
**Materials Research Report
Final Report**

July 2014

**UNF Project
Contract No. BDK82-977-05**

Degradation Assessment of Internal Continuous Fiber Reinforcement in Concrete Environment

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The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the State of Florida Department of Transportation.

SI CONVERSION FACTORS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
LENGTH				
In	inches	25.4	millimeters	mm
Ft	feet	0.305	meters	m
yd	yards	0.914	meters	m

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
AREA				
in²	square inches	645.2	square millimeters	mm ²
ft²	square feet	0.093	square meters	m ²
yd²	square yard	0.836	square meters	m ²

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
VOLUME				
ft³	cubic feet	0.028	cubic meters	m ³
yd³	cubic yards	0.765	cubic meters	m ³
NOTE: volumes greater than 1,000 L shall be shown in m ³				

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
MASS				
oz	ounces	28.35	grams	g
Lb	pounds	0.454	kilograms	kg

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
FORCE and PRESSURE or STRESS				
lbf	pound force	4.45	newton	N
lbf/in²	pound force per square inch	6.89	kilopascals	kPa
kip	1,000 pounds force	4.45	kilonewton	kN

TECHNICAL REPORT DOCUMENTATION

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.	
4. Title and Subtitle Degradation Assessment of Internal Continuous Fiber Reinforcement in Concrete Environment		5. Report Date July 2014	
		6. Performing Organization Code	
7. Author(s) Dr. Adel ElSafty, Dr. Brahim Benmokrane, Dr. Sami Rizkalla, Dr. Hamdy Mohamed, and Dr. Mohamed Hassan.		8. Performing Organization Report No.	
9. Performing Organization Name and Address University of North Florida School of Engineering, 1 UNF Drive Jacksonville, FL 32224		10. Work Unit No. (TRAIS)	
		11. Contract or Grant No. BDK82-977-05	
12. Sponsoring Agency Name and Address State of Florida Department of Transportation 605 Suwannee St. MS 30 Tallahassee, FL 32399		13. Type of Report and Period Covered Final Report	
		14. Sponsoring Agency Code	
15. Supplementary Notes			
16. Abstract The Fiber Reinforced Polymer properties include high strength-to-weight ratio, high stiffness-to-weight ratio, high-energy absorption, and outstanding corrosion and fatigue damage resistance. Therefore, the use of continuous basalt, glass, and carbon fiber reinforcement in concrete structural applications seems to be promising for reinforcing new concrete structures and strengthening applications. The main objective of this study was to evaluate the characteristics of glass fiber reinforced polymer (GFRP) bars, basalt fiber reinforced polymer (BFRP) reinforcing and prestressing bars, and carbon fiber composite prestressing cables. The test variables included the tensile strength, the modulus of elasticity, behavior, and durability under severe environmental exposures. The study investigated the physical and mechanical properties, durability, and bond strength to concrete of the FRP composites. In addition, the investigators tested concrete beams and slabs reinforced with basalt (BFRP) bars and investigated the bond-dependent coefficient. The tested GFRP bars have shown a value of tensile capacity retention over 80%, meeting the D1 requirement of CSA S807 for the alkali resistance in high pH solution with load. The study also confirmed that some types of basalt FRP (BFRP) bars meet the requirements of ACI 440 and CSA S807 concerning their physical and mechanical properties. Moreover, the researchers in this study investigated the long-term performance of these BFRP bars in different environments and under different exposure conditions and recorded some degradation. Some tested basalt FRP bar did not satisfy the CSA S807 and ACI 440 requirements. SEM analysis of some BFRP has shown that porosity, large voids, and microcracks. The carbon fiber composite prestressing cables showed excellent performance, maintaining very high guaranteed tensile strength retention and elastic modulus retention after conditioning for over 7,000 hours in alkaline solution at 60°C. That indicates a great potential for implementing the use of the carbon prestressing cables in prestressed concrete applications.			
17. Key Word FRP, degradation, basalt, carbon, glass		18. Distribution Statement No restrictions	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 297	22. Price

ACKNOWLEDGEMENTS

The authors would like to thank the Florida Department of Transportation for their support of this research. In particular, special thanks go out to Mario Paredes (the project manager) and the engineers of the FDOT Materials Research Center in Gainesville, Florida. The authors would also like to include special thanks to the laboratory staff and the technicians at the FDOT Materials Research Center.

EXECUTIVE SUMMARY

Advanced composite materials have many desirable properties, such as high performance, high strength-to-weight ratio, high stiffness-to-weight ratio, high-energy absorption, and outstanding corrosion and fatigue damage resistance. Therefore, the use of continuous glass and carbon fiber reinforced polymer composites in concrete structural applications is promising for reinforcing new concrete structures, strengthening applications, and for replacement of steel reinforcement. Recently, few researchers also investigated the basalt fiber reinforced polymer composite as a new reinforcement material. The fiber reinforced polymer bars are steadily replacing conventional steel bars due to their resistance to corrosion in many applications including bridge decks. From the literature and this research study, the material characteristics show that the basalt fibers offer a higher ultimate strength than glass fibers at similar ultimate strain, thus providing a higher modulus of elasticity. However, carbon fibers provide the highest modulus of elasticity and the closest to reinforcing steel, but are subject to a very brittle rupture at relatively low strains. Some literature indicates that there is a need for more investigation to study the alkali resistance of basalt and glass fibers. In addition, due to the limited amount of research on the use of basalt fibers for structural applications, this research was conducted to investigate the confidence in using the basalt fibers to reinforce concrete structures in place of steel bars. It was determined that mechanical properties, durability, and bond strength to concrete are the most important characteristics that should be examined for any new basalt reinforcing bars before their use as reinforcement for concrete structures.

The main objective of the testing program in this study was to evaluate the characteristics of glass fiber reinforced polymer (GFRP) bars, basalt fiber reinforced polymer (BFRP) reinforcing and prestressing bars, and carbon fiber composite prestressing cables. The test variables included the tensile strength, the modulus of elasticity, behavior, and durability under severe environmental exposures. The researchers in this study investigated the physical and mechanical properties, durability, and bond strength to concrete of the FRP composites. In addition, the investigators tested concrete beams and slabs reinforced with basalt (BFRP) bars and investigated the bond-dependent coefficient. The researchers conducted comparisons between the predicted and exper-

imental ultimate load, strains, and deflection. The researchers analyzed the results and drew conclusion regarding the FRP performance.

The study concluded that the immersion in high pH did not significantly affect the elastic modulus of GFRP bars. The tested GFRP bars have shown a value of tensile capacity retention over 80%, meeting the D1 requirement of CSA S807 (2010) for the alkali resistance in high pH solution with load (required limit is 70%).

The study also confirmed that some types of basalt FRP (BFRP) bars meet the requirements of ACI 440 (2008) and CSA S807 (2010) concerning their physical and mechanical properties. Moreover, the researchers in this study investigated the long-term performance of these BFRP bars in different environments and under different exposure conditions and recorded some degradation. Some tested basalt FRP bar did not satisfy the CSA S807 and ACI 440 requirements; in particular, the thermal properties (T_g and cure ratio) are very low and not acceptable. SEM analysis of some BFRP has shown that porosity, large voids, and microcracks. The carbon fiber composite prestressing cables showed excellent performance, maintaining very high guaranteed tensile strength retention and elastic modulus retention after conditioning for over 7,000 hours in alkaline solution (high pH of 12.8) at 60°C. That indicates a great potential for implementing the use of the carbon prestressing cables in prestressed concrete applications.

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CHAPTER 1

INTRODUCTION

1.1 Background

The use of continuous basalt, glass, and carbon fiber reinforcement in concrete structural applications seems to be promising for reinforcing new concrete structures, strengthening applications, and for replacement of steel reinforcement. The FRP bars are steadily replacing conventional steel bars due to their resistance to corrosion in many applications including bridge decks. From the literature, the material characteristics show that the basalt fibers offer a higher ultimate strength than glass fibers at similar ultimate strain providing a higher modulus of elasticity. However, carbon fibers provide the highest modulus of elasticity and the closest to reinforcing steel, but are subject to a very brittle rupture at relatively low strains. Some literature indicates that there is a need for more investigation to study the alkali resistance of basalt and glass fibers. In addition, due to the limited amount of research on the use of basalt fibers for structural applications, further investigations are still required to provide confidence in the use of the basalt fibers to reinforce concrete structures in place of steel bars. It was determined that mechanical properties, durability, and bond strength to concrete are the most important characteristics that should be examined for any new basalt reinforcing bars before their use as reinforcement for concrete structures. Many researchers investigated the mechanical properties of carbon, glass, Aramid, and basalt fibers. The researchers concluded that basalt FRP can be a potential competitor to glass, carbon and Aramid FRP and could potentially replace steel reinforcing bars for buildings. The results indicated that the basalt fibers offer a higher ultimate strength than glass fibers at similar ultimate strain providing a higher modulus of elasticity. The carbon fibers provide the highest modulus of elasticity and the closest to the reinforcing steel, but are subject to a very brittle rupture at relatively low ultimate strain.

1.2 Research Significance

The use of continuous fiber reinforced polymer (FRP) composites in concrete structural applications is promising due to high performance, high strength-to-weight ratio, high stiffness-to-weight ratio, high-energy absorption, and outstanding corrosion and fatigue damage resistance. Many researchers investigated the GFRP and CFRP composites. However, few researchers investigated the basalt fiber reinforced polymer composite as a new reinforcement material. Literature indicated that there is a need for more investigation to study the alkali resistance of basalt, glass, and carbon FRP. In addition, due to the limited amount of research on the use of basalt fibers for structural applications, this research was conducted to investigate the confidence in using the basalt fibers to reinforce concrete structures in place of steel bars. It was determined that mechanical properties, durability, and bond strength to concrete are the most important characteristics that should be examined for any new basalt reinforcing bars before their use as reinforcement for concrete structures. The outcome of this research assessed the feasibility of using GFRP, CFRP, and BFRP to replace steel reinforcement.

1.3 Objectives

The main objective of the testing program is to evaluate the characteristics of the glass fiber reinforcing bars, carbon fiber prestressing cables, and basalt fiber reinforcing bars and basalt fiber prestressing bars. The test variables include the tensile strength, the modulus of elasticity, performance, and durability under severe environmental exposures. The research variables also include the physical and mechanical properties, durability, and bond strength to concrete of the FRP composites. In addition, the study investigates concrete flexure members reinforced with basalt (BFRP) bars and bond-dependent coefficient. The objectives also included providing a comprehensive review of fiber-reinforcing materials, and identifying possible material degradation of fiber, filler, and outer epoxy material components. The researchers also determined the implications of possible material degradation on mechanical behavior of fiber-reinforcing material.

