linear stress-strain constitutive relationship of concrete accounting only for the flexural effects on deflection [3.14]. It was observed that by using non-linear constitutive models, the deflection predictions are much better when the load reaches a higher level. The effects of diagonal shear cracks may also be a governing factor that is required to be addressed. The assumption of perfect bond between concrete and the reinforcing bar may also be a critical parameter in the case of FRP bars as the bond strength is significantly lower in this case as compared to the corresponding steel bar. It is also to be noted that the flexural stiffness varies along the length of bar and it may be an important factor to be considered for deflection analysis [3.15].
It was observed by various researchers that the deflection prediction based on effective moment of inertia as proposed by Branson is dependent on the reinforcement ratio. As per Yost [3.5], there is a correlation between the degree of overestimation of the effective moment of inertia and the reinforcement ratio. As the ratio of provided reinforcement ratio to balanced reinforcement increases, the error in the prediction decreases (ACI 440.1R). It was observed by Ashour [3.4] that the reinforcement ratio of GFRP bars had a considerable effect on the flexural stiffness and deformations of the tested beams.

4.2. Research Significance
Unlike steel reinforced concrete beams, the serviceability dominates the design of FRP-RC flexural elements. Therefore, existence of a reliable approach for deflection calculation seems inevitable. Although, ACI 440.1R procedure is commonly used in design offices, several researchers have argued that it cannot satisfy all type of FRP-RC members. The aim of this study is investigate ACI 440.1R procedure along with other approaches so as to be applied to BFRP-RC beams, and to establish suitable model for \( I_e \) which plays key role in deflection calculation of FRP-RC beams.

4.3. Background
Mid-span flexural deflection \( \Delta \) of a simply supported beam with total span \( L \) and shear span \( S \) under two equal symmetric point loads of \( P/2 \) can be calculated as:

\[
\text{(Eq. 4.1)}
\]
where $E_c$ is concrete modulus of elasticity, and $I_e$ is effective moment of inertia. For steel reinforced concrete beams, ACI-318 recommends classic Branson’s equation (Eq. 4.2)

$$\text{Eq. 4.2}$$

Where $M_{cr}$ is cracking moment, which is based on modulus of rupture and is calculated by empirical equation of ACI 318, $M_a$ is moment at mid-span (point of maximum deflection), $I_g$ and $I_{cr}$ are gross and cracked moment of inertia respectively. Because of the fact that FRP bars have lower stiffness compared to steel, Branson’s expression gives larger value of $I_e$ and consequently under-predicts deflection of FRP-RC beams. To adjust Branson’s expression to suit the FRP-RC beams, many researchers have been working on deriving a suitable expression. As Branson’s equation is simple and widely used in designs, most of researchers tried to keep the original form of Branson’s equation and just add appropriate correction factors to it to be consistent. ACI 440.1R-03 [4.14] offered a factor $\beta$ to include the effects on $I_g$ based on relative modulus of elasticity of FRP and steel along with a bond dependent coefficient $\alpha$, which is considered to be 0.5 until more research evolves

$$\text{Eq. 4.3}$$
To complete this equation, Yost et al. have presented an expression for $\alpha$, based on experimental study on GFRP-RC beams. They have found that the ratio of FRP reinforcement to balanced reinforcement as key factor for coefficient $\alpha$. The following expression was extracted by a linear regression analysis [4.5].

\[ \text{(Eq. 4.4)} \]

The ACI committee 440 [4.15] then revised the equation in 2004 and proposed a simpler reduction factor $\beta$, this time relative reinforcement ratio (ratio of $\rho_f$ to $\rho_{fb}$) seems to be dominating instead of elasticity ratio.

\[ \text{(Eq. 4.5)} \]

A further investigation to establish corrective coefficients for Branson’s equation was performed by Rafi and Nadjai [4.16]. They analyzed a large number of beams and slabs reinforced with GFRP, CFRP, and AFRP bars to derive appropriate factors to modify ACI 440.1R calculation method of the effective moment of inertia. As they figured out, $I_e$ is influenced mainly by modulus of FRP, $E_f$. Therefore, with the same definition of $\beta$, they suggested a factor $\gamma$ to effect on second term of Eq. 4.3, $I_{cr}$.
Al-Sunna et al. [4.17] offered a new set of modification coefficients for Eq. 4.2 based on the experimental study of 28 beams and slabs reinforced with GFRP and CFRP. In fact, they proposed a constant coefficient $\alpha$ for different types of rebar, either FRP or steel, and a new expression for $\beta$ involving reinforcement ratio and elastic properties of rebar, which is

On the other hand, Bischoff et al. introduced a new approach for the calculation of the effective moment of inertia which is based on “springs in series” method [4.8]. He
claimed that this approach could be valid for all reinforcement ratios for both steel and FRP reinforcement.

\[
\text{(Eq. 4.13)}
\]

The above methods formed the basis for establishing the relevant method for BFRP-RC beams in this study.

4.4. Test Beams

A total six beams reinforced with BFRP bars were tested. Amongst all, four beams of the first group were made to a length of 7 feet (2135 mm) and beams of second group had a length of 10 feet (3048 mm). The elevation of the test setup is shown in Fig 4.1. Due to uncertainty of BFRP reinforcements and possibility of bars slipping, the beams were designed with 12 inch overhang from each side which makes span of 5 feet (1524 mm) and 8 feet (2438 mm) for the first and second group respectively. The load spreading beam had a span of 6 inch (152.4 mm). To validate the recorded deflections data, mid-span deflections were measured with two devices. For each set up, a linear variable differential transducer LVDT was provided at the bottom of beam at mid-span and data were recorded via a digital MTS data logger. By using this device, deflections were recorded up to 60 to 75 percent of ultimate beam capacity, after which LVDT was removed to prevent any possible damage to it. On the other hand, the hydraulic actuator was able to measure deflection during loading process, which made it possible to record
data from start to the collapse of beam specimens. Later on, comparing measured
deflection through both devices showed accuracy and reliability of actuator.

The beams were designed to include a range of areas of basalt fiber bar reinforcement
varying from 0.045 inch$^2$ (29 mm$^2$) to 0.469 inch$^2$ (303 mm$^2$). The balanced basalt fiber
reinforcement ratio works out to be between 0.002 and 0.003 when the concrete strength,
the basalt bar strength, and modulus of elasticity corresponding to this test program are
used. The test program was designed to capture the flexural performance of concrete
beams reinforced with basalt fiber bars over a wide range of reinforcement ratios. This
approach establishes the flexural performance of such beams over the practical ranges of
reinforcements. Therefore, reinforcement ratios were selected between 54% and 349% of
the balanced reinforcement ratio for the beam size and the properties of the materials
used in this test program, as it is tabulated in Table 4.1. This range could give a good
overview of flexural behavior of BFRP reinforced concrete members.
For this study, three different sizes of BFRP bars were used with designation as R4, R7, and R10. Number following “R” represents the rounded approximate gross diameter of bars in millimeter. Diameter, area, and mechanical properties such as modulus of elasticity and tensile strength are summarized in Table 4.2. Tabulated mechanical properties of reinforcement bars are presented in chapter 2. Moreover, fiber volume fractions of bars reported by the manufacturer were about 50% for these R4, R7, and R10 bars.

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Reinforcement</th>
<th>$b$, in</th>
<th>$d$, in</th>
<th>$\rho/\rho_b$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group 3</td>
<td>$f_c'=5.90$ ksi; $E_c=4,380$ ksi</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2B-1</td>
<td>3 R4</td>
<td>8</td>
<td>6</td>
<td>0.54</td>
</tr>
<tr>
<td>2B-2</td>
<td>2 R4 + 1 R7</td>
<td>8</td>
<td>6</td>
<td>0.85</td>
</tr>
<tr>
<td>2B-3</td>
<td>2 R7</td>
<td>8</td>
<td>6</td>
<td>0.99</td>
</tr>
<tr>
<td>2B-4</td>
<td>2 R7 + 1 R4</td>
<td>8</td>
<td>6</td>
<td>1.17</td>
</tr>
<tr>
<td>Group 4</td>
<td>$f_c'=6.80$ ksi; $E_c=4,700$ ksi</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slab</td>
<td>4 R10</td>
<td>12</td>
<td>4</td>
<td>3.49</td>
</tr>
<tr>
<td>Cap Beam</td>
<td>4 R10</td>
<td>12</td>
<td>13</td>
<td>1.07</td>
</tr>
</tbody>
</table>

Note: 1 in = 25.4 mm; 1 ksi = 6.9 Mpa

<table>
<thead>
<tr>
<th>Bar</th>
<th>Diameter Dimension</th>
<th>Area Dimension</th>
<th>$f_{t,Ave}$ (ksi)</th>
<th>$E_{t,Ave}$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$D$, in (mm)</td>
<td>$A$, in$^2$ (mm$^2$)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>R4</td>
<td>0.17 (4.3)</td>
<td>0.0225 (14.5)</td>
<td>160.9 (1,110)</td>
<td>5,950 (41.1)</td>
</tr>
<tr>
<td>R7</td>
<td>0.28 (7.1)</td>
<td>0.0615 (39.7)</td>
<td>157.2 (1,084)</td>
<td>6,030 (41.4)</td>
</tr>
<tr>
<td>R10</td>
<td>0.39 (9.8)</td>
<td>0.1172 (75.6)</td>
<td>154.7 (1,067)</td>
<td>6,530 (45.1)</td>
</tr>
</tbody>
</table>
The concrete batches were supplied by a local concrete supplier. Compressive strength for each group $f'_c$ is determined from average strength of standard cylinders (4 in x 8 in) at uni-axial compressive test, also modulus of elasticity of concrete $E_c$ is estimated from empirical expression of ACI 318-08, which are listed in Table 4.1 for each group.

4.5. Comparison of Theoretical and Experimental Results

Fig. 4.2 shows the experimental and various theoretical load-deflection curves for small beams of group 1 (a) and big beam and slab of group 2 (b). Theoretical curves are based on previously mentioned equations (Eqs. 4.2, 4.4, 4.5, 4.6, 4.8, 4.9 and 4.10) for the effective moment of inertia $I_e$, derived by numerous researchers. Also, the curve labeled as ACI 440- Beta Factor was created based on ACI 440 recommended equation for $I_e$. It is clear in Fig. 4.3 that all theoretical equations resulted in fairly close prediction to the recorded experimental data. Most of theoretical equations, including ACI 440 recommendation, underestimated the beam deflections; however, the Bischoff’s relationship (Eq. 4.13) always indicated larger deflection.

Values of experimental $I_e$, which were calculated by applying recorded deflection data of tested beams using general Eq. 4.1, were compared with predicted theoretical $I_e$ computed by different existing theoretical expressions as well as proposed equation by the author. The results of this comparison are illustrated in Fig. 4.3. Yost et al. claimed
that theoretical $P_{cr}$ was an important factor in Branson equation and proposed this value to be close to the recorded data from experiments. Referring to Fig. 4.2 the theoretical $P_{cr}$ are fairly close to recorded values, which decreases the effect of $P_{cr}$ on $I_e$. It can be seen in Fig. 4.3 that theoretical and experimental results represent good match for loads levels before crack, while that correlation does not exist for load after this level. Unlike all theoretical equations, ACI 440 relationship overestimated $I_e$ except for seawall slab with relatively large $\rho/\rho_b$. The reason is that ACI recommended equation does not work for low level of reinforcement ratio. On the other hand, adjustments have been made to original Branson equation that were mostly based on Glass, Carbon and Aramid FRP rebar, so results are likely to deviate from those predicted by the existing equations for BFRP-RC beams. A Larger sample sizes with variety in rebar characteristics and reinforcement ratio may lead to a more detailed analysis to give a better understanding of BFRP-RC beams.

Prior to analysis, the load-deflection data from beam tests were compared to those predicted using the existing equation to find out which model works better for Basalt FRP reinforced concrete beams. Amongst all, two models proposed by Bischoff and Al-Sunna indicated better predictions than others. As previously mentioned, Bischoff equation was derived using a different approach, which makes it look completely different from Branson equation. Therefore, it was decided to follow and modify Al-Sunna equation to keep the form similar to what has been common for several years. The Al-Sunna equation contains one fixed and one variable coefficient for each part of the classical Branson equation. The variable coefficient is a function of reinforcement ratio and modulus of
elasticity of FRP rebar. In this study, his equation was modified to the simplest possible form while attempting to predict the best curve for BFRP-RC beams for all values of reinforcement ratio.

![Graph](image)

**Fig. 4.2** (a) Load-Deflection Curves Group 1 - Beam 2B1
Fig. 4.2 (a) Load-Deflection Curves Group 1 (continued) Beam 2B2

Fig. 4.2 (a) Load-Deflection Curves Group 1 (continued) Beam 2B3
Fig. 4.2 (a) Load-Deflection Curves Group 1 (continued) Beam 2B4

Fig. 4.2 (b) Load-Deflection Curves Group 2 – Cap Beam
Fig. 4.2 (b) Load-Deflection Curves Group 2 (continued) – Seawall Slab

Seawall Slab

Deflection, mm

Load, KN

Deflection, in

Load, Kips

Seawall Slab Experiment
Branson’s Eq.
ACI 440, Beta Factor
Yost et al.
Rafi & Nadjai
Al-Sunna 0.95
Bischoff & Scanlon
Proposed Equation

I_{ef}/I_{x}

Load, lb

2B-1

Test
ACI 440
Bischoff
Yost
Al-Sunna
Rafi & Nadjai
Proposed Equation
Fig. 4.3 (a) Theoretical and Experimental Relative Effective Moment of Inertia Group 1- Beam 2B1

Fig. 4.3 (a) Theoretical and Experimental Relative Effective Moment of Inertia Group 1 (continued) - Beam 2B2

Fig. 4.3 (a) Theoretical and Experimental Relative Effective Moment of Inertia Group 1 (continued) - Beam 2B3
Fig. 4.3 (a) Theoretical and Experimental Relative Effective Moment of Inertia Group 1 (continued) - Beam 2B4

Fig. 4.3 (b) Theoretical and Experimental Relative Effective Moment of Inertia Group 2 – Cap Beam
From the load-deflection and moment of inertia graphs shown, Eq. 4.3 and Eq. 4.6 recommended by ACI 440 cannot predict a reliable $I_e$, which is a key parameter for beams deflection computation, for every type of FRP-RC beams and any reinforcement ratio. Rafi and Nadjai compared 120 beams and four slabs to find a realistic relationship for $I_e$. They also mentioned that $I_e$ is mainly influenced by $E_f$ and there is a need to consider it in Branson equation. This has been noticed by other researchers such as Al-Sunna. Therefore, an equation was derived for coefficient $\beta$ of Eq. 4.10 to make it applicable to Basalt FRP rebar. The following general format was assumed for $\beta$ with unknown coefficients a, b and c.

$$\beta = \text{unknown coefficients a, b and c.}$$

(Eq. 4.14)
Each of the above coefficients plays different role in forming the final load-deflection curves; \( a \) shifts the entire curve horizontally, \( b \) and \( c \) influence the concavity of the curve. Also, the other coefficient \( \alpha \), which was decided to be a fixed factor, deals with the beam’s stiffness, causing the upper part of the curve to stand steep or to be gradual.

A nonlinear multiple regression analysis was performed to determine the coefficients. Available sample size was limited to five beams and one slab which made a small sample size. The theoretical calculated deflections were compared with the corresponding recorded deflections of the test specimens at different load levels from 25 to 80 percent of failure loads by applying the proposed equation for \( I_e \), which increased the data points to much larger than six.

The values of \( a, b, c \) and \( \alpha \) were determined by the iterative process of nonlinear regression. This approach works in a way to minimize the squared sum of difference between data and theoretical fit. The accuracy of the iterative process could be represented with the value \( R^2 \) to be as close to unity as possible. The analysis outcome formed the following equation for \( \beta \) and \( I_e \) while \( \alpha \) disappeared as it was equal to one;

\[
\text{—— ———} \quad \text{(Eq. 4.15)}
\]

\[
\text{——} \quad \text{(Eq. 4.16)}
\]
The results of the proposed equation are graphically presented in Fig. 4.2. The graphs show that the derived equation could predict the deflections of BFRP-RC beams reasonably well.

4.6. Discussion

A total of five approaches for the estimation of the effective moment of inertia were considered to determine which one could give an accurate prediction of the deflection of BFRP reinforced beams. Amongst all, the first four relations are based on Branson’s equation with different reduction factors, so the calculations related to the original Branson’s equation are added to results. And the last one is an alternative approach proposed by Bischoff and Scanlon [4.8]. To compare the prediction of theoretical expressions with test results, average ratios were calculated for all four groups at different load levels in range of 25 to 80 percent of the ultimate applied load $P_u$ which are listed in Table 4.3. Besides that, the load curves are graphed which demonstrate the accuracy of different methods in each beam sample.

For groups 3 and 4 at the load levels of 50% and above, most of the ratios for two last columns are fluctuating around unity with average tolerance of -0.002 for Al-Sunna equation as well as +0.053 for Bischoff and Scanlon approach, while the results demonstrates that rest of the relations underestimate the mid-span deflections by about 35%.
Values at the load level of $P_{cr}$ are varying a lot because the calculated $M_{cr}$ may not be exactly what is occurring in the beam tests although ACI-318 relationship for estimating cracking moment has sound basis. On the other hand, as it is clear that there is a big gap between deflection before and after cracking, therefore the ratios vary over a wide range. But, as graphs in Fig. 4.2 demonstrate, the calculated cracking points are close to the experimental results in this study. The statistical analysis of the data points generated by the proposed equation resulted in $R^2=0.983$ which represents a good fit curve for such a small sample size. To refine the equation, a larger number of experimental tests is required.