1.4 Methodology

In this study, the researchers investigated the physical and mechanical properties, durability, and bond strength to concrete of the FRP composites. For GFRP, the research team investigated the physical characteristics, tensile properties, alkali resistance of GFRP in high pH under load and without load. For carbon fiber prestressing cables, the researchers performed physical characterization and long-term durability in alkaline solution and elevated temperatures. For BFRP prestressing and reinforcing bars, the researchers conducted physical characteristics (fiber content, thermal expansion, void content, water absorption, glass transition temperature “ T_g ”, and cure ratio), SEM analysis, tensile properties, shear strength in high pH under load and without load, and other tests for different types of BFRP. In addition, the investigators tested concrete beams reinforced with basalt (BFRP) bars and investigated the bond-dependent coefficient. In addition, the research team investigated the flexural behavior of slabs reinforced with BFRP. The researchers conducted comparisons between the predicted and experimental ultimate load, strains, and deflection. The researchers conducted the tests in accordance to the requirements of the ACI Committee Report 440.3R-04, which provides a guide for the test methods for Fiber-Reinforced Polymers (FRPs) for reinforcing and strengthening concrete structures. The researchers also investigated the FRP materials according to the CAN/CSA S6-06 and CAN/CSA-S806-12.

CHAPTER 2

LITERATURE REVIEW

2.1 General

In many parts of the world, corrosion of steel reinforcement in concrete structures is a major durability problem, leading to structural degradation and consequent costly repairs and loss of serviceability. In the recent decade, the use of advanced composites, normally called fiber reinforced polymers (FRP), as reinforcement for concrete structures has emerged as one of the most promising new technologies in construction to overcome the problem of corrosion. Several design guides and codes on reinforcing structural concrete members with FRP reinforcement were developed and published in several countries (ISIS design manual No. 3, 2007; CSA S6, 2006; ACI 440.1R, 2006); CNR-DT 204-06, 2006); FIB *Task Group 9.3*, 2007; and CSA S806, 2012). Countries such as, Canada, United States (USA), Japan and some other European countries have already implemented the use of FRP in bridges deck slabs, parking structures, barrier walls, continuous pavement, and other concrete structures. To date, however, FRPs have not realized their full potential within the construction industry. One of the contributing factors is limited information regarding their long-term performance. In particular, since the service life of a civil engineering structure is typically 50 to 100 years, knowledge of the long-term durability of FRPs is of prime importance. While, the durability of FRP materials depends on numerous factors including the fiber properties, resin properties and manufacturing process used. As such, knowledge of durability and degradation mechanisms of construction materials will play a crucial role in order to build confidence within the industry and to identify both the limits and potential of FRPs in structural applications.

2.2 FRP Composite Materials

The term composite material is a generic term used to describe a combination of two or more materials that yields a product that is more efficient than its constituents. One constituent is called the reinforcing or fiber phase (one that provides strength); the other in which the fibers are embedded is called the matrix phase. The matrix, such as a cured resin-like epoxy, polyester, vinylester, acts as a binder and holds the fibers in the intended position, giving the composite material its structural integrity by providing shear transfer capability.

The four FRPs commonly used are composites reinforced with glass fibers called glass FRP (GFRP), composites reinforced with carbon fibers called carbon FRP (CFRP), composites reinforced with Aramid fibers called Aramid FRP (AFRP), and composites reinforced with Basalt fibers called Basalt FRP (BFRP). Basalt FRP (BFRP) is the latest FRP composite that has developed within the last ten years and has been proven to have some advantages of enhancing safety and reliability of structural systems compared with CFRP, GFRP, and AFRP composites. Ad-

vanced composite materials have many desirable properties, such as high performance, high strength-to-weight, high stiffness-to-weight ratios, high-energy absorption, and outstanding corrosion and fatigue damage resistance. Besides, FRP reinforcement bars have more flexibility, elasticity, and minimal environmental impact particularly when used in infrastructure such as tunnel application as soft-eyes. This avoids damages to cutter heads and does not delay the work progress as pilling or cutting through FRP bars is unproblematic. The fiber bars are split in small pieces, which do not harm slurry pipes. Hence, partial or complete adoption of FRP composites as structural members can significantly enhance structural safety and sustainability. FRP products are manufactured in different forms such as bars, fabrics, 2D grid, 3D grid, or standard structural shapes. Figure 2.1 shows various types and shapes of currently available FRP products.

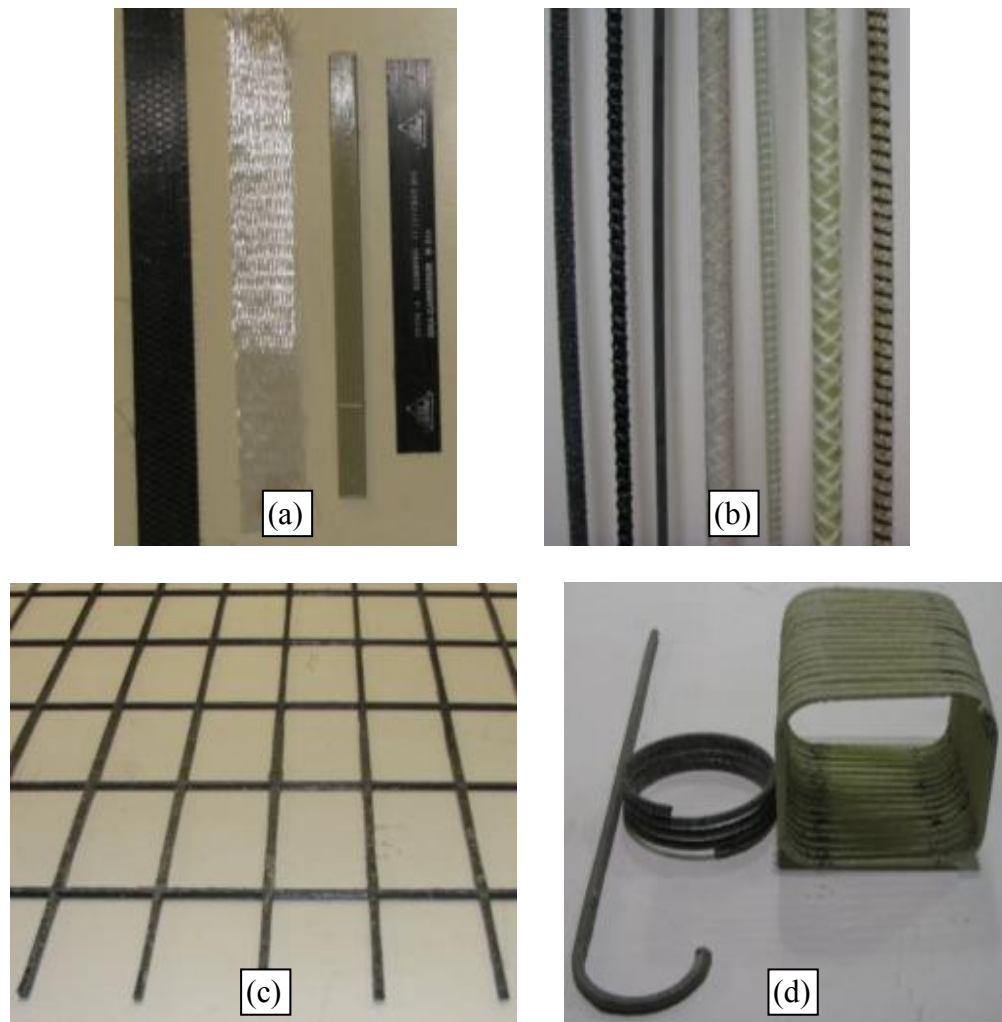


Figure 2.1: Different FRP products: (a) fabrics and strips; (b) straight bars; (c) grids; (d) spiral stirrups and curved bars.

2.3 General Characteristics of FRP Reinforcing Bars

FRP reinforcing bars are manufactured from continuous fibers (such as carbon, glass, aramid, and basalt) embedded in matrices (thermosetting or thermoplastic). A key element in evaluating

the FRP properties is the characterization of the relative volume and/or mass content of the various constituent materials. The physical and mechanical properties influence the FRP reinforcing bars in concrete structures. Design variables include the choice of constituents (fiber and polymeric matrix), the volume fractions of fiber and matrix, fiber orientation, and the manufacturing process. Other factors such as dimensional effects and quality control during fabrication play an important role in determining the characteristics of FRP bars. The loading history, duration of loading, temperature, and humidity also affect FRP materials. FRP bars are produced in different diameters, depending on the manufacturing process. FRP bars normally have tensile strength higher than the tensile strength of conventional steel bars. This relatively high tensile strength makes FRP bars suitable as reinforcement for concrete structures. The tensile behavior of FRP bars having one type of fiber material is characterized by a linearly elastic stress-strain relationship up to failure. The FRP bars do not exhibit any plastic behavior before rupture. Figure 2.2 shows typical tensile stress-strain relationships of FRP reinforcement compared to conventional steel bars. The figure also shows that the modulus of elasticity of FRP bars is lower than that of steel bars. The CFRP has the highest modulus of elasticity, which ranged from 60% to 75% of that for steel. While the GFRP bars has the lowest modulus of elasticity, which ranged from 20% to 25% of that for steel. Table 2.1 shows the mechanical properties of some commercially available FRP reinforcing bars.

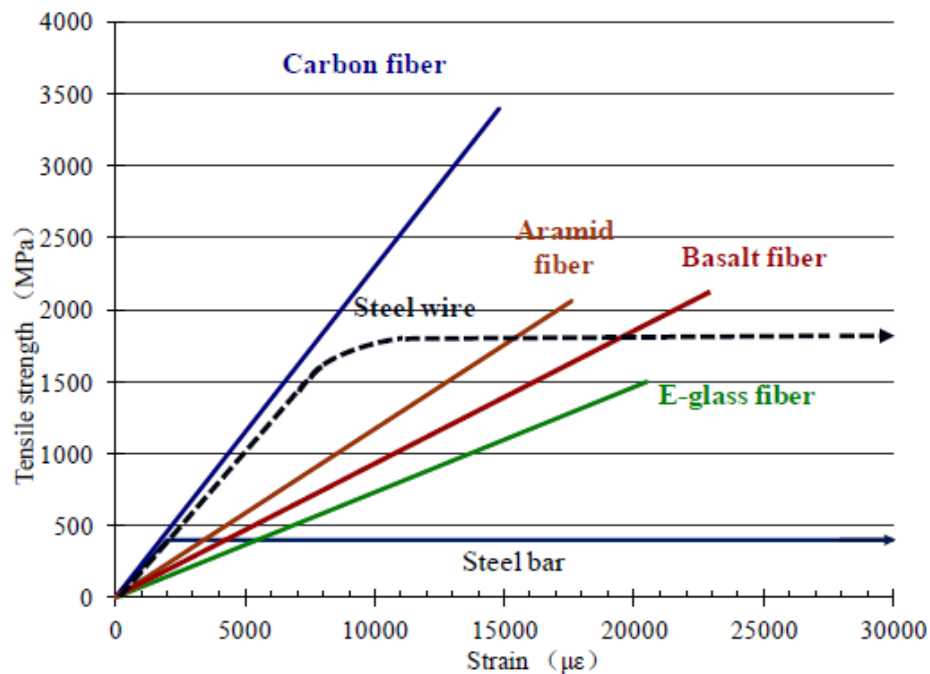


Figure 2.2: Typical stress-strain relationships of different FRPs compared to steel bars (Zhishen et al., 2012)

Bond behavior of an FRP bar depends on the surface preparation and mechanical properties of the bar itself as well as the environmental conditions. The FRP bar surface preparations can be categorized in two forms according to the bond stresses transfer between the FRP bar and the concrete, friction forming preparations and bearing forming preparations. The bars in the first category are coated with a granular material before the bars are completely cured. These granular particles increase bond transfer through friction between the bars and concrete. Another way of

increasing the bond strength of the bars is through the formation of indentations or deformations on the bar before full curing. The V-ROD FRP bars; which have sand-coated surface and are produced by Pultrall Inc., Quebec, Canada, stand as example of the bars of first category, whereas Leadline™ CFRP bars; which have indented surface and are produced by Mitsubishi Chemical Cooperation, Japan, stand as example of the bars of second category. In addition, both methods may be combined, whereas the surface of the Aslan FRP bars produced by the Hughes Brothers Inc., USA, contains indentations as well as a granular coating. Figure 2.3 shows different surfaces types of sand-coated and deformed FRP bars. Further information concerning the physical and mechanical properties, time dependent behavior, and durability of FRP reinforcement, can be found in the following: (JSCE, 1997; ACI 440.1R, 2006; ISIS design manual No. 3, 2007; CSA S806, 2012; CSA S807, 2010; and CSA S6, 2006).