However, in the range of practical interest (i.e., service load level) which is about 60 to 80 percent of the failure load, the deflections determined by using the proposed equation are in very good agreement with the test results.
Table 4.3. Average Deflection Ratios for each Group

<table>
<thead>
<tr>
<th>Load level %</th>
<th>Branson's β factor</th>
<th>ACI 440 β factor</th>
<th>Rafi &amp; Nadjai</th>
<th>Yost et al.</th>
<th>Al-Sunna α=0.95</th>
<th>Bischoff &amp; Scanlon</th>
<th>Proposed Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group 3 (f'_c=5.90 \text{ ksi}; E_c=4,380 \text{ ksi})</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>0.139</td>
<td>0.139</td>
<td>0.139</td>
<td>0.139</td>
<td>0.139</td>
<td>0.139</td>
<td>0.139</td>
</tr>
<tr>
<td>30</td>
<td>0.079</td>
<td>0.266</td>
<td>0.320</td>
<td>0.268</td>
<td>0.570</td>
<td>0.267</td>
<td>0.519</td>
</tr>
<tr>
<td>40</td>
<td>0.115</td>
<td>0.340</td>
<td>0.425</td>
<td>0.379</td>
<td>0.719</td>
<td>0.793</td>
<td>0.654</td>
</tr>
<tr>
<td>50</td>
<td>0.169</td>
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<td>80</td>
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<td>0.786</td>
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<td>1.196</td>
<td>0.969</td>
</tr>
<tr>
<td>Group 4 (f'_c=6.80 \text{ ksi}; E_c=4,700 \text{ ksi})</td>
<td></td>
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<tr>
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<td>0.448</td>
<td>0.406</td>
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<td>0.448</td>
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<td>0.224</td>
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<td>0.876</td>
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<tr>
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<tr>
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<td>0.610</td>
<td>0.706</td>
<td>1.046</td>
<td>1.040</td>
<td>0.992</td>
</tr>
</tbody>
</table>
4.7. Summary

The experimental test results of Basalt fiber reinforced polymer reinforced beams evaluated with the current theoretical study and statistical analysis, lead to the following conclusions:

1- The current $I_e$ recommended by ACI 440 for FRP-RC beams is not applicable for all types of FRP rebar and all ranges of reinforcement. The ACI 440 equation just works for larger reinforcement ratio.

2- There is no proposed relationship to be able to calculate $I_e$ for Basalt FRP-RC beams, as most of research was carried out on Carbon, Glass and Aramid FRP bars. Although, BFRP bar showed behavior similar to GFRP bars, the available relationships for GFRP reinforced concrete beams could not predict a good fit for Basalt FRP-RC beams.

3- The proposed equation for BFRP showed a close match with the test results particularly in the service load range of practical interest. The nonlinear multiple regression analysis resulted in a reliable value for $R^2$ of 0.983, which represents good accuracy of the proposed method. A simple and easy-to-use equation was established for practical use. The proposed equation could be developed for other types of FRP-RC beams and slabs as well.
CHAPTER V

DURABILITY OF BFRP REINFORCING BARS

5.1. Introduction

Many reinforced concrete structures never achieve their expected service life due to durability issues. A large amount of money has been attributed to repair and maintenance of reinforced concrete infrastructures in many countries [5.1]. Years of research has shown that a feasible solution for steel corrosion problem could be the substitution of FRP reinforcement in reinforced concrete structures. Although, well-known fibrous materials GFRP, CFRP and AFRP had proven their advantages over steel, there is still a shortage of data on FRP bars response and performance in severe environmental conditions [5.2]. Therefore, some design codes like American and Japanese standards had cut down the material strength by applying environmental reduction factors to remain on the safe side in different environmental conditions [5.3, 5.4]. On the other hand, The Canadian standard decided to be more moderate regarding the material strength and tried
to limit use of each FRP in certain environmental condition [5.5, 5.6]. This case would be more complicated when the environmental situation is combined with existence of a sustain load. For instance, ACI 440.1R-03 recommends a coefficient between 0.14 and 0.16 for GFRP [5.3], which clearly prevents designers to use the full capacity of GFRP under sustained loads. Many designers have criticized the coefficients as those are very conservative and are based on results of research on early generation of GFRP, which did not have very good quality. So, more research and studies are required on long-term behavior of FRP. This requirement seems to be even more important for BFRP, because there are no data for this new type of FRP bars. American standard code ACI 440.1R has not mentioned BFRP in its design approaches for short-term and long-term, creep and relaxation behavior. In this chapter, the creep behavior of BFRP rebar in alkaline environment is presented based on ACI 440.3R recommendation and ASTM standard testing method to determine creep strength and capacity of FRP bars.

Creep is defined as the progressive deformation of a material under sustained load with time. The creep behavior of material differs significantly from their metallic counterparts. The viscoelastic response of the polymeric resin and their temperature sensitivity render an FRP material more sensitive to creep and other rate dependent phenomena. For instance, most of materials start exhibiting significant creep when a substantial amount of load is imposed at a temperature exceeding the 40% of their melting temperature [5.7]. The behavior of FRP composites under sustained loading tends to be more complex as the degradation of material could depend on fiber and resin as well as their interface bond properties. [5.8]. However, it is well accepted that for most practical civil infrastructure
applications, creep properties are dominated by resin dependent properties rather than interfacial properties of fiber [5.9].

To characterize the creep behavior of a material in an accelerated manner, a constant load would be applied under controlled temperature and elongation of the material may be determined as a function of time. However, based on the limited research conducted in this particular field, it is worth mentioning that accelerated environmental conditioning can significantly affect the governing dynamics of creep. Moreover, it should be noted that the FRP material has always been used in conjunction with concrete to serve different structural purposes. When they are applied as internal reinforcement, there is evidence that the members are primarily subjected to bending stresses. Environmental conditioning can lower the stiffness of the FRP bars and bond between FRP and concrete, thus subsequently reducing the post-cracking stiffness of the beams [5.10]. The intrinsically lower modulus of elasticity associated with FRP materials may also play a pivotal role for the overall creep characterization.

Apart from the structural issues, it is well accepted that the manufacturing parameters also govern the creep behavior of the polymeric materials. It has been shown that curing conditions can affect the creep properties of thermoset polymers [5.11]. Based on the studies done by Bradley [5.12], it is reported that vinyl esters that were cured at room temperature had greater creep exponent (i.e. lower creep resistance) than the vinyl esters that were post cured at 200°F (93°C) [5.11].

Two primary points to be noted for creep are:
1) Creep Strain under long term load
2) Tensile strength under sustained load [5.7]

5.1.1. Concepts of Creep
A typical creep history of a FRP material consists of three different sections. After the initial elastic strain, there is a primary creep region where the creep strain grows faster with time. This will be followed by a secondary creep region with almost constant slope extending over a long period of time. This region is particularly important for analysis, as the structures will be serviceable in this region [5.13]. The tertiary region is associated with the higher level of stresses and represents the simultaneous accumulation of creep strain with time along with observable material damage. The schematic diagram in Fig. 5.1 shows three regions.

The creep response of most polymers could be estimated with a power law model as given by [5.11]:

![Fig. 5.1 Typical Creep Strain vs Time](image_url)
where

\[ \begin{align*}
\epsilon &= \text{creep} \\
\sigma &= \text{applied stress} \\
T &= \text{temperature} \\
M &= \text{material dependent factor} \\
\tau &= \text{time}
\end{align*} \]

A correlation equation also was developed which relates the creep strain and time by the following equation [5.11]:

(Eq. 5.2)

where

\[ \begin{align*}
\epsilon_0 &= \text{initial strain} \\
A &= \text{coefficient}
\end{align*} \]

If a curve is plotted with strain versus logarithm of time axis, it was found that most FRP materials approximate to a linear relationship [5.14]. The total strain in the material at any time ‘t’ then can be expressed as

(Eq. 5.3)
where $\beta$ factor is the creep rate parameter, which is the instantaneous slope of the total strain-time curve.

Results from experimental programs performed on small FRP bars made of GFRP, AFRP and CFRP, showed that there is a linear relationship between creep rupture strength and the logarithm of time, for intervals up to 100 hours [5.15].

As per ACI 440.1, Glass, Aramid and Carbon fibers can sustain 0.30, 0.47 and 0.91 times their ultimate strengths, respectively [5.3]. Creep strain for CFRP, at room temperature (around 20°C) and normal humidity, remains under 0.01% after 3000 hours at 80% load level (80% of ultimate strength). It was 0.15% to 1% for AFRP and 0.3% to 1% for GFRP at the similar conditions [5.16].

5.2. Various Experimental Approaches adopted by Researchers

Prior to planning the accelerated creep rupture test, it was important to select a suitable testing approach from a pool of methods proposed by various researchers. The major components of testing were the loading machine itself, and the method of alkaline solution circulation around the BFRP bars specimens.

5.2.1. Loading machines for Creep Rupture Test

There were many approaches and machines used by different researchers. A selected number of test methods are discussed in the following sections:
5.2.1.1. Sen, Mullins and Salem, 2002 [5.17]

Sen et al. approach was based on ACI 440.3R, section B.6, which is accelerated test method to study alkali resistance of GFRP bars. This section recommends three procedures as follows:

1- Procedure A-A: FRP bars immersed in Alkaline solution without any load
2- Procedure B-A: FRP bars immersed in Alkaline solution under sustained load
3- Procedure C-A: FRP bars surrounded by moist concrete subjected to sustained load

The simple frame used in the study was capable to apply controlled load on the specimen. As it is clear in Fig. 5.2, amount of load could be adjusted with the help of bolts and the attached load cell may lead to control the required load on each test specimen. As a requirement of the ACI manual, all tests carried out in 140°F (60°C). Also, Fig. 5.3 demonstrates the final set up after all loads were applied on each specimen. One of the advantages of this approach was that it did not need too much laboratory space.
Fig. 5. 2 Set up Arrangement [5.17]

Fig. 5. 3 Durability Test Setup [5.17]
5.2.1.2. Nkurunziza, Benmokrane, Debaiky, and Masmoudi, 2005 [5.18]

Nkurunziza et al. studied creep behavior of GFRP bars in different environmental conditions. They applied loads at two levels, 25% and 38% of ultimate strength of the bars and exposed them to alkaline solution and iodized water at room temperature. Loading frame shown in Fig. 5.4 was an intelligent way to apply the force with significantly smaller amount of loads. In this method, the frame would magnify the amount of load with a simple mechanism. A tubular PVC reservoir was attached on free length of test specimen to let the GFRP bar to be exposed to the solutions. Also electrical strain gages were installed at the mid length of the bars. This magnifying frame seems to be very useful when the tensile specimen need to experience higher levels of loads.
Fig. 5. 4 Testing Frame Sketch [5.18]
Fig. 5. 5 Loaded Specimens and Testing Frames [5.18]
5.2.1.3. Mukherjee and Arwikar, 2005 [5.19]

This research group performed their experimental work in a different way. They did not test the GFRP bars directly by tensile test, but decided to load entire concrete beam reinforced with GFRP bars. As it is clear in Fig 5.6, two beams were held together by end brackets while a pair of steel spring, with specific stiffness, keeps the beams away from each other. This set up works like a 4-point loading. Twisting the nuts on both ends makes beams to crack and therefore bars would be exposed, while maintaining controlled amount of loading on beams and consequently on GFRP bars. After formation of cracks, they were submerged in hot water.

![Fig. 5.6 Test Setup [5.19]](image)

5.2.1.4. Almusallam and Al-Salloum, 2005 [5.20]

One of previously mentioned procedures of ACI 440.3R was that the FRP bar may embed in wet concrete under sustained load. Almusallam and Al-Salloum fabricated special reinforced concrete beams. The GFRP reinforcement was covered by high alkali cement at center before the rest of concrete was placed. The beams were then loaded in a
way that GFRP bars experience 20 to 25 percent of their ultimate tensile capacity. The sustained load was held on the beam for time periods of 4, 8 and 16 months.

Fig. 5.7 Form Works [5.20]
Fig. 5.8 Beam Detail [5.20]
5.2.1.5. Benmokrane, Wang, Ton-That, Rahman and Robert, 2002 [5.21]

Benmokrane et al. reported three types of tests in their work. Each type of tests was done with a different set-up. One frame was similar to Nkurunziza et al. [5.18] magnifying frame. The other loading system was a self-supporting tool made of a steel cylinder and a steel spring that was arranged in a way to impose a load on the GFRP bar specimen by fastening a bolt. They applied load levels in range of 22 to 68 percent, and exposed to three different alkaline solution, NAOH, simulated pore-water, and moist concrete. Figs. 5.9 and 5.10 show two types of set-up.

Fig. 5.9 Test Setup 1 [5.21]
5.3. Prediction Models of Long-term Performance of FRP bars

The accelerating technique used for ageing test is found on Arrhenius principle, which assigns high temperature as accelerating factor in different environments. The Arrhenius empirical model is one of the most accepted and used method by researchers to predict the failure time of FRP bars as a function of temperature as it is clear in the following equation (Eq. 5.4):
5.2.2. Analysis Methodology

The tests performed in this study were in accordance with the test methods recommended by ACI 440.3R-04 and ASTM standard [5.22] approach for creep rupture of FRP reinforcing bars. Unlike steel rebars, creep capacity of FRP bars is significantly less than static tensile strength (ACI 440). Therefore, it is very important to determine creep rupture capacity of FRP bars to design for sustained load effects. ACI 440 has recommended to load FRP bars to different load levels varying between 20 and 80 percent of tensile strength. It is required to have rupturing time representing three decades of time. All recorded data points from the tests are drawn on a semi-logarithmic graph with stress level on vertical axis and logarithm of time on horizontal axis. A best-fit linear curve is drawn, by least square method, to form the predicting line similar to the following equation:

\[
Y_c = a_1 + b_1 \log T
\]

where,

\( Y_c \) = load level (%)

\( a_1, b_1 \) = empirical coefficients

\( T \) = Time (hr)
By extending the plotted line, it is possible to find the load level corresponding to the one million hours, equal to 114 years. This load is the million-hour creep capacity of that type of FRP bars and the corresponding stress is million-hour creep strength, which is calculated with Eq. 5.6 as per ACI 440.3R.

\[(\text{Eq. 5.6})\]

where,

\[f_r = \text{million-hour creep rupture strength}\]
\[F_r = \text{million-hour creep rupture capacity}\]
\[A = \text{Cross-sectional area of FRP bars}\]

On the other hand, because of applying an accelerated method in this study, it was required to convert the rupture time at accelerated condition (simulated alkali solution and elevated temperature of 140°F) to the time of rupture in normal situation. There is a pool of convertor relations in the published literature for different materials such as Carbon and Glass FRP bars. The accelerating technique used for ageing test is found on Arrhenius principle, which assigns high temperature as accelerating factor in different environments. The Arrhenius empirical model is one of the most accepted models and used by researchers to predict the failure time of FRP bars as a function of temperature as it is clear in the following form [5.1]
where,

\[ t \]
= real failure time

\[ T \]
= exposure temperature (°K)

\[ B \]
= the Boltzmann constant \((8.617 \times 10^{-5}/\text{K})\)

\[ a \]
= acceleration factor

\[ e \]
= activation energy factor (varies between 0.3 and 1.5)

Vijay and Gangarao [5.23] developed a converting formula to correlate the accelerated ageing test to the natural ageing for GFRP rebar. Their equation was derived based on the Northeastern US climate, especially West Virginia State.

(Eq. 5.7)

where,

\[ \tau \]
= natural age (days)
Another equation was proposed by Porter and Barnes [5.24] which was based on experimental work performed in the State of Iowa, US:

\[ T = 87.32 + 0.021 E - 0.00004 E^2 \]  

(Eq. 5.8)

where,

\[ T = \text{exposure temperature (°F)} \]

\[ E = \text{exposure time (days)} \]

Also, a set of useful coefficients for GFRP has been proposed by Chen et al. [5.25] for different exposure temperatures based on a new method of testing. Regarding the fact that there is not a well-established convertor for BFRP bars, an accelerating factor for GFRP bars proposed by Chen et al. was used to convert the conditioned time to normal time. The GFRP bars they used were of size #3 bars made of E-glass fibers and vinyl ester resin. Outside surface was slightly sand coated and helically wrapped. Moreover, the bars had fiber content of more than 70% by weight. The proposed coefficients for different conditions are listed in Table 5.1.
Table 5.1. Acceleration Factors

<table>
<thead>
<tr>
<th>Temperature °F (°C)</th>
<th>GFRP in Alkaline Solution</th>
</tr>
</thead>
<tbody>
<tr>
<td>140 (60)</td>
<td>1.8</td>
</tr>
<tr>
<td>104 (40)</td>
<td>1.28</td>
</tr>
<tr>
<td>68 (20)</td>
<td>1.00</td>
</tr>
</tbody>
</table>

5.4. Experimental Program

In the current study, fifteen accelerated creep tests were conducted on BFRP bars. BFRP bars were subjected to load levels between 20 to 80% of their average ultimate tensile strength, which was determined in chapter III, under simulated alkaline environment at an elevated temperature of 140°F (60°C). Alkaline solution was prepared as per ACI 440.3R-06 [5.3] recommendation.

5.4.1. Materials

The creep tests in this section were conducted on one size of BFRP reinforcing bars, R4. These bars, as introduced in chapter III, were manufactured by a new approach known as wet-layup method, which is believed to be a cost effective approach compared to pultrusion. The bars were helically wrapped and the outside surface slightly coated with sand. The R4-BFRP bar geometrical properties such as cross sectional area $A_r$ and volume fraction $V_r$ are presented in Table 2.3 (from Chapter II). Also, values in Table (Chapter III) show that tensile strength of R4 bars tolerated between 156 and 182 ksi.
(1,077 to 1,255 MPa) with an average of 163.8 ksi (1,148 MPa), while modulus of elasticity varied from 6,000 (40 GPa) to 8,000 ksi (55 GPa) with an average of 6,488 ksi (4,473 MPa).

5.4.2. Sustained Loading Frame

There is currently no standard testing device for creep rupture test. Therefore, the researchers in the area tried to explore different methods. The loading frames used by Nkurunziza et al. [5.18] and Benmokrane et al. [5.21] tend to be the best choice for developing a sustained load on test specimens (Fig 5.4 and 5.9).