Figure 2.3: Different surfaces types of FRP bars

Table 2.1: Typical mechanical properties of FRP reinforcement bars

Trade name	Fiber type	Guaranteed ten- sile strength	Modulus of elasticity	Ultimate ten- sile strain
------------	------------	----------------------------------	--------------------------	------------------------------

		(MPa)	(GPa)	(%)
V- ROD ¹	Carbon	1356-1765	120-144	1.18-1.13
V- ROD LM ¹	Glass	666-804	43-45	1.34-1.89
V- ROD SM ¹	Glass	703-941	53-57	1.33-1.79
V- ROD HM ¹	Glass	1,000-1,372	63-66	1.15-2.11
Aslan 200 ²	Carbon	2,068-2,241	124	1.17-1.81
Aslan 100 ²	Glass	551-896	46	1.19-1.94
ComBAR ³	Glass	1,000	64	1.17
Leadline ⁴	Carbon	2250	147	1.50
RockBAR ⁵	Basalt	1107-1350	43-48	2.72-3.10
Dost Re-Bar ⁶	Carbon	2,300 [*]	130	1.80
Dost Re-Bar ⁶	Aramid	1,400 [*]	60	2.40
Dost Re-Bar ⁶	Glass	1,000 [*]	40	2.8

¹ Pultrall Inc. (<http://www.pultrall.com>).

² Hughes Brothers Inc. (<http://www.aslanfrp.com>).

³ Schöck Inc. (<http://www.schoeck.ca>).

⁴ Mitsubishi Chemical Corporation Inc. (ISIS manual 3, 2007).

⁵ Kammeny Vek Inc. (Serbescu, A. 2009).

⁶ DostKimya Inc. (<http://www.dostkimya.com>); (^{*}) tensile strength.

2.4 Durability Issues of FRP for Civil Infrastructure

The steel corrosion in concrete structures is one of the most important causes of deterioration and structural deficiency (Figure 2.4). The costs of repairs and restoration in USA, Canada, and in the majority of the European countries constitute a high percentage of their total expenditure on infrastructure. The American Society of Civil Engineers (ASCE) estimates that over \$1.6 trillion would be needed in the next five years in the U.S. to alleviate potential problems with the civil infrastructure. As discussed in the literature (Benmokrane and Mohamed, 2013), Canada's deficit for its municipal infrastructure, which represents 70 percent of the country's total infrastructure, was estimated to be \$60 billion in 2004, and expected to grow at \$2 billion per year. The use of (FRP) composites as a cost effective alternative to traditional construction materials is a promising solution. Despite the resistance of FRP to electrochemical corrosion, the performance of FRP may deteriorate due to environmental, physical, or chemical conditions, leading to loss of strength and stiffness. The literature (Chu and Karbhari, 2005; Chen et al., 2006; Chen et al., 2007; Riebel and Keller, 2007) indicates that the performance of FRPs deteriorates due to certain physical (e.g., cyclic or sustained loading, moisture diffusion, extreme temperature variation) or chemical (e.g., alkalinity) exposure. The degree of deterioration depends on a variety of factors such as the type and volume of fibers and resin matrix, the exposed environment, and the manufacturing process. Furthermore, due to the addition of FRP composites to concrete structures, the durability performance of FRP reinforced concrete structures becomes more complex due to the combined effect of FRP composites, interface, concrete, and various environmental and mechanical conditions. Therefore, assessing the durability of FRP composites, in association with concrete structures and as an individual material, is a very complex task. The literature indicated

several durability influence factors: fluids (moisture; chemical solutions); alkalinity; freeze–thaw; creep/relaxation.



Figure 2.4: Concrete structure deteriorated by the corrosion of steel reinforcement

2.4.1 Main factors affecting FRP internal reinforcement durability

The most commonly available FRPs, which can be used for internal reinforcement, are glass (GFRP), carbon (CFRP) and Aramid (AFRP). The degree of damage/deterioration of internal FRP reinforcement depends on fundamental factors such as the type and volume fraction of fibers, type of resin, morphology, and strength of the fiber-matrix interface, severity of the exposure environments and the process of fabrication. The E-glass fibers are the most susceptible to degradation due to the moisture and alkalinity while carbon fibers are relatively inert to such environments. Aramid fibers, on the other hand, are highly resistant to abrasion and impact, but are sensitive to creep, moisture, and ultraviolet light. To achieve appropriate performance, a suitable type of resin must protect the fibers. The durability of the resin system is dependent on several factors such as the resin components, their individual properties and proportions, as well as curing time and conditions.

Moisture

Concrete with internal FRP reinforcement is generally exposed to alternating wet/dry cycles, natural weathering, and corrosive media. Even if concrete provides an excellent first line of defense, the permeability could eventually transmit moisture and other corrosive elements to the internal reinforcement. The effect of fluids on the performance of FRP composites has been one of the most studied subjects related to durability of composites during the past decades. In general, the sorption behavior of fluid into FRPs depends on types of fluid (water, acid, base), fluid concentration, temperature, external applied stress, type of fiber and resin, interphase, process of molding and state of material (damage, curing condition). (Ben Daly et al., 2007) showed that moisture diffusion took place in pultruded composites. The maximum saturation level could be related to the presence of fillers and additives in the polymer matrix. The literature indicated that the rate of sorption is controlled by the chemical structure of the matrix (degree and type of cross-linking, presence of void), interface/interphase, and manufacturing process. Researchers

have attempted to control diffusion process by using resin matrix with lower permeability (Benmokrane, 2000), modifying interphase region by using suitable sizing chemistry or select appropriate process of molding to reduce void content. In addition, moisture ingress can degrade resin by chemical attack (hydrolysis) or drop in glass transition temperature (Chin et al., 2001). For this reason, fluids affect matrix dominant properties such as transverse and shear strengths of FRP composites and decreases of these properties become more important with increased exposure time and temperature (Liao et al., 1998).

Glass fibers are particularly sensitive to fluids ingress, due to their susceptibility to chemical and physical attack. The level of degradation depends on composition of fibers, fluid type and concentration and exposed temperature. Extensive studies have been conducted in this research area (Chin et al., 2001; Al-Zahrani, 2005; Chu and Karbhari, 2005; Chen et al., 2006). Typically, the tensile strength of E-glass/vinylester composites would decrease by 40% in 100% relative humidity environment at 93°C. In general, (Al-Zahrani, 2005) has shown that the modulus of elasticity was less affected by all the exposure conditions than the tensile strength. Carbon fibers are not affected by fluids ingress, but resin matrix is usually affected. Consequently, the performance of composites is also affected in the case of CFRP. For unidirectional carbon composites, this usually leads to reduction in the compressive strength and shear strength, but only a small effect on the tensile strength since this property is especially dominated by fibers which are not affected by fluids (Dejke, 2001). (Hancox and Mayer, 1994) reported minimal weight gain and strength loss for carbon/epoxy specimen exposure to 65% humidity for over four months and to boiling water for over three weeks. Aramid fibers are affected by fluids, mostly at higher temperatures. AFRP composites saturated in water have been reported to lose 35% of their flexural strength at room temperature (Allred, 1984), and up to 55% if stressed and under wet/dry thermal cycles (Sen et al., 1996). On the other hand, Technora and Kevlar fibers, also categorized as Aramid fibers, show different behavior under combined effects of fluids and temperatures. For Technora fibers, no degradation was observed in distilled water at any temperature; however, the decreases in strength were reported for specimens immersed in acid and alkali solutions and the reduction was increased with time and temperature.

Alkaline environments

The concrete environment has high alkalinity, with pH between 12 and 13 depending on the concrete mixture design and the type of cement used, (Benmokrane et al., 2006). This alkaline environment damages glass fibers through loss in toughness, strength, and embrittlement. In general, carbon fibers are known to exhibit the best alkaline resistance followed by Aramid then glass fibers. Glass fibers are damaged due to the combination of two processes (1) chemical attack on the glass fibers by the alkaline cement environment, and (2) concentration and growth of hydration products between individual filaments (Murphy et al., 1999). The embrittlement of fibers is due to the nucleation of calcium hydroxide on the fiber surface. The hydroxylation can cause fiber surface pitting and roughness, which act as flaws severely reducing fibers properties in the presence of moisture. In addition, calcium, sodium, and potassium ions found in the concrete pore solution are highly aggressive towards glass fibers. Therefore, the degradation of glass fibers not only depends on the high pH level, but also due to the combination of alkali salts, pH, and moisture. Aramid fibers show strength degradation in an alkaline environment. Kevlar 29 exposed to 10% sodium hydroxide solution for 1,000 hours loses 74% of its strength. High modulus Aramids such as Kevlar 49 demonstrates better alkaline resistance (Malvar, 1998). Carbon

fibers are commonly believed not to be affected by alkaline environment. (Judd, 1971) reported that carbon fibers were resistant to alkaline solutions at all concentrations and all temperatures up to boiling. Carbon tows immersed for 257 days in a very basic 50% sodium hydroxide solution showed variations in strength and elastic modulus only around 15%. Although an appropriate resin matrix (vinylester, epoxy) provides certain level of protection to fibers from alkaline degradation, migration of high pH solutions and alkali salts through resin (or through void, crack, interface between fiber/matrix) to the fiber surface is possible. Katsuki and Uomoto (Katsuki and Uomoto, 1995) used Electron Probe Microscopy to track ingress of alkali ions (sodium ions) into Aramid, carbon and glass reinforced vinyl ester rods. Sodium ions penetrated into GFRP in radial direction with time. No degradation was noticed in AFRP, or CFRP rods immersed for 60 days compared to GFRP rods. Chin et al. (Chin et al., 2001) observed, by Energy Dispersive X-Ray Analysis, that appreciable amounts of sodium, potassium and calcium were found in the interior of isopolyester specimens exposed to 60°C pore solution for 60 days. However, the authors also noticed that no evidence for the ingress of ions was found for the vinylester specimen. Hojo et al. (Hojo et al., 1991) studied the corrosion behavior of resins in aqueous solutions, and compared it with that of the metal. Three forms of corrosion mechanisms were found namely surface reaction, corrosion layer forming, and penetration type. By using corrosion depth, with immersion time, the authors found that the concept of corrosion rate could be applied even in polymeric materials. The corrosion rates were dependent on the chemical structure and reactivity between resin and environments. Resin cracking and fiber delamination creating a pathway for moisture and ionic species, may also be observed (Benmokrane et al., 2006).