Based on these two test frame designs, a pair of steel frames were designed and fabricated at The University of Akron. The frames were designed to be stronger than the required loading to be applicable for future studies. Figs. 5.10 and 5.11 show general overview and details of the frames. The frames had two parallel magnifying arms, which help to reach the required load on tensile specimens by loading small weights on designated section. This frame was able to magnify the load by 40 times the applied load when both arms remain horizontal. A bearing was welded to the top of the column, where the larger arm sits on the column, to let it pivot freely for accurate set up. There was a turnbuckle installed at the end of each frame to make simple adjustments. It was possible to set the arms to remain horizontal by twisting the turnbuckle. Standing angle of arms were sensitive due to magnifying characteristic of the frame. Therefore, in addition to
turnbuckles on end, a threaded rod with two bolts was installed on other side of each frame under the specimen for adjustment. This arrangement assisted in easy control of the level and angle of arms by twisting these two adjustment tools. The frame without any load was imposing equivalent load of 500 pounds on the specimen when both arm remain horizontal. It was important to consider this a considerable load during the actual loading of the test specimens. Each frame was fabricated separately, and then two frames were coupled with each other for the sake of stability. Two X-braces were used in horizontal and vertical directions to connect the frames together. Most of the connections were achieved with bolts for ease of attaching and detaching the parts.
Fig. 5.11 Loading Frame (Perspective)
As per ASTM standard D7337 [5.22] recommendation, the loading process takes between 20 seconds to 5 minutes, then the creep time should be measured from the moment that specimen is fully loaded. The turn buckle was provided at the end of the frame for load adjustments. Fig. 5.13 shows the adjustment in progress by author.
Fig. 5.13 Load Adjustment of Test Specimen
5.4.3. Simulated Concrete Environment (Alkaline Solution)

To simulate the alkali environment of concrete, a composition recommended by ACI 440.3R-04 was used to make a solution with a pH of 12.6 to 13. The alkaline solution contained 118.5 g of Ca(OH)$_2$, 0.9 g of NaOH and 4.2 g of KOH in one liter of deionized water. Plastic transparent tubes, acting as reservoirs, were used to expose the central length of BFRP tensile specimens to alkaline solution. In addition, an elevated temperature of 140°F (60°C) was required to expedite the progress of alkali effects on BFRP bars. Regarding the fact that the applied tube, as reservoir, had a relatively small diameter, it was not easy to heat up and control the temperature of the content of the reservoirs directly. Therefore, a customized set up was designed to heat up the alkaline solution externally prior to pumping into reservoirs. For this reason, an electronic water bath was used to heat up the water to 140°F. Then two plastic cylinders were placed in the water bath filled up with alkaline solution. Finally, submersible pumps were fixed inside each cylinder to circulate the heated alkaline solution through hoses around the test specimens. Returning hoses were completing the circulation process by leading the solution back to the plastic cylinders. Cylinders were sealed to avoid any interaction with the atmospheric CO$_2$ and prevented any evaporation. In addition, the water bath was covered with plastic wrap to prevent any water evaporation (Fig 5.14).
Fig. 5. 14 Water Bath (Heater) and Cylinders with Circulating Pumps
5.5. Results

The specimens were loaded with different load levels until their rupture. The strain and time to rupture of each specimen were recorded in order to plot the required graphs. Some of the specimens failed prematurely due to slipping of BFRP bar, which might have happened because of shorter time for curing of filler epoxy compared to other specimens. It is worth mentioning that probably there were some relaxation in anchorages during loading similar to what other researchers observed [5.26], which actually had no effect on the test results because of adjustability of the test frame. The results of the creep rupture tests are tabulated in Table 5.2. There were two specimens, which slipped before any rupture and one specimen with 25% load which did not fail during 3 months of exposure after which the test was terminated. Fig. 5.15 shows two set of data points, one is the actual rupture time and another is factored time based on the available coefficient for GFRP. Logarithmic trend lines were drawn and extrapolated to the point of 1,000,000 hours (114 years). Fig. 5.16 was plotted to an enlarged scale within the range of practiced interest.
Table 5.2. Creep Rupture Test Results

<table>
<thead>
<tr>
<th>Load</th>
<th>Load Level</th>
<th>Exposed Time</th>
<th>Converted Time</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>lb</td>
<td>%</td>
<td>Hours</td>
<td>Hours</td>
<td></td>
</tr>
<tr>
<td>2960</td>
<td>80</td>
<td>0.1</td>
<td>0.18</td>
<td></td>
</tr>
<tr>
<td>2775</td>
<td>75</td>
<td>2.1</td>
<td>3.78</td>
<td></td>
</tr>
<tr>
<td>2590</td>
<td>70</td>
<td>2.3</td>
<td>4.14</td>
<td></td>
</tr>
<tr>
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<td>96</td>
<td>172.8</td>
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<td>160.2</td>
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<td>Premature Failure</td>
</tr>
<tr>
<td>1924</td>
<td>52</td>
<td>160</td>
<td>288</td>
<td></td>
</tr>
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<td>50</td>
<td>80</td>
<td>144</td>
<td></td>
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<tr>
<td>1665</td>
<td>45</td>
<td>143</td>
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<tr>
<td>1591</td>
<td>43</td>
<td>7*</td>
<td>12.6*</td>
<td>Premature Failure</td>
</tr>
<tr>
<td>1591</td>
<td>43</td>
<td>149</td>
<td>268.2</td>
<td></td>
</tr>
<tr>
<td>925</td>
<td>25</td>
<td>2160</td>
<td>3888</td>
<td>Did not Failure</td>
</tr>
</tbody>
</table>

Note: * Outlier
Fig. 5.15 Creep Rupture Curves for Basalt FRP R4 Bars
The projected durability for BFRP bars was obtained from Fig. 5.16 and summarized in Table 5.3.

Table 5.3 Projected Durability of BFRP Bars

<table>
<thead>
<tr>
<th>Service Life (years)</th>
<th>Creep Rupture Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>0.27</td>
</tr>
<tr>
<td>10</td>
<td>0.25</td>
</tr>
<tr>
<td>20</td>
<td>0.22</td>
</tr>
<tr>
<td>50</td>
<td>0.17</td>
</tr>
<tr>
<td>100 or greater</td>
<td>0.14</td>
</tr>
</tbody>
</table>
Also, the creep coefficient versus time is shown in Fig. 5.17. The creep coefficient $\phi_{pt}$ is the ratio of plastic strain, under a sustained load, to elastic strain of FRP bars. This ratio was calculated for various load levels with different creep rupture time [5.27]. Elastic strain used for the calculation was obtained from mechanical properties tests, which was presented in chapter III. Therefore, the elastic strain at each level was load level coefficient times average rupture strain. In addition, the plastic strain was measured from creep rupture test.

\[
\text{---} \quad \text{(Eq. 5.10)}
\]

Based on logarithmic best-fit line, the following equation was derived for creep rupture coefficient, which could give a prediction for one million hour creep coefficient. This equation was derived based on a very small population in this study and therefore, having more data points in the regression analysis may lead to a more reliable equation.

\[
\text{---} \quad \text{(Eq. 5.11)}
\]
5.6. Summary

Creep rupture tests were performed on R4 BFRP reinforcing bars at different load levels between 25 and 80 percent under environmental conditions of alkali solution with PH of about 13 and elevated temperature of 140°F (60°C). Alkali environmental and elevated temperature were used to simulate concrete environment and expedite the degradation process. Test results are presented in tables and graphs to determine creep rupture strength and strains for different life spans. According to the graph, extended lines for actual exposure and converted times intersect the point of one million hours at 11 and 15
percent respectively. By finding the million-hour creep rupture capacity (load level), it is possible to calculate million-hour creep rupture strength with Eq. 5.6.

The creep strength and capacity presented above were calculated for 114 years of structure’s life, which is very conservative. These coefficients significantly reduce the capacity of BFRP bars under sustained load. Therefore, creep reduction factors for different ages like 5, 20 and 50 years are shown in Figs. 5.15 and 5.16 and Table 5.3 which gives a more realistic coefficients based on different design ages for various purposes.

In addition to creep strength and capacity, the one million hours (114 years) predicted creep coefficient of BFRP R4 reinforcing bar is shown in Fig. 5.17. Unlike other graph (Figs. 5.14 and 5.15), the creep coefficient prediction line has a positive slope, which means the creep coefficient increases as the time passes. Based on extrapolated logarithmic line, the one million hours creep coefficient for BFRP R4 bars tends to be about 13%.
6.1. Introduction and Basic Definition

In this chapter an application of BFRP bars is evaluated in order to investigate the feasibility of practical applications of BFRP bars in reinforced concrete.

6.1.1. Seawall Slab

Seawall is a type of retaining wall, that is used in coastal areas to stabilize the soil and prevent its movement. A large number of concrete seawalls could be found in the State of Florida. Generally, most of the walls are reinforced with steel reinforcing bars, which are highly susceptible to corrosion especially when exposed to seawater and hot temperature of Florida. Fig. 6.1 shows a typical concrete seawall with a cap beam at the top. Typical
details of an anchored seawall panel system currently used by Collier Seawall & Dock LLC, in Florida consist of precast slabs embedded in seabed at the bottom. The top edge of the precast walls is embedded along the long direction within a reinforced concrete cap beam that is tied back at an approximate spacing of 10 feet with tie-back rods. The role of the cap beam is to provide support to the seawall at the top end and also to increase rigidity of the wall in the long direction. The end of each tie-back rod is anchored to a reinforced concrete deadman (not seen in the picture). Each unit of the seawall system is reinforced with steel reinforcing bars. A schematic of the seawall structure with all its components is shown in Fig. 6.2.