A recent study on the durability of GFRP bars in moist concrete was conducted (Robert et al., 2009). Sand-coated GFRP bars with a nominal diameter of 12.7 mm were embedded in concrete and exposed to tap water at 23, 40, and 50°C for periods of 60–240 days. The tensile test results showed that at 40 and 50°C, the decrease of the tensile strength was 10% and 16%, respectively, of the original tensile strength after 240 days of exposure. In a field study (Mufti et al., 2005) concrete cores were taken from five in-service concrete bridge structures of 6–8 years age across Canada, reinforced with GFRP bars. On the basis on microscopic and chemical analysis, they concluded that the concerns about the durability of GFRP in alkaline concrete, based on simulated laboratory studies in alkaline solutions, are unfounded. The performance of the glass fiber-reinforced polymer bars was evaluated by conducting tensile tests on the bars extracted out from the concrete prisms after exposure to different conditions. In addition, scanning electron microscopy was used to investigate the degradation mechanism of the bars. After 18 months of exposure, test results showed that both the tap water at 50°C and the alkaline solution at 50°C had the maximum harmful effect on the tensile strength of glass fiber-reinforced polymer bars. The two field conditions showed almost no degradation (1% - 2%) in the tensile properties of the tested bars, (Almusallam et al., 2013).

Creep and relaxation

Polymer resins generally exhibit creep and relaxation behavior. Since glass and carbon fibers are linear elastic to failure, the addition of these fibers increases the creep resistance of the resins. Moreover, creep behavior of FRP composites strongly depends on the structure of material and load condition of the material. Consequently, creep and relaxation behaviors are more pronounced when load is applied transverse to fibers or when the composite has a low fiber volume fraction. For these reasons, for unidirectional FRP composites, the creep compliance is less af-

affected by the creep behavior of the polymer matrix when the material is loaded along fiber direction. For off-axis loading, the creep behavior is strongly dependent on the creep of the matrix. FRP composites are reinforced with discontinuous randomly oriented or with continuous bi-directional woven fibers. The properties of the material is matrix dominated, therefore creep of polymer matrix is the main reason for creep behavior of FRP composites. Typically, thermosetting resins (unsaturated polyesters, vinylesters, epoxies, and phenolics) are more resistant to creep than are thermoplastics (polypropylene, nylon, polycarbonates, etc.). Under-cured resins are susceptible to creep during the early stages of service but this susceptibility diminishes with time. In addition, physical aging, temperature, moisture and stress level could affect creep behavior of FRP composites. Some data related to these influencing parameters could be found from recent review (Weitsman and Elahi, 2000). The data on the effect of moisture absorption to creep behavior is rare. Fluids and time often can affect creep behavior of FRP composites in a manner similar to the effect of temperature and time. Fluid absorption in FRP composites will lead to developing residual stress and plasticizing of the resin, which can accelerate time-dependent behavior of FRP composites. (Liao et al., 1998) schematically described the effect of time, temperature, and fluids on creep behavior of FRP composites. The author concluded that creep compliance increased with the increase in fluid content and temperature over time.

In general, carbon fibers are the least susceptible to creep-rupture, Aramid fibers are moderately susceptible, and glass fibers are the most susceptible to creep-rupture. Creep-rupture tests have been conducted on GFRP, CFRP, and AFRP bars with 0.25 in (6.35 mm) diameter. The bars were tested at different load levels at room temperature. The results indicated that a linear relationship exists between creep-rupture strength and the logarithm of time for all load levels. Extrapolation of short term creep data to longer service lifetimes at room temperature air suggest rupture strengths of 29-55%, 47-66% and 79-93% of the initial strength for essentially unidirectional GFRP, AFRP and CFRP bars, respectively (Greenwood, 2002). (Benmokrane, 2000) studied stress rupture mechanism of GFRP bars in various alkaline environments (NaOH, simulated pore water, moist concrete) under different stress levels of 22 to 68% of ultimate strength (Benmokrane and Mohamed, 2013; Benmokrane, 2000). The results obtained indicate that stress rupture mechanism of GFRP bars depends on the environment and stress level. Under a stress level of about 20-30% of the ultimate strength, in NaOH solution, the stress rupture mechanism mainly involves the damage of fibers. While in simulated pore water solution and concrete environment, the stress-rupture is caused by interface damage. At high stress levels, above 55%, matrix and fiber cracking are the most dominant mechanisms. Nkurunziza et al. (Nkurunziza et al., 2005) conducted stress rupture tests on GFRP bars. Two test series; with 9.5 mm diameter, sand coated bars were investigated under two different stress corrosion environmental conditions. The first series consisted of 10 specimens immersed in de-ionized water (pH 7.0), stressed of 25 and 38% of ultimate, and subjected to ambient temperature. The other series consisted of 10 specimens immersed in alkaline solution (pH 12.8), subjected to the same temperature and stress level. After 417 days of exposure, the average residual tensile strength was found to be 139 and 144% of the design tensile strength for bars conditioned in de-ionized water at 25 and 38% stress level respectively. In alkaline solution, this range was 126 to 97%. These results showed that the testes GFRP bar performed very well under these extreme loading and environmental conditions. More important, no significant change in the elastic modulus was observed under the stress levels and environmental conditions used (Nkurunziza et al., 2005).

Freeze and freeze-thaw cycles

In general, freeze and freeze-thaw exposures do not affect fibers, although they can affect the resin and the fiber/resin interface. Majority of studies in this subject were made towards the aerospace materials. The reported literature on effect of freeze-thaw on pultruded FRP composites is very limited. In terms of material degradation due to low temperature and freeze-thaw cycles, (Lord and Dutta, 1988) gave an extensive review on this subject (Lord and Dutta, 1988). In general, at low temperature, complex residual stress arises within the FRP composites as a result of matrix stiffening and mismatch of thermal expansion coefficients of matrix and resin as well as FRP and concrete. Residual stresses can cause microcracks in the matrix and fiber/matrix interface, which can grow under low temperature thermal cycling and coalesce to form transverse matrix cracks and fiber/matrix debonding, leading to degradation of FRP composites. The presence of deicing salt in wet conditions with subsequent freeze-thaw cycling can cause microcrack formation and gradual degradation due to crystal formation and increased salt concentration, in addition to effects of moisture including swelling and drying (Karbhari et al., 2003).

In general, the reported literature showed that unidirectional tensile strengths decreased in the -10 to -40°C range, whereas the off-axis and transverse strengths may increase due to matrix hardening, and increasing freeze-thaw cycles have shown to accentuate residual stresses resulting in increased severity and density of cracks. The apparent increase in matrix brittleness and decrease in tensile strength was also reported (Lord and Dutta, 1988; Benmokrane et al., 1998; Burgoyne, 2001). On the other hand, Shao and Kouadio (Shao and Kouadio, 2002) showed that samples from pultruded GFRP sheets saturated in water have an excellent resistance to freeze-thaw cycles. The authors observed nearly no changes in tensile properties of FRP composites after 564 cycles between 4.4 and -17.2°C, even if samples were saturated with water. Mashima and Iwamoto (Mashima and Iwamoto, 1993) studied the change in bond characteristics due to freeze-thaw action on non-metallic reinforcement made up of synthetic fibers such as carbon, Aramid, and vinylon fiber. The bond strength was measured by the pullout test after 200 freezing and thawing cycles. The freeze-thaw cycle testing of the 10 x 10 x 10 cm cube specimens began after 14 days curing in water of 20°C. Four types of FRP rods, CFRP, GFRP, AFRP, and steel reinforcing bars were used in the study. It was determined that bond strength of glass, vinylon, and carbon FRP rods were not influenced by freeze-thaw, but Aramid FRP (both braided and coiled types) rods showed a gradual reduction of bond strength up to about 20 percent with progress of freezing and thawing.

Effect of thermal expansion

In conventional steel-reinforced concrete structures, the coefficient of thermal expansion for steel reinforcing bar and concrete is almost similar. On the other hand, the coefficients of thermal expansion of FRP reinforcing bars vary in the longitudinal and transverse directions depending on the types of fiber and resin, volume fraction of fiber, and orientation of fiber. Moreover, the resin as a binding material has a very large expansion coefficient. Therefore, thermal stresses developed at the interfaces between FRP reinforcing bar and concrete. FRP reinforcing bars generally consist of unidirectional fibers, except when surface configuration is modified through the use of veils or fiber types. Absence of transverse fibers resulted in transverse coefficient of thermal expansion being higher than the longitudinal one and is around five times larger than that of concrete that causes an important mismatch. Longitudinal thermal expansion of FRP reinforcing bar is relatively low, since it is mainly contributed by the fiber. However, like it was just said, the

transverse expansion coefficient of FRP reinforcing bar that is mainly due to the resin is three to five times larger than that of concrete. The coefficient of thermal expansion of GFRP reinforcing bars is 9.9×10^{-6} mm/mm/°C (Ehsani et al., 1993). However, the expansive coefficient of other FRP such as CFRP, AFRP, and GFRP reinforcing bars is different from that for concrete and steel and even becomes negative in the case of Aramid FRP rod. Consequently, due to the difference in expansion and contraction between the FRP reinforcing bar and concrete, forces develop in the concrete member subjected to temperature variation. Presence of moisture or other chemicals in the FRP reinforcing bars contributes to further degradation.

Fatigue

Fatigue performance is an important limit state that must be considered by designers of bridges, parking garages, and other structures that are subjected to cyclic or moving loading. Over the last several decades, extensive fatigue data of FRP composites have been generated for other industrial applications (Liao et al., 1998; Weitsman and Elahi, 2000). However, limited studies have been conducted on fatigue performance of pultruded FRP composites (bars, tendons, sheets, plates, and FRP structural shapes) for civil infrastructure applications. The fatigue performance of FRP composite materials depends on the matrix composition and on type of fiber (Curtis, 1989). Curtis (Curtis, 1989) demonstrated that the matrix composition has a greater effect on the fatigue performance than the type of fiber used. Cyclic loading promotes creation and development of flaws and microcracks in FRP materials. Also, as frequency of loading increases, stress difference between fiber and resin increases, resulting in interfacial damage. Cyclic loading category in bridges is called a high-cycle or fatigue loading, where the load history contains cycles in the range of two million at a low bond stress range. Such high cycle loading is considered a problem at service load levels. The most significant effect of high level repeated loads is the reduction of bond at failure. Moreover, since bridge decks are subjected to excessive cyclical loads and degrade in properties under long-term environmental exposure, endurance limit of FRP reinforcement in tension-fatigue should be established. In addition, endurance limits under varying temperatures (-28 to 60 °C) have to be established to arrive at threshold limits for designing with FRP reinforcement.