![Fig. 6.1 Typical Seawall](image)

This case study provides an alternative design of seawall system using BFRP reinforcing bars. The design parameters for the design of BFRP reinforced concrete seawalls were developed in this study. The design is based on the design strength of the individual steel
reinforced member. Alternative design is suggested for each component of the system with identical dimensions, but with BFRP reinforcing bars in place of steel reinforcing bars. Typical steel reinforced seawall slab and an alternative new design with BFRP bars are shown in Fig. 6.3 and Fig 6.4 respectively.

The following four members/elements are considered in the seawall system:

(a) Seawall slab
(b) Cap beam
(c) Deadman slab
(d) Tie-back rod
Fig. 6.2 Typical Seawall Used in Florida
Fig. 6.3 Details of a Typical Steel Reinforced Seawall Slab
Fig. 6.4 Proposed Details of a Typical Basalt Fiber Bar Reinforced Seawall Slab
6.1.2. Cap Beam

Typical details of a steel reinforced concrete cap beam are shown in Fig. 6.5. The beam section is provided with four #5 steel reinforcing bars with two bars placed at the top and two bars placed at the bottom. The longitudinal bars are tied with #3 ties at 12 inches on center. The cap section is 24 inches deep and 24 inches wide as shown in the figure.

Calculations similar to those developed for the seawall slab in the previous section were carried out to detail a cap beam using BFRP reinforcing bars. The longitudinal bars needed to be 0.4 inch (10.5 mm) diameter (net fiber diameter) bars with the arrangement of bars as in the steel reinforced concrete beams. The ties are tentatively suggested to be made from bars provided at the same spacing as in the steel reinforced cap beam. The suggested details are shown in Fig. 6.6. The area of steel bars being used in the current steel reinforced cap beams is below the ACI 318 specified limit for minimum area of steel. The BFRP reinforcing bars are designed to provide about the same moment strength as the steel reinforced cap beams.
Fig. 6.5 Typical Details of Steel Reinforced Concrete Cap Beam

- 2'-0" (610 mm) width
- (4) #5 Rebar
  - Lap 24" (610 mm) Min.
- 3/4" (19 mm) Chamfer (Typ.)
- 3/4" (19 mm) Chamfer (Typ.)
- #3 Ties @ 12" (305 mm) O.C.
  - Place Ties 6" (152 mm) Each Side of Tie Back Rod
- #8 HDG Steel Tie-Back Rod (Typ.)
- 12" (305 mm) Hook Shall Wrap Slab Steel
- Wall Slab

Fig. 6.6 Proposed Details of a BFRP Reinforced Concrete Cap Beam

- 2'-0" (610 mm) width
- (4) R10 BFRP Bars
  - Lap 36" (915 mm) Min.
- 3/4" (19 mm) Chamfer (Typ.)
- 0.2 (5 mm) Diameter Ties @ 12" (305 mm) O.C.
  - Place Ties 6" (152 mm) Each Side of Tie Back Rod
- Tie-Back Rod 1" (25 mm) Diameter BFRP Bar (Typ.)
- Wall Slab
6.1.3. Deadman Slab

Currently, each 3’ x 3’ x 1’ deadman block is reinforced with steel reinforcing bars comprising #5 bars @ 12 inch O.C. both ways. The design of deadman with BFRP reinforced bars was developed based on ACI 440.01-06 report similar to the designs presented in the earlier sections of this report. The basalt fiber reinforcing bar of 10.5 mm (0.41 inch) diameter (net fiber diameter) will provide the required design moment strength for the deadman.

6.1.4. Tie-Back Rods

At this stage, a 0.5 inch (12.5 mm) diameter (net fiber diameter) bar is expected to do the job of the tie-back rod.

6.2. Test Program for Evaluation

A test program was developed to load test the components (members) of the seawall system. Prior to the test set up, a set of finite element analysis was performed on the wall to study the behavior of the seawall and to predict the failure load for the test specimens. Therefore, this chapter includes two steps;

(1) Finite Element Analysis
(2) Experimental Tests
6.2.1. Finite Element Analysis

The numerical analyses were performed using Plaxis 2D v.9.0, a commercial finite element analysis software. It is possible to run plane strain analyses in plane strain conditions with this program. The domain and mesh used for modeling is shown in Fig. 6.7. The boundary conditions are specified to model the site condition. In addition, different water levels were applied to find the worst case scenario.

There are several ways to model the soil behavior. The simplest form for modeling stress-strain relationship of soil is Hooke’s law, which needs just two parameters; modulus of elasticity $E$ and Poisson’s ratio $\nu$. However, this model is too basic to be able to give a realistic simulation. With the use of other models, the Mohr-Coulomb is the first one that is mostly used for initial study of soil behavior. Mohr-Coulomb model can simulate soil
as elastic-perfectly plastic material. To do this, it considers five soil parameters which are modulus of elasticity $E$, Poisson’s ratio $\nu$, Cohesion $C$, friction angle $\varphi$ and finally angle of dilatancy $\psi$. The first two parameters are for soil elasticity, and the second pair are for modeling the plasticity of soil. Other mentioned methods are more advanced approaches and they apply different soil parameters. For example, Hardening Soil model considers three different stiffness input values, which cause a more accurate representation of soil stiffness. For a preliminary study of the soil behavior, it is recommended to use Mohr-Coulomb approach because the analysis time is shorter and the level of accuracy is acceptable at this stage.

Therefore, the soil behavior was modeled by Mohr-Coulomb model in this study and the applied soil parameters are tabulated in Table 6.1. The model was meshed using planar 15-node triangular elements, and then the mesh was refined near the seawall for the sake of increasing the level of accuracy stress and strain.

<table>
<thead>
<tr>
<th>$E$</th>
<th>$\nu$</th>
<th>$c$</th>
<th>$\varphi$</th>
<th>$\psi$</th>
<th>$\gamma_{\text{sat}}$</th>
<th>$\gamma_{\text{unsat}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>lb/ft$^2$</td>
<td>0.2</td>
<td>0</td>
<td>35</td>
<td>2</td>
<td>150</td>
<td>130</td>
</tr>
</tbody>
</table>

A plate element was used to model the seawall, by entering the normal stiffness ($EA$) and flexural rigidity ($EI$) of the wall, PLAXIS software automatically calculates the equivalent thickness ($d_{eq}$). The Poisson’s ratio of concrete considered as zero. The plate was modeled with its total length of 16 feet while the lower length of 6 feet was
embedded in soil and the upper length was in contact with soil on one side and water on the other. Soil and structure interaction was also considered in the analysis by using Interface feature.

The tie back was also modeled using a Geogrid element with the axial stiffness of the Basalt FRP bar. This element was modeled as tension only element, and cannot handle any compressive forces. The tie back was restrained to a fix-end anchor, which represents the deadman at the site.

Typical shear force and bending moment diagrams of the seawall slab are represented in Fig 6.8 and Fig. 6.9. The maximum bending moment occurred within the upper part of the wall. To perform the experimental tests, it was decided to use a 10 ft long slab between the points of zero moment instead of full length.
Fig. 6.8 Shear Force Development Along the Seawall

Fig. 6.9 Bending Moment Development Along the Seawall
To validate the results of the finite element model, a simple hand calculation were done using classic method of free earth support. There are several classic methods for designing the anchored sheet-pile walls. Two of them are more frequently used: Free Earth Support and Fixed Earth Support. In the first method, it is assumed that toe of the wall is free (simply supported) and it is required to have a minimum depth of penetration and no pivot point exists below the dredge line. The pivot point is the point that separates active and passive pressure zone along the wall. On the other hand, assumption of the second method is that toe of the wall is fixed. Based on penetration depth and consequently earth pressure distribution, it could be determined which of the two methods is more realistic for design. In these set of calculations, it is assumed that the wall is rigid and the stiffness of sheet-pile wall is not considered as a parameter for the sake of simplicity. While the flexibility of wall changes the deflection and bending moment distribution along the wall. [6.1, 6.2]

Some researchers used finite element methods to analyze sheet-pile walls to study the effect of wall’s stiffness. The results for different walls stiffness shown in Fig. 6.10 are reported by Potts and Fourie [6.3]. The diagrams show how the wall’s stiffness affects deflections and bending moment of the seawall. It is clear that by decreasing the wall’s stiffness, deflection increases and maximum bending moment decreases. Also, the bending moment diagram shows double curvature for sheet-pile wall, which is the result of soil and structure interaction during transition part when passive pressure becomes active pressure under the drainage line.
Fig. 6.10 Bending Moment and Wall Movement for Different Stiffness of Retaining Wall [6.3]

Therefore, along with the hand calculations (for a rigid wall) a sheet-pile wall with high stiffness was also modeled and analyzed with PLAXIS 2D to compare the results of all three models together. Fig 6.11 demonstrates this comparison. As it was expected, the wall model with higher stiffness shows similar behavior to rigid wall (from hand calculation). While the seawall model with actual stiffness contains a point of inflection and an extra point of zero moment in the embedded zone which might be assumed as a simply supported boundary condition. The maximum bending moment occurs around 5 feet from the top of the seawall.
6.2.2. Experimental Tests

According to the results of the finite element analysis, the experimental tests were designed to represent the structural behavior of seawall slabs and cap beams. Therefore, referring to bending moment diagram in Fig 6.11, because the maximum bending moment located in 5 feet from top, it was decided to test a 1 foot wide 6 inch thick seawall slabs of 10 feet length over a span of 8 feet. The loading condition simulates about the same maximum bending moment and maximum shear force as the actual seawall within the expected water level variation due to low tides and high tides. Two slabs, one with steel reinforcing bars, and another with equivalent BFRP bars were tested.
Two cap beams were also tested. These beams were 12 inch wide, 16 inch deep and 10 feet long. The beams were tested over a span of 8 feet simulating the actual span of the cap beams as shown in one of the structural drawings of the seawall system commonly used by the sponsor. Two beams were tested, one with steel reinforcing bars as shown in the drawing, and one with equivalent BFRP bars.

These tests are representative of the seawall slabs and cap beams. It was found that the design details of the seawall slabs shown in this chapter will work to achieve the required performance of the slab as intended in the original designs.

However, an optimized design as suggested in this chapter with components reinforced with BFRP bars need to be tested in full scale for further design optimization. Alternatives to the lifting hooks which are currently 1/2 inch diameter steel pick-up strands with 270 ksi strength were also designed but not used.

6.3. Experimental Procedure

6.3.1. Seawall Slabs

Two of the test specimens are representative of the seawall slab with 1 foot (305 mm) width (instead of actual 5 or 6 feet width (1525 or 1829 mm)) of 6” (152.4 mm) thick seawall slab. These slabs were tested over 8 feet (2,440 mm) span. The dimensions and span lengths used for the small scale test specimens will result in about the same maximum bending moment (7.44 kip-ft per foot width) and maximum shear force (1.7 kips per foot width) as the actual seawall within the expected variation of low tide and
high tide water levels. Two slab specimens were tested, one with steel reinforcing bars, and the other with equivalent BFRP bars. The details of the test slab specimens are shown in Fig. 6.12.

6.3.2. Cap Beams
Two of the test specimens are representative of cap beams. The cross-sectional dimensions of the specimens are 1 foot wide x 1’-4” deep (305 mm x 406 mm). The total length of each cap beam tested is 10 feet (3050 mm). The design calculations showed that the section used in the small scale test program is able to adequately carry the bending moments and shear forces resulting from the loads acting in the seawall system. Two test specimens were tested, one with steel reinforcing bars and one with equivalent BFRP bars.
bars, which is also shown in Fig. 6.13. Section dimensions and reinforcement details for all seawalls and cap beams are listed in Table 6.2.

![Steel Reinforced Beam](image1)

![Basalt Reinforced Beam](image2)

**Fig. 6.13 Typical Reinforcement Details and Cross-Section of Beams**

**Table 6.2. Reinforcement Details of the Test Beams and Slabs**

<table>
<thead>
<tr>
<th>member #</th>
<th>$f'_{c},$ psi</th>
<th>$f'_{c},$ MPa</th>
<th>Steel Bars</th>
<th>Basalt Bars</th>
<th>$A_{s},$ in$^2$</th>
<th>$A_{ft},$ in$^2$</th>
<th>Reinforcement Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SWSS</td>
<td>7,065</td>
<td>48.7</td>
<td>2 #5</td>
<td></td>
<td>0.6136</td>
<td>0.01704</td>
<td></td>
</tr>
<tr>
<td>SWSB</td>
<td>7,065</td>
<td>48.7</td>
<td>4 R10</td>
<td></td>
<td>0.4688</td>
<td>0.00977</td>
<td></td>
</tr>
<tr>
<td>Beam</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SWBS</td>
<td>7,065</td>
<td>48.7</td>
<td>2 #5</td>
<td></td>
<td>0.6136</td>
<td>0.00393</td>
<td></td>
</tr>
<tr>
<td>SWBB</td>
<td>7,065</td>
<td>48.7</td>
<td>4 R10</td>
<td></td>
<td>0.4688</td>
<td>0.00301</td>
<td></td>
</tr>
</tbody>
</table>
6.3.3. Materials

Concrete was supplied by a local ready mix concrete supplier. The 28 day target strength was 5,500 psi (37.92 MPa). Steel reinforcing bars were supplied by a local steel reinforcement distributor (Akron Rebar).

BFRP Reinforcing bars were supplied by the sponsor and properties of the bars were as given in Chapter II.

6.3.4. Fabrication of Test Specimens

The formwork for the beams was fabricated in the materials laboratory. The concrete was carefully placed in the forms, consolidated with a needle vibrator, and finished manually. The wet concrete was allowed to harden for 24 hours within the forms after which the forms were removed. The beams were wrapped with wet burlap and plastic sheets to prevent drying of the test specimens soon after de-molding. Fig. 6.14 shows the formwork for four test specimens and Fig. 6.15 shows the pouring of the concrete in the forms. Fig. 6.16 shows the test specimens after they were poured and finished.
Fig. 6.14 Wooden molds prepared for Seawalls and Cap Beams

Fig. 6.15 Placement Concrete
6.3.5. Test Set-up and Instrumentation

The seawall slab tests and the cap beam tests were performed in a test frame with approximately 300 kip (1,335 kN) capacity. Typical test setup is shown in Fig. 6.17 for the slab tests and Fig. 6.18 for cap beam tests. The actuator used for the tests has a capacity of 55 kips (245 kN). Strain gages were attached to the reinforcing bars prior to the placement of concrete. These strain gages recorded the strains developed in the steel reinforcing bars and BFRP bars during testing. Strain gages were also attached to the concrete surface at the top of test slabs and beams to record the concrete strains at the top surface of the beams. Deflections were measured with linear variable differential transducer (LVDT) at the mid-span of the test beams during the initial part of the loading, after which the deflection gage (LVDT) was removed to prevent any possible damage to the gage due to sudden breaking of the test beams. The movement of the actuator head was recorded in the MTS controller software program over the entire loading period. The movement of the actuator head closely follows the specimen and was found to be very closely matching with the mid-span deflections recorded with the LVDT.
Fig. 6.17 Test Set-Up (Seawall Slab)

Fig. 6.18 Test Set-Up (Cap Beam)
6.4. Tests Results

6.4.1. Structural Tests and Typical Failure Modes

The seawall slabs were loaded gradually and uniformly at a rate of about 1200 lbs per minute, and the cap beams were loaded at a rate of about 2000 lbs per minute. A Vishay 5000 series data acquisition system was used to record the strain gage readings. The loads and deflections were recorded constantly at a rate of one reading per second. The development of cracks in the slabs and beams was tracked and noted. The failure mode was documented after the failure occurred in each test specimen. The actuator head moved with a constant rate after reaching the peak load, and caused significant bending and crushing of the specimens.

The failure modes of two seawall slabs are shown in Fig. 6.19, Fig. 6.20 and Fig. 6.21, and failure mode of cap beams are shown in Fig. 6.22 and Fig. 6.23. The beams failed in a ductile manner with large mid-span deflections, with the maximum deflections at failure reaching values well over 2 inches (50 mm). Cracks formed at nearly uniform spacing close to the mid-span of the slabs and beams. Also, crack maps for all beams and slabs are drawn in Fig 6.24 and Fig 6.25.
Fig. 6.19 Overall Failure Mode of Seawall Slab Reinforced with Steel (SWSS)

Fig. 6.20 Overall Failure Mode of BFRP Reinforced Slab (SWSB)
Fig. 6.21 Failure Mode of Seawall Slab Reinforced with Basalt (SWSB)

Fig. 6.22 Failure Mode of Cap Beam Reinforced with BFRP (SWBB)
Fig. 6.23 Failure Mode of Cap Beam Reinforced with Steel (SWSB)

Fig. 6.24 Crack Map of Cap Beam Reinforced with Basalt (SWBB) and Steel (SWBS)
The concrete strength on the day of the testing was 6,814 psi (47 MPa) which is much greater than 5,500 psi (37.9 MPa). Therefore, the test results were correlated to reflect the concrete strength of 5,500 psi, which is the design strength for a typical seawall design. The loads carried by the test slabs and the corresponding predicted loads are shown in Fig. 6.26 and Fig. 6.27. The total loads calculated were based on the required strength of the seawall system due to soil and water pressures and are also shown in Fig. 6.27.
Fig. 6.26 Design Adjustment for Seawall Slab

Fig. 6.27 Design Adjustment for Seawall Slab
The test results indicate that the test slabs failed very close to the predicted loads, particularly the BFRP reinforced test slab. A comparison of mid-span deflections for the two test slabs is shown in Fig. 6.28. The load corresponding to the service condition, factored load condition with a load factor of 1.6, and the required load for both steel reinforced test slab and the BFRP reinforced test slab are also shown in Fig. 6.28. The figure demonstrates that the actual strength of the test slabs is greater than the corresponding required load indicating that the details used in the current test program are adequate for the BFRP reinforced test slab.

The steel reinforced cap beam failed at a load of 34.8 kips (154.8 kN). The beam failed in flexure by tensile rupture of the bottom two steel reinforcing bars. The steel reinforced
beam is an under-reinforced concrete beam. The predicted load at first yielding of the steel bars is 23 kips (102.3 kN). If strain hardening is allowed for in the calculations, the predicted strength of the steel reinforced cap beam is 34.9 kips (155.2 kN), which is reasonably close to the maximum load carried by the beam at failure.