Concrete bridge deck slabs are affected by long-term fatigue endurance and durability of constituent materials. Experimental research on the behavior of FRP-reinforced concrete bridge decks in general have been limited, especially those on fatigue performance (El-Salakawy et al., 2005). Different reinforcement configurations were used, including isotropic reinforcement pattern (equal reinforcement ratios with a minimum of 0.3%; top and bottom in all directions) and orthotropic reinforcement pattern (transverse reinforcement ratio higher than that in the longitudinal direction). It was found that the damage caused by a moving wheel load was much more severe than the damage caused by a stationary pulsating load. The fatigue life of the isotropic steel-reinforced bridge decks under moving wheel load was about 20 times that of the orthotropic steel-reinforced ones (Pardikaris and Beim, 1988; Matsui et al., 2001). The fatigue life of the deck panels with the same static strength and reinforcement ratio increases as the effective depth increases, while the fatigue life of the deck panels with compressive reinforcement was shorter than that of the ones without it (Pardikaris and Beim, 1988). The rate of fatigue degradation in decks reinforced with FRP bars compared well with decks reinforced with steel bars. They had a linear variation in stiffness degradation even after 2 million fatigue cycles; which were assumed as 80% of the fatigue life of these decks (Kumar and GangaRao, 1998; Rahman et al., 2000). El-

Ragaby et al. (El-Ragaby et al., 2007) concluded that the GFRP-reinforced concrete bridge deck specimens had better fatigue performance and longer fatigue life (about 2.5 times) compared to steel-reinforced specimens. This may be due to the close value of the modulus of elasticity for GFRP composite bars and concrete and the linear–elastic behavior of GFRP bars up to failure. The proposed FRP-reinforcement ratio recommended by the updated version of Section 16 of the CHBDC (CAN/CSAS6-06) is adequate to meet the fatigue strength, serviceability, and fatigue life requirements of concrete bridge decks.

2.4.2 Principal factors affecting FRP external reinforcement durability

This section presents the results from some previous studies carried out to understand the influences of various chemical and physical environments on the integrity of bond between FRP and other materials. External reinforcement is likely to see a more varied environmental exposure than that of internal reinforcement including moisture cycling, chemical solutions, and UV. In adhesively bonded FRP-concrete systems, one face of the material is adhered to the concrete and one is exposed to the environment. Consequently, the exposure conditions (related to moisture) for the FRP composite are affected simultaneously by the surrounding environment and the underlying concrete. Highly porous concrete will readily transmit moisture (if available) to the bond line. Accumulation of salts at the bond line may cause large pore pressures to develop, thus reducing the bond capacity. Some results of bond performance between FRP and concrete under various conditions of exposure are presented in the following sections.

High temperature

One of the major concerns of applying FRP reinforcing materials for strengthening of concrete structures is the temperature dependence of both the FRP and of the FRP-concrete bond. Bond strength is expected to be influenced whenever temperature increases. Woods (Woods, 2003) conducted a comprehensive study to evaluate the performance of FRP-concrete bond under various simulated aggressive environments. The experiments consisted of testing, in shear, 364 unidirectional CFRP coupons each 33 mm wide and 3 mm thick bonded to 76 x 76 x 305 mm concrete blocks with a 152 mm bond length. In order to evaluate the influence of temperature, ovens at 60°C and 71°C were used. The test results showed a decrease in the fracture toughness values with an increase in the temperature. Gamage et al. (Gamage et al., 2005) conducted an investigation on the bond behavior of concrete members strengthened with CFRP under exposure to elevated temperatures. Observations and results obtained from this study indicated that the bond strength of CFRP-strengthened concrete members decreases with increasing temperature. The observed failure mode at epoxy temperatures exceeding 60°C was peeling-off of the CFRP sheet, while in specimens tested below 50°C, a combination of concrete rupture and adhesive failure was noted. It was concluded from comparison of the bond strengths at high temperatures that the rise in the epoxy temperature adversely affects the CFRP-concrete bond strength especially when the epoxy reaches a temperature between 60°C and 70°C where rapid strength loss happens. (Lublóy et al., 2005) conducted experiments to study the influence of elevated temperatures on the bond behavior of CFRP wires as well as externally bonded sheets. Concrete cover was one of the variables studied. Pull-out tests were carried out with a constant bonded length but various concrete covers (10, 20, and 30 mm) and temperatures (23, 50, 75, 100, 200, and 250°C). The results showed that three types of specimen failure modes were recognized: pull-out failure; splitting bond failure; and resin type failure under higher temperatures. Also, the resin showed softening in the temperature range of 100 to 200°C.

High humidity

As a part of a larger research program, (Woods, 2003) studied the bond durability and life expectancy of CFRP bonded to concrete under a combined influence of high temperature (37.7°C) and high humidity. The purpose was to generate conditions normally encountered in regions such as Florida. The objective was to understand how a combined effect of heat and water influences the properties of the FRP and its bond to a substrate. Tests were conducted at various exposure times and fracture toughness was evaluated both at the initiation and propagation of cracking. It was found that there was a large decrease in fracture toughness at the crack initiation load in this environment, with respect to the control specimen. From the larger test program conducted (Ghosh and Karbhari, 2006) over a 12-month period, the following conclusion can be drawn as pertaining to a high humidity exposure: The composite system showed high susceptibility to high temperature/humidity environmental conditions with the failure surface of all the pull-off samples being at the epoxy/concrete interface. For the specimens exposed to 95 percent RH at 37.7°C, one carbon fabric composite system, with high density carbon/epoxy structure had strength degradation of 78 percent. With the exception of two pultruded strip composite systems, which showed no degradation in bond performance, most other specimens showed strength degradations in the range of 20 to 30 percent. It appears that improper mixing of the resin or manufacturing flaws may possibly result in susceptibility of the composite systems to poor bond characteristics. Therefore, proper mixing is needed to ensure reliable performance of these new construction/rehabilitation materials under different environmental exposures. Literature indicated that exposure to 98 percent RH at 37.7°C had the second worst overall effect on bond performance after the freeze exposure condition.

Freeze-thaw conditions

Many concrete structures are often subjected to a large number of freeze-thaw cycles. Changes in the environmental temperature induce stresses in the adhesive layer due to differential thermal expansion. Also, freezing and thawing generally involve the action of water which may cause bond damage and premature bond failure. Thus, both degradation of the FRP material itself and deterioration at the FRP concrete interface have to be taken into consideration. Several researchers investigated the resistance of FRP bond to freeze-thaw conditioning. Green et al. (Green et al., 2000) conducted a study to examine the effects of freeze-thaw cycling on the bond between FRP and concrete. An experimental investigation was conducted using both single lap pull-off and bond beam specimens. Their experimental program was divided into two main phases. One part of the work was conducted using single lap CFRP-concrete joints loaded in shear. Modified flexural specimens were used in the other phase to examine similar bond parameters. Ghosh and Karbhari (Ghosh and Karbhari, 2006) conducted a study aimed to evaluate the effect of different environmental conditions, including freeze exposure, on the performance and durability of bond between FRP composite and concrete. Contrary to the findings of Green et al. (Green et al., 2000), the results from pull-off tests completed at 6-month cycle indicate that of all the environmental exposures, the maximum strength degradation was observed for freeze exposure condition at -17°C. Lower pull-off strengths indicated that the epoxy resin/adhesive matrix becomes brittle when exposed to low temperature. Moreover, a majority of the specimens failed either at the top layer of concrete or at the epoxy resin/adhesive bondline, which is also a good indication that the bond strength is too low and not enough to force failure into the concrete.

Saline environment

There are several reports indicating considerable degradation of FRP when subjected to salt solutions. Often, the deterioration is not attributed to salt attack but to other reactions, such as reactions with alkali from the concrete pore solution or resin plasticization by water uptake. However, there are some indications that an exposure to saline solution is more severe than an exposure to fresh water. Immersion in saltwater (5% NaCl solution by weight) at 73° F was another environmental exposure studied by (Ghosh and Karbhari, 2006) to evaluate the durability and behavior of FRP-concrete bond in marine environments. The results of specimens exposed to saltwater, tested after the 12- and 18-month cycles indicated that the specimens prefabricated carbon pultruded strip/epoxy adhesive system experienced the most degradation when subjected to saltwater, with a bond strength reduction of 69 percent after 12 months and that of 100 percent after 18 months of exposure. The specimens failed at the epoxy resin/concrete interface. (Woods, 2003) also studied the long-term performance of bond in a saline environment. Specimens were subjected to six aggressive environments, each resembling a different field condition; one of which was seawater (pH=8.3). They measured the mode II fracture toughness (both at initiation and during propagation) of FRP coupons bonded to concrete after subjecting them to a saline environment for a length of time. Of all six environments, seawater was found to have the most drastic effect on FRP bonds due to the fact that the concentration of chloride ions had a detrimental effect on the concrete itself which is the weak link in the FRP-concrete bond. The calcium hydroxide environment, on the other hand, proved to have the least effect on FRP bonds.

Alkaline environment

When FRP systems are used with concrete, for internal or external reinforcing, the fibers and matrix will be exposed to the high alkaline environment present in the concrete. In a study (Homam et al., 2000), performance of FRP coupons, FRP/FRP bond, FRP concrete bond, and the confinement efficiency of FRP after exposure to sodium hydroxide (NaOH) solutions with different concentrations were evaluated. The specimens were immersed in NaOH solutions at 38°C, using pH 10 and pH 12 concentrations for the FRP coupons and single lap bonded (SLB) specimens and pH 10, pH 12, and pH 13.7 concentrations for the FRP-bonded concrete prisms and FRP-wrapped concrete cylinders. In order to ensure constant alkalinity throughout the FRP-concrete bond-line, the cylinders and prisms were conditioned for a week before wrapping with FRP, curing and further exposure. The conditioning scheme consisted of a three-day drying period at 50°C until the specimens reached their constant dry weight, three days of submergence in respective alkaline solutions, and finally one day of drying at room temperature. After the specimens were conditioned, wrapped/bonded, cured for seven days and immersed in alkali solutions for seven, 14, 28 and 84 days, they were retrieved, rinsed with water, dried with a cloth and then tested for post-exposure residual mechanical properties. The test results indicated that the tensile strength of GFRP coupons showed a reduction of about 10 percent due to exposure to NaOH solution. For the CFRP coupons, however, the strength was actually found to have slightly increased. Alkali exposure had similar effects on CFRP-CFRP and GFRP-GFRP bonds, with a degradation of as much as 14 percent in the bond strength. FRP-FRP bonds displayed more susceptibility to environmental conditions than FRP coupons. The strength of the FRP-concrete bonds in different exposure environments followed a similar trend as the compressive strength of plain concrete.

In order to assess the effect of different environments on the FRP-concrete bond performance several mechanical tests were conducted on FRP rods exposed to accelerated environmental pre-conditioning, (Al-Dulaijan, 2001). The specimens were subjected to three different aqueous solutions: 10 percent ammonia at 80°C (AM), 0.6 percent acetic acid at 80°C (AC), and deionized water at 80°C (W), for a period of 28 days. The AM, AC and W solutions had pH values of 12.5, 2.92, and 7.0, respectively. It was found that the alkaline environment at high temperature was concluded to have the most severe effect on the bond of the FRP materials evaluated (glass/vinyl ester system), while the other two aqueous solutions, representing acidic and neutral environments, caused a lesser amount of degradation in the FRP bond. Environmental pre-conditioning can alter the surface smoothness of FRP rods and, in turn, influence the development of bond strength. This surface effect overshadows all other material influences when the friction controls the bond. Test results revealed that the carbon/epoxy system is less susceptible to degradation than the two other FRP composite systems (carbon/vinylester and glass/vinylester) used in the study.

2.4.3 Design and application recommendations

For design purpose, it is imperative for a design engineer to account for the durability parameter. Civil engineers, contractors, and designers must always be aware of the long-term behavior of FRP composites in design structures under different environmental conditions. For this reason, relatively large safety factors are commonly proposed to account for the environmental uncertainty. The extensive research performed on FRP reinforcements led to the publication of codes and handbooks on the use of the FRP reinforcing bars and prestressing tendons in the reinforcement and the rehabilitation of new and existing bridges and structures. Design codes/guidelines in North-America and Japan (ISIS-M03-01, 2001); ACI 440.1R, 2006); CAN/CSA-S6-06 (Edition 2010); CAN/CSA-S806, 2012); Japanese Society of Civil Engineers, 1997; ACI 440.3R, (2004); ACI 440.4R, 2004) provide strength reduction factors or material factors to account for the effects of environmental conditions, creep from sustained loads, and fatigue from repeated loads. In ACI 440 code, the design tensile strength of FRP is given by the following expression:

$$f_{fu} = C_E \cdot f_{fu}^* \quad (2.1)$$

where f_{fu} = design tensile strength of FRP bar; f_{fu}^* = guaranteed tensile strength of the FRP defined as the average tensile strength of less than three times its standard deviation; and C_E = environmental reduction factor (RF). The value of C_E depends on the type of fiber, exposure conditions, and application of FRP internally or externally to the concrete structures. The temperature effects are included in the value of C_E . In addition, the effects of applied stress during exposure are kept separate in ACI 440 due to insufficient data on combined weathering and applied stress. The environmental RF, C_E , is also applied to the design rupture strain. However, it is assumed that modulus is typically not affected by environmental conditions and hence the design allowable is taken to be the value reported by the manufacturer, assumed to be the population mean.

In other countries, such as Canada, Norway, and Japan, a similar reduction factor for tensile strength is proposed to account for the long-term durability of FRP. The durability-related RF for FRP used as internal or external reinforcement is incorporated as a single factor, which may lead to under-conservative or over-conservative design depending on the application and its environmental conditions. Similar to that in ACI 440, the effects of elevated temperature were not specified in the above factors, which may result in under-conservative design for some applications in

hot regions (Huang and Aboutaha, 2010). For Example, the allowable stress limits for GFRP bars recommended by North American guidelines and codes are presented in Table 2.2.

Table 2.2: Reduction factors for GFRP bars suggested by various codes and design guides

Code	(ACI 440.1R, 2006)	(CAN/CSA S6, 2006)	(CAN/CSA S608, 2012)
Properties	Average- 3σ	Average- σ	Average- 3σ
Reduction due to the environmental degradation	Environmental reduction coefficient' 0.70–0.80	ϕ_{FRP} strength factor' 0.55	ϕ_{FRP} strength factor' 0.75
Reduction due to the sustained load	NA	0.8–1.0	The maximum strain shall not exceed 0.002
Design limit for creep	0.2	0.25	0.3

σ is the standard deviation of the test population.

2.4.4 FRP performance in bridge decks and two-way slabs parking garages

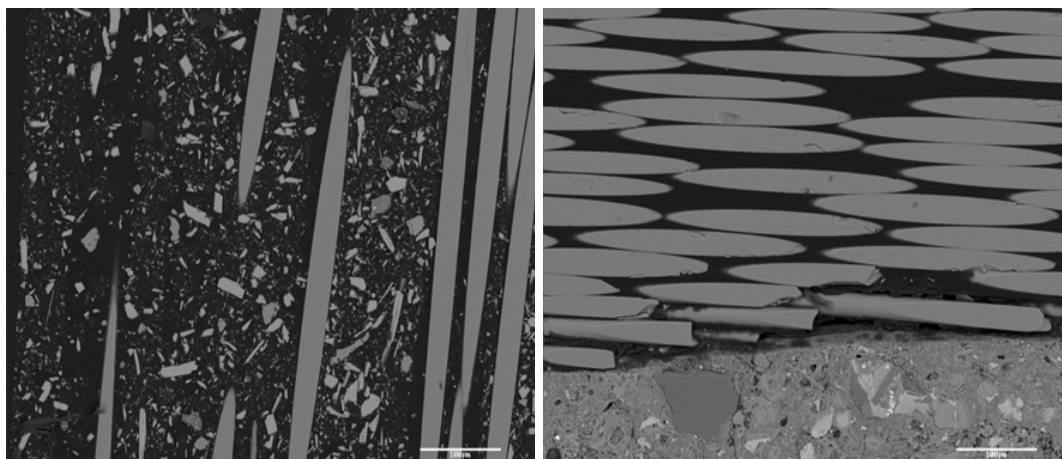
In 2004, ISIS Canada, a Canadian Network of Centers of Excellence, launched a major study to obtain field data with respect to the durability of GFRP in concrete exposed to natural environments. The objective of the study described in this chapter was to provide data on the performance of the GFRP that has been used in several structures across Canada. Concrete cores containing GFRP were taken from several five-to eight-year old exposed structures, and the GFRP was analyzed for its physical and chemical composition at the microscopic level.

Five ISIS Canada field demonstration structures built in five different provinces (Hall's Harbor Wharf in Nova Scotia, Joffre Bridge in Québec, Chatham Bridge in Ontario, Crowchild Trail Bridge in Alberta, Waterloo Creek Bridge in British Columbia) were exposed to a wide range of environmental conditions were chosen, (Figure 2.5). This choice corresponds to a wide range of environmental conditions that are representative of the Canadian climate. Three research teams from four Canadian universities performed microanalyses of the GFRP and surrounding concrete independently. They used a variety of analytical methods to (a) investigate whether or not the GFRP in concrete field structures had been attacked by alkali, and (b) compare the composition of GFRP removed from in-service structures to that of control specimens, which were saved from the projects and not exposed to the concrete environment, (Figure 2.6). Direct comparisons were carried out with "virgin" GFRP rods preserved under controlled laboratory conditions. Results indicate that no deterioration of GFRP took place in any of the field demonstration structures included in this study, and no chemical degradation processes occurred within the GFRP due to the alkalinity of concrete. The overall conclusion was that the GFRP is durable in concrete through its life cycle. The outcome of this work has led the second edition of CHBDC to permit GFRP as primary reinforcement and prestressing tendons in concrete structures (ISIS 2006).

FRP materials have been promoted to be a corrosion-free alternative to traditional steel reinforcement and strengthening material. Corrosion resistance is without a doubt the main motive and attraction to use FRP over steel. Application of FRP reinforcement in different structures has proved to be very successful to date. Based in the investigated study the durability performance of FRP materials is generally very good in comparison with other, more conventional, construction materials. However, it should be equally clear that the long-term durability of FRPs remains not fully understood. A large research effort is thus required to fill all of the gaps in knowledge. Research needs with respect to the durability of FRP materials in infrastructure applications were highlighted recently the ACI 440.



(a) Joffre Bridge (b) Waterloo Creek Bridge
Figure 2.5: Pictures from the five field demonstration projects considered



(a) Joffre Bridge (b) Waterloo Creek Bridge
Figure 2.6: SEM Longitudinal Section Micrographs from various core specimens around GFRP-concrete interface

Bouguerra et al. (Bouguerra et al., 2011) studies the behavior of FRP-reinforced concrete bridge deck slabs under concentrated loads. A total of eight full-scale deck slabs measuring 3000-mm long by 2500-mm wide were constructed. The test parameters were: (i) slab thickness (200, 175 and 150 mm); (ii) concrete compressive strength (35–65 MPa); (iii) bottom transverse reinforcement ratio (1.2–0.35%); and (iv) type of reinforcement (GFRP, CFRP, and steel). The slabs

were supported on two parallel steel girders and were tested up to failure under monotonic single concentrated load acting on the center of each slab over a contact area of 600×250 mm to simulate the footprint of sustained truck wheel load (87.5 kN CL-625 truck). All deck slabs failed in punching shear. The punching capacity of the tested deck slabs ranged from 1.74 to 3.52 times the factored load (P_f) specified by the Canadian Highway Bridge Design Code (CHBDC) CAN/CSA S6-06. Besides, the ACI 440.1R-06 punching strength equation greatly underestimated the capacity of the tested slabs with an average experimental-to-predicted punching capacity ratio (V_{exp}/V_{pred}) of 3.17.

El-Gamal et al. (El-Gamal et al., 2005) investigated the behavior of edge restrained concrete bridge deck slabs reinforced with glass FRP and carbon FRP bars was investigated. Six full-scale deck slabs 3000 mm long x 2500 mm wide x 200 mm deep were constructed and tested to failure in the laboratory. Three deck slabs were reinforced with glass FRP (GFRP) bars, two deck slabs were reinforced with carbon FRP (CFRP) bars, and the remaining slab was reinforced with steel bars as control. The test parameters were the reinforcement type and ratio in the bottom transverse direction. The deck slabs were supported on two steel girders spaced at 2000 mm center-to-center and were subjected to a monotonic single concentrated load over a contact area of 600×250 mm to simulate the footprint of sustained truck wheel load (87.5 kN CL-625 truck) acting on the center of each slab. It was observed that the mode of failure for all deck slabs was punching shear with carrying capacities of more than three times the design factored load specified by the Canadian Highway Bridge Design Code. It was also concluded that the maximum measured crack widths and deflections at service load level were below the allowable code limits. The maximum measured deflection at the service load level was less than 1.5 mm, which is well below the allowable code limits. In addition, for these deck slabs, the increase of the FRP reinforcement ratio has an insignificant effect on the deflection behavior.

Hassan et al. (Hassan et al., 2000) studied the punching shear behavior of two full-scale models of a portion of highway bridge deck slab reinforced with FRP reinforcement. Each model consisted of three continuous spans of 1.8 m each and two cantilevers, with overall dimensions of $7.2 \times 3.0 \times 0.2$ m as shown in Figure 2.7. The first model was reinforced totally with carbon FRP bars. The second was reinforced with hybrid glass FRP in the top and steel reinforcement in the bottom. The models were tested under static loading up to failure using concentrated loads acting on each span of the continuous slab and the two cantilevers. They found that the failure was due to punching shear at a much higher load than the ultimate design load specified by the AASHTO and the CHBDC codes. In addition, the presence of top reinforcement in continuous bridge deck slab had a negligible effect on the punching shear capacity. To satisfy serviceability and ultimate capacity requirements for span-to-depth ratios ranging between 9 and 15, they recommended the following;

- For CFRP reinforcement, they recommended to use 0.3% as top and bottom reinforcement in all directions.
- For GFRP reinforcement, they recommended to use 1.2% for the bottom reinforcement and 0.6% for the top reinforcement in the transverse direction, as well as 0.6% as top and bottom reinforcement ratios in the longitudinal direction.

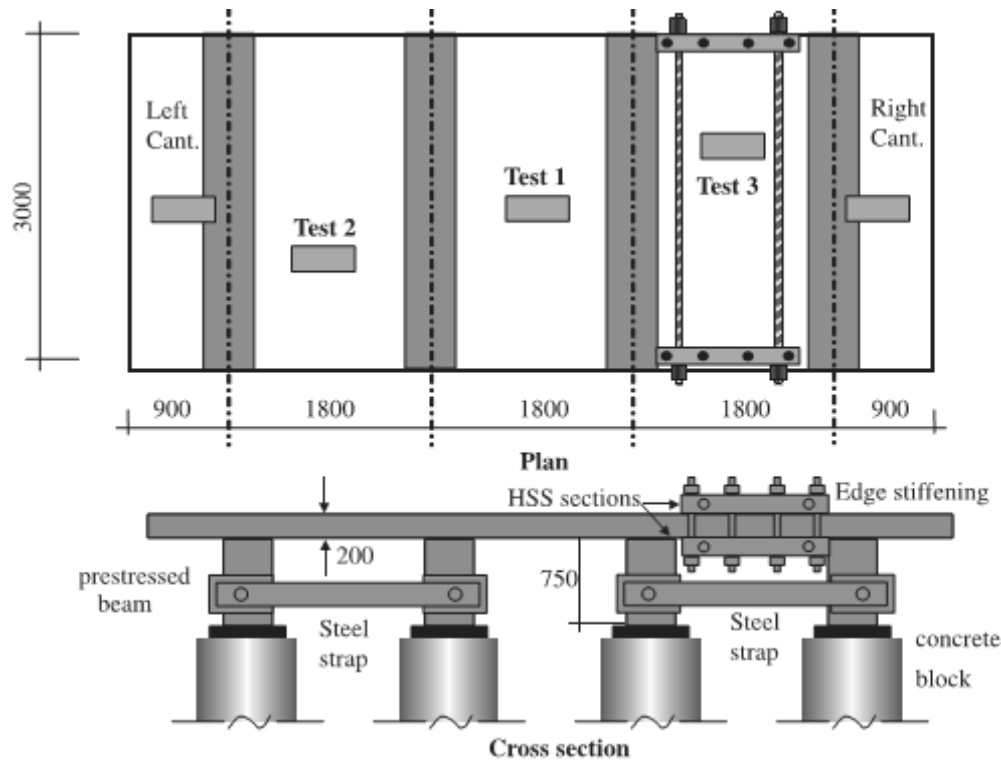


Figure 2.7: Full-scale bridge deck panel (Hassan et al., 2000)

Lee et al. (Lee et al., 2009) tested six specimens under punching shear test represent two-way slabs. The slabs had a side dimension of 2300 mm square and thickness of 150 mm. The slab was loaded with either equal concentrated loads around the perimeter to simulate a uniformly distributed load on the test specimen. The main variables were the reinforcement material, the concentration of reinforcement around the column, and the presence of steel fibers in the concrete. Four specimens were reinforced with uniform and banded distribution (within a distance $1.5h$ from the column faces, where h is the slab thickness) GFRP bars while two control steel specimens for comparisons. The flexural reinforcement ratios of the specimens were varied between 1.18% and 3%. The test results indicated that concentrating the top mat of flexural reinforcement within a distance 1.5 times the slab thickness from the column faces resulted in slightly higher punching shear strength, more uniform distribution of strains in the top flexural bars and better crack control compared to the companion slab with a uniform distribution of the same amount of reinforcement. The increase in punching shear strength due to the banded distribution of top reinforcement was 5% and 11% for the steel and GFRP specimens, respectively. In addition, the punching shear failure plane for the slabs with banded reinforcement surfaced at a greater distance from the column faces. However, excessive concentrations of the reinforcement ($\rho = 3\%$) seems to be ineffective in increasing the punching resistance of GFRP-reinforced concrete slabs.

Hassan et al. (Hassan et al., 2013) investigated the punching shear behavior of two-way flat slabs reinforced with glass fiber-reinforced polymer (GFRP) bars. A total of 17 full-scale interior slab-column specimens measuring 2500 mm $\times$ 2500 mm reinforced with GFRP and steel bars were constructed and tested under concentric loads until failure. The test parameters were: (i) reinforcement type (GFRP and steel) and ratio (0.34% to 1.66%); (ii) slab thickness (200 mm and 350 mm); (iii) column dimensions (300 mm $\times$ 300 mm; 450mm $\times$ 450 mm); (iv) concrete

strength (30 to 47 MPa); and (v) GFRP compression reinforcement crossing the column cross section. The test results indicated that regardless of the reinforcement type and ratio, the final mode of failure in all the tested specimens was punching shear failure (Figure 2.8). The slab specimens with low reinforcement ratios, however, showed large deformations before the punching shear failure. In addition, at punching failure, neither concrete crushing in the compression zone nor rupture of GFRP reinforcement was observed. The punching shear capacity of the compression zone was controlled by concrete splitting tension rather than concrete crushing. This paper contributed to the first field implementation for GFRP bars in flat slab parking garages through a demonstration area (350 m²) in Hôtel de Ville parking garage, and La Chancelière parking garage (the world's first flat-slab parking garage totally reinforced with GFRP bars) in Québec, Canada, in 2010, and 2011, respectively.

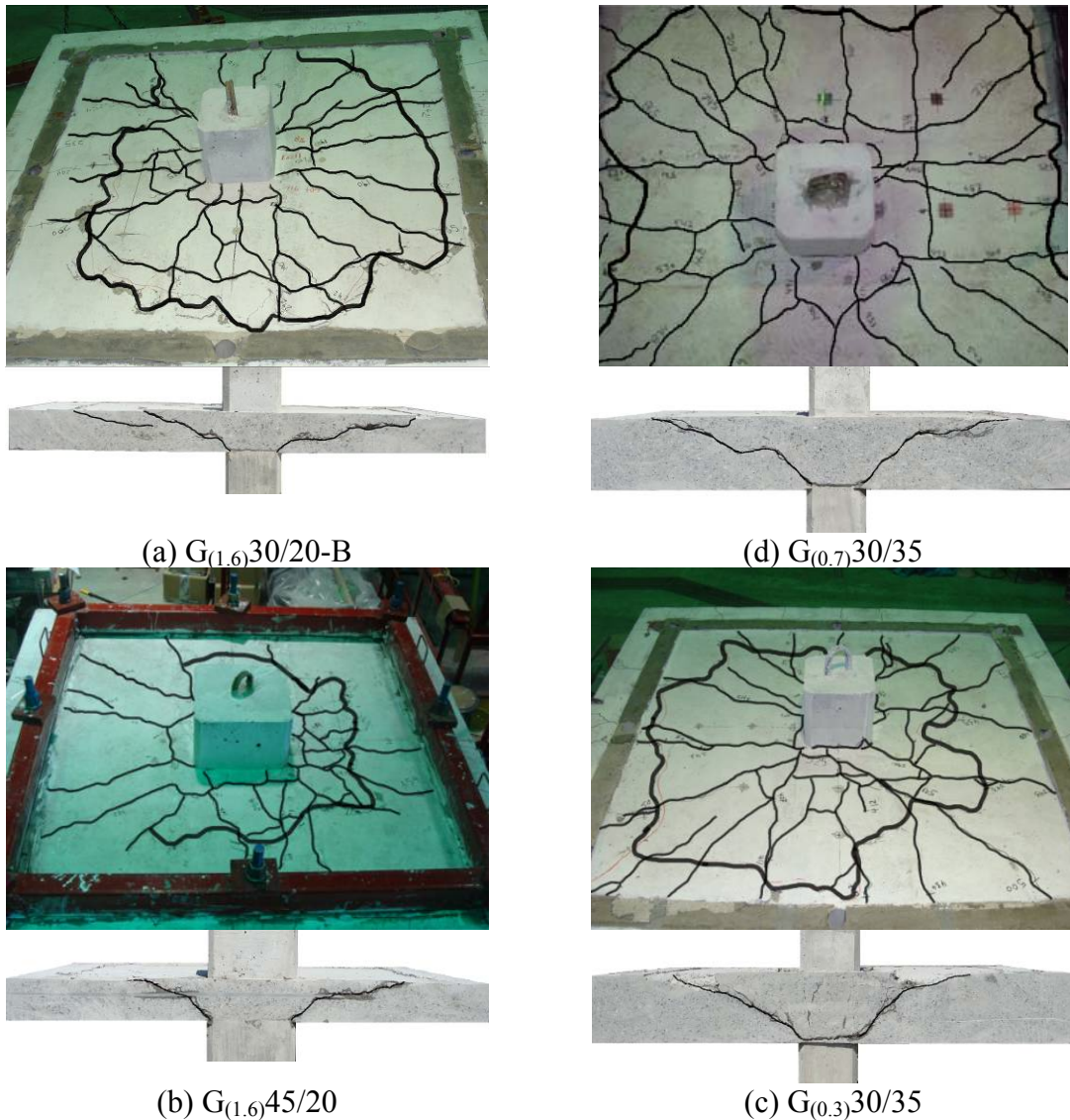


Figure 2.8: Typical punching-shear failure for the tested specimens (Hassan et al., 2013)

2.5 Design Codes and Guidelines

A number of committees from professional organizations around the world have addressed the use of FRP bars in civil structures. These have published several guidelines and/or standards relevant to FRP as primary reinforcement for structural concrete. The recommendations ruling the design of FRP RC structures currently available are mainly given in the form of modifications to existing steel RC codes of practice, which predominantly use the limit state design approach. Such modifications consist of basic principles, strongly influenced by the mechanical properties of FRP reinforcement, and empirical equations based on experimental investigations on FRP RC elements. In North America, several codes and design guidelines for concrete structures reinforced with FRP bars have been published from 2000 to 2010. In 2000, the Canadian Highway Bridge Design Code (CHBDC) [CAN/CSAS6-00, (CSA 2000)] has been introduced including Section 16 on using FRP composite bars as reinforcement for concrete bridges (slabs, girders, and barrier walls). Design manual (ISIS-M03, 2001) for reinforcing concrete structures with FRP was presented by the Canadian Network of Centres of Excellence on Intelligent Sensing for Innovative Structures (ISIS). In 2002, CAN/CSA-S806, 2002) has been published by the Canadian Standards Association (CSA 2002) for design and construction of building components with FRP bars. The American Concrete Institute (ACI) introduced the first and second guideline (ACI 440.1R) for the design and construction of concrete reinforced with FRP bars in 2001 and 2003, respectively. The ACI 440.1R design guidelines are primarily based on modifications of the ACI-318 steel code of practice (ACI 318, 2002), as a result of the valuable, enormous and great research efforts on different types of FRP-reinforced concrete structures in worldwide during the last decade, the aforesaid North American codes and design guidelines have been updated and modified to encourage the construction industry to use FRP materials (ISIS-M03, 2007; CAN/CSAS6, 2006; CAN/CSAS6, 2006 edition 2010; ACI 440.1R, 2003; ACI 440.1R, 2006).

Nowadays, the (CAN/CSA-S806, 2012) is the most recently issued Canadian guidelines on the design and construction of building components with FRP. The CSA S806 has been completely revised. Many of its provisions have been improved based on the latest research results and experience in the field. The (CSA S806, 2012) contains new provisions on (1) Punching shear at slab-column connections with or without moment transfer; (2) Confinement of columns by FRP internal ties or hoops; (3) Design of FRP reinforced member for combined effects of shear, torsion and bending; (4) Reinforcement development length and detailing; (5) Strut and tie model for deep beams, corbels and brackets; (6) Shear strengthening of reinforced concrete members by externally bonded reinforcement; and FRP retrofit of reinforced concrete members for enhanced seismic resistance. The new standard covers all the basic design requirements for FRP reinforced and retrofitted structures. In addition to the design of concrete elements reinforced or prestressed with FRP, the guidelines also include information about characterization tests for FRP internal reinforcement. As for the predominant mode of failure, the (CSA S806, 2012) remarks that, “All FRP reinforced concrete sections shall be designed in such a way that failure of the section is initiated by crushing of the concrete in the compression zone”. In this code, new design equations are included for design punching shear capacity of FRP-RC flat slab. Also, it is of interest to mention that this code permits using FRP bars in columns and compression members.

In order to establish stringent guidelines and values for FRP manufacturers and quality control mechanisms for owners to ensure a high comfort level of product supplied, ISIS Canada together

with the manufacturer had initiated the “Specifications for product certification of FRP’s as internal reinforcement in concrete structures” (ISIS Canada Corporation 2006). This document was the basis for the new standard (CSA S807, 2010) on Specification for Fiber Reinforced Polymer (FRP). This Standard covers the manufacturing process requirements of fiber-reinforced polymer (FRP) bars or bars that are part of a grid for use in non-prestressed internal reinforcement of concrete components of structures (e.g., bridges, buildings, and marine structures). The FRP bars are classified on the basis of their fibers, strength, stiffness, and durability.

2.6 Recent FRP Field Applications

2.6.1 Highway bridge structures

The corrosion of steel reinforcing bars in concrete bridge decks leads to excessive cracking, spalling, reduced strength, and loss of structural integrity. That represents a major problem in terms of rehabilitation costs and traffic disruption. Concrete bridge decks deteriorate faster than any other bridge components because of direct exposure to the environment, de-icing chemicals and ever-increasing traffic load. In North America, around half of the maintenance budget of the Ministry of Transportation (MT) is spent on concrete structures damaged by corrosion of steel. Therefore, since the late 1990s, the Structures Division of the MT at different provinces has been interested in building more durable bridges with an extended service life of 75–150 years. For example, the MT at Québec (MTQ), Canada has carried out, in collaboration with the University of Sherbrooke (Sherbrooke, Québec), several research projects utilizing the straight and bent non-corrodible FRP bar in concrete deck slabs and bridge barriers (El-Salakawy et al., 2003b; Mohamed and Benmokrane, 2012b). The use of FRP bars as reinforcement for concrete bridge provides a potential for increased service life and economic and environmental benefits.

In the last ten years, the FRP bars have been used in hundreds of bridge structures across Canada and USA. These bridges were designed using the Canadian Highway Bridge Design Code or the AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings. Straight and bent FRP bars (carbon or glass) were used mainly as internal reinforcement for the deck slab and/or for the concrete barriers and girders of these bridges. In general, all the bridges that included with FRP reinforcements though the ten years ago are girder-type with main girders made of either steel or prestressed concrete. The main girders are simply supported over spans ranging from 20.0 to 90.0 m. The deck is a 200 to 260 mm thickness concrete slab continuous over spans of 2.30 to 4.0 m. Most of these bridges have been reinforced with the glass FRP bars as a result of their relatively low cost compared to other types of FRPs (carbon and Aramid). The FRP bars were used mainly as reinforcement to the deck slabs, barriers, and girders. Figure 2.9 shows some field application of FRP in highway bridge decks.

2.6.2 Water tanks

Reinforced concrete (RC) tanks have been used for water and wastewater storage and treatment for decades. Design of these tanks requires attention to not only strength requirements, but also crack control and durability. RC water treatment plant structures are subject to severely corrosive environments as a result of using the chlorine to treat the wastewater before it is released. Therefore, the challenge for the structural engineer and municipalities is to design these structures us-

ing noncorrosive fiber-reinforced polymers (FRP) reinforcing bars. The first worldwide concrete chlorination water treatment tank totally reinforced with FRP bars was designed in 2010 and the construction started and finished in 2012. The project is located in Thetford Mines city, Quebec, Canada, and it is considered as one component of the water treatment plant for the municipality. The volume capacity of the tank is 4500 m^3 , and it has the dimensions 30.0 m wide, 30.0 m long and 5.0 m high. The structural system of the tank is a rectangular underground tank resting on a raft foundation that supports the vertical walls and top slab. The design of the tank was made according to CAN/CSA-S806-02, Design and Construction of Building Components with Fiber-Reinforced-Polymers. This included the use of High Modulus GFRP reinforcing bars (Grade III, CSA S807) as main reinforcement for the foundation, walls and top slab. The tank is well instrumented at critical locations for strain data collection with fiber-optic sensors. Figure 2.10 shows the FRP bar reinforcements in the vertical walls and overview of the complemented FRP tank. The field test results under actual service conditions for the strain behavior in the FRP bars at different location in the tank are indicated by a significant value less the 1.0 % of the ultimate strain. In conclusion, the construction procedure, serviceability performance under real service conditions (water and earth pressure), and monitoring results of the FRP-reinforced walls and slabs of the tank, in terms of strain, cracking and deflection were very conservative and satisfactory when compared with the serviceability requirements and strength needed.

2.6.3 GFRP soft eyes in tunnels

Building tunnels with tunnel boring machines (TBM) is today state of the art in different ground conditions. Launching and receiving the TBM in shafts and station boxes has recently required a considerable construction effort. Breaking through the steel reinforced walls of the excavation shaft with a TBM required extensive measurements and preparation works (Schürch and Jost 2006). FRP is an anisotropic composite material with a high tensile strength in axial direction and a high resistance against corrosion. The anisotropy of the material is quite advantageous at excavation pits for the starting and finishing processes at automated excavation like tunnel boring machine (TBM) and Pipe jacking (Figure 2.11). Therefore, using FRP bars in reinforced walls and piles of the excavation shaft allows the designer and contractor today to find innovative solutions for the well-known situation and save time and costs on site. Soft eyes consist usually of bore piles or diaphragm walls that are locally reinforced with GFRP bars and stirrups (Figure 4). The sections below and above the tunnel opening are reinforced steel bars. Depending on the designer and contractors' preferences, full rectangular sections are built out of GFRP bars or the fiber reinforcement follows the tunnel section more closely, resulting in a circular arrangement of the GFRP links and similar adjustments for the vertical bars. Building the corresponding reinforcement cages out of GFRP bars on site requires the same working procedures as for an equal steel cage, (Figure 2.12). The necessary bars are tailor made and delivered to the site where the assembly takes place. The bars are fixed together with binding wire, cable binders or similar products. U-bolts are used for clamping bars together when high loads have to be transferred over a connection. This is the case for example in the connection between vertical GFRP bars and the corresponding steel bars which have to carry the dead load of the reinforcement cage during the lifting process and lowering of the cage into the trench. Welding as is commonly done with steel reinforcement in such situations is not possible with GFRP bars (Schürch and Jost, 2006).

2.6.4 Parking garages

The need for sustainable structures has motivated the Public Works and Government Services Canada (PWGSC) in the use of FRP bar as internal reinforcement in concrete infrastructure applications. One of the most important successful applications is using FRP bar to reinforce parking garages. An agreement between PWGSC and the University of Sherbrooke was reached to reconstruct the interior structural slabs of the Laurier–Taché parking garage (Hull, Quebec) using carbon and glass FRP bar, (Figure 2.13). The design was made according to (CAN/CSA-S806, 2002). This project allowed direct field assessment and long-term monitoring of FRP composite bars in a structure subjected to harsh environmental and loading conditions. In 2010, the new large parking garage (La Chancelière parking garage, area 3,000 m²) in Quebec City was designed and constructed using the FRP bar, (Figure 2.14). This design was made according to the (CAN/CSA-S413, 2007) for parking structures and (CAN/CSA-S806, 2002) for design and construction of building components with fiber reinforced polymers. The two-way flat slabs of La Chancelière had a maximum span of about 9.0 m. The thickness of the slabs was 250 mm, which increased to 355 mm over the columns through the drop panels, (Figure 6). The increased thickness over the columns was required to satisfy the punching stresses around the columns' area. The punching strength of the two way slabs was verified using the new punching equations that are being incorporated in the new version of the S806 Standards (2012), (Benmokrane et al., 2012).



(a) FRP decks/app slabs/ barriers – Skagit River – BC MOT (2009)



(b) FRP decks and barriers – Gateway Blvd/23rd Ave – Alberta (2009)



(c) Reinforcement of the bridge deck, 410 overpass bridge Quebec, Canada
Figure 2.9: Recent FRP-reinforced concrete bridges



(a) FRP reinforcement bars of the vertical wall



(b) Over view of the tank

Figure 2.10: FRP-reinforced concrete tank, Quebec, Canada



Figure 2.11: TBM cutting through FRP-reinforced concrete drilled shaft wall



(a) Overview of the GFRP soft eyes



(b) Handling and lifting the GFRP soft eyes

Figure 2.12: GFRP reinforcement for soft eyes





Figure 2.13: Laurier-Taché Parking Garage



Figure 2.14: La Chancelière Parking Garage

2.7 Basalt Fiber

2.7.1 Basalt fiber and the comparison between basalt fiber and other fibers

Currently, common FRP composites used in engineering are CFRP, AFRP and GFRP. All of them have some limitations: (1) Carbon fiber: ideal combination property, but high price, and PAN-precursor technology is nearly monopolized and controlled abroad; (2) Glass fiber: low price, localized, but low modulus of elasticity and insufficient durability; and (3) Aramid fiber: high strength and elasticity modulus, especially perfect toughness and shock resistance, but low moisture proof and ultraviolet resistance, low compressive strength and modulus, furthermore, high price, precursor technology is also nearly monopolized and controlled abroad.

Basalt fiber is made from volcanic, melted under high temperature as 1500°C, and drawn speedily to be continuous fiber. There are many kinds of basalt fiber reinforced plastics (BFRP), which take basalt fiber as reinforcements. Compared with CFRP, GFRP and AFRP, BFRP has many advantages and special properties: (1) The raw material is natural rock, a purely natural inorganic material, with outstanding mechanic and perfect durability, and good flexibility in all environments; (2) Rich raw material, with proprietary intellectual property rights and becoming localized; (3) Low cost: current price is about 1/5~1/8 of carbon fibers, and expected to approach to glass fiber's level, which is likely to break the price bottleneck of FRP application; (4) ultimate strain and ductility that are suitable for earthquake resistance for structure; (5) good fatigue resistance, and strong bonding force with resin.

2.7.2 BFRP studies

Researchers investigated the durability and performance under elevated temperature of basalt fiber (Sim et al., 2005). They found that when immersing the fibers into an alkali solution, both the basalt and glass fibers lose their volumes and strengths; however, the carbon fiber did not show significant strength reduction. From the accelerated weathering test, the basalt fiber provided better resistance than the glass fiber (Sim et al., 2005). Another researcher studied the chemical durability and mechanical properties of basalt fiber and its epoxy resin composites (Wang et al. 2008). After exposure to alkali solutions for three months, including saturated Na₂CO₃