The BFRP reinforced cap beam failed at a load of 31.05 kips (138.1 kN). This load corresponds to a tensile stress in the BFRP bars of about 234 ksi (1,613 MPa). The beam is failed in shear mode. The predicted concrete shear strength ($V_{c,f}$) of the test beam is 5.5 kips (24.5 kN), and the contribution of the steel shear reinforcement in the form of #3 stirrups spaced at 12 inches (305 mm) is about 15.9 kips (70.7 kN).

Based on the soil pressures, the soil reaction pressures, the force transferred from the seawall slabs as reaction to the cap beam, the required maximum factored moment for the cap beams is about 21 kip-ft (30 kN-m), and the maximum factored shear force is about 12 kips (53.4 kN). These internal forces relate to the required point loads of about 12 kips (53.4 kN) for the steel reinforced cap beam, and about 15.2 kips (67.6 kN) for the BFRP bar reinforced cap beam for the current test configuration. When these loads are compared with the actual maximum loads recorded in the tests, it is clear that the failure loads of the two cap beams in the tests exceeded the factored load required in the design by a large margin (a factor of 2). There is a significant amount of reserve strength in the design of cap beams. There seems to be room for some optimization of the beam details because both the moment strength and shear strength obtained from the tests exceeded the respective factored moments and factored shears required from the design.
A comparison of the load deflection curves for the two cap beams is shown in Fig. 6.29. The two curves are seen in the figure to be somewhat comparable. The shape of the deflection curves for the two beams is also similar. A comparison of concrete compressive strains obtained for the two beams is shown in Fig. 6.30. The shapes of these load-strain curves are similar to the corresponding strains in the reinforcing bars. The tensile strains in the reinforcing bars are shown in Fig. 6.31. The strains are developed in a predictable manner except for the difference which is expected due to the difference in the elastic modulus values of the two materials.

![Fig. 6.29 Comparison of Mid-Span Deflections of Cap Beams](image)
Fig. 6.30 Comparison of Maximum Concrete Compressive Strains of Cap Beams

Fig. 6.31 Comparison of Stains in the Reinforcing Bar of the Two Cap Beams
6.5. Summary

Several alternative designs were developed for the components of a typical Florida seawall system using BFRP bars. The objective was to make the designs cost effective and viable relative to the current steel reinforced concrete components. A test program was developed to test representative specimens to simulate typical Florida seawall slabs and cap beams. The test results indicated that the seawall slab test specimens and the seawall cap beam test specimens performed in a predictable manner providing the required basis for the design approach followed in developing the details of the seawall system with BFRP reinforcing bars.

The design details of the seawall slabs shown in this chapter will work to achieve the required performance of the slab as intended in the original designs. The cap beams have some amount of reserve strength, and therefore, there is some potential to reduce the amount of BFRP reinforcement provided in the suggested details. The work described in this chapter relates to the evaluation of the designs utilizing small scale tests. Full scale test may be needed for further optimization of a concrete seawall system reinforced with BFRP bars. The test results in the chapter demonstrate that the implementation of BFRP reinforced concrete seawall system is feasible.
7.1. Conclusion

This dissertation is a summary of a project that includes coordinated analytical and experimental studies that were performed on Basalt FRP reinforcing bars and concrete members reinforced with BFRP bars. It was initiated to investigate the mechanical and creep-rupture properties of these bars, and also to study the feasibility of BFRP application as internal reinforcement in seawall structures. Therefore, several experimental tests were conducted on BFRP bars and members reinforced with BFRP bars.

Initially, the mechanical properties of BFRP were studied by performing tensile tests on different size of BFRP bars. As a result, the ultimate and guaranteed tensile strength, rupture strain, and modulus of elasticity of BFRP were all determined. Flexural behavior
of BFRP reinforced concrete beams were considered as the second step. Therefore, several beams with a wide range of reinforcement ratio was fabricated and tested with the standard 4-point loading method. In this step, the strength of each beam were compared with the prediction based on ACI 440 approach and three well-known strain compatibility methods to check the compatibility of these approaches for design of BFRP-RC beams. Also, the deflection of each beam was monitored and recorded, and was used to study load-deflection behavior. Along with that, the recorded deflections were used for deriving a new equation for effective moment of inertia $I_{eff}$ that is a key factor for the prediction of beam deflections at service load level.

The experimental studies were followed by expedited creep-rupture tests in which the BFRP bars were exposed to a simulated alkali environment at elevated temperature under different load levels. Strains were recorded continuously from the time of loading to rupture. The creep rupture strength and creep rupture coefficient curves were developed as a function of time in semi-logarithmic plots. The creep rupture coefficients were calculated for different service lives to be used for design in a similar manner to the ACI 440.1R recommendation. A BFRP reinforced concrete seawall slab and cap beam were considered as a case study. A seawall slab and its cap beam was designed with BFRP internal reinforcement to be a good substitute for currently used designs in the State of Florida. Commercial finite element analysis software PLAXIS 2D was applied to study the behavior of the seawall while interacting with soil and interfacing with water, and also to determine the maximum bending moment and shear force which the wall may
experience. The seawall test elements were fabricated and tested under 4-point loading method to study the flexural behavior and ultimate strength under static loading.

Each chapter in this dissertation has a section on conclusions and summary with discussion. The followings are conclusions derived from the experimental and analytical studies in this project.

1- Basalt FRP bars manufactured by using the automated wet lay-up method were found to have inherently high average tensile strength, acceptable rupture strain, and an average modulus of elasticity that is comparable to GFRP bars. Also, the guaranteed mechanical properties obtained from test results in this study are comparable to those of GFRP bars that are currently used in civil engineering construction industry.

2- BFRP reinforced concrete beams failed in a ductile manner with several small cracks distributed around the mid-span, which proves a good level of bond between BFRP bars and concrete. Most of the tested beams had higher flexural strength compared to the corresponding ACI 440.1R prediction, while their strengths were approximately equal to or less than those predicted using the strain compatibility approaches. This proves that strain compatibility method predictions are closer to actual strength of reinforced concrete beams reinforced with BFRP bars. However, for consistency of design methods, and ACI 440.1R approach is satisfactory and acceptable for practical applications.
3- The current effective moment of inertia $I_{eff}$ recommended by ACI 440.1R for FRP-RC beams is not suitable for BFRP reinforced concrete beams for all ranges of reinforcement ratios. The ACI 440.1R method is useful to predict the deflections only for larger reinforcement ratios. Although BFRP reinforcing bars have shown behaviors similar to GFRP bars, the available relationships for GFRP are not able to predict a good fit for Basalt FRP-RC beams. The proposed equation for $I_{eff}$ specific to BFRP showed a close match with the test results particularly in the service load range that is of practical interest to a design engineer. The nonlinear multiple regression analysis resulted in a reliable value for $R^2$ of 0.983 representing good accuracy of the proposed method. A simple and easy-to-use equation is established and recommended for practical use.

4- Creep coefficients for BFRP bars that were obtained from expedited experiments in this study present smaller value compared to GFRP coefficients and larger value compared to CFRP coefficients, which means that BFRP reinforcing bars have a larger plastic elongation (plastic strain) in compare to CFRP and but smaller plastic elongation in compare to GFRP bars. The reduction factors required for beams designed for creep rupture seem to be closer to those for GFRP than for CFRP. Creep rupture coefficients required for creep design according to ACI 440.1R are established and provided in a table for different service lives from 5 to 114 years, which may be selected based on
the application of the structure. These values may be used by design engineers when designing reinforced concrete reinforced with BFRP reinforcing bars.

5- The test results of seawall slab test specimens demonstrated that the alternative seawall system design with BFRP reinforcement studied in this project will function in a predictable manner to achieve the required performance of the slab as intended in the original design with steel reinforced concrete. The cap beams have large reserve strength, and therefore, there is a possibility to reduce the amount of BFRP reinforcement provided in the suggested details. The test results demonstrate that the implementation of BFRP reinforced concrete seawall system is feasible and achievable.

7.2. Recommendations

The following recommendations may be used for the future works on Basalt Fiber Reinforce Polymer, BFRP, reinforcing bars and their application in structural engineering.

- The proposed equation for calculation of the effective moment of inertia $I_{eff}$ of BFRP reinforced concrete beams can be extended to be applicable to other types of FRP-RC beams and slabs as well.

- The method used to study the creep rupture behavior of BFRP reinforcing bars in alkaline environments could be used for different environmental conditions.
• Other type of experimental durability tests may be performed on BFRP reinforcing bars. For example, the freezing and thawing cyclic test would be useful to investigate the behavior of BFRP-RC beams in cold regions.
References


[1.6] ACI Committee 440, Guide for the Design and Construction of Structural Concrete Reinforced with FRP Bars, 440.1R-06 A., American Concrete Institute, USA. 2006.


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[3.1] ACI Committee 440, Guide for the design and Construction of Concrete Reinforced with FRP Bars, ACI 440.1R-06, American Concrete Institute, Farmington Hills, MI, 2006.


[4.3] Canadian Standard Association, CSA, Concrete design handbook: Canadian Standard A23.3.94. Rexdale, ON, Canada; 1998.


[5.3] ACI Committee 440-06. Guide for the design and Construction of Concrete Reinforced with FRP Bars, ACI 440.1R-06, American Concrete Institute, Farmington Hills, MI, 2006.


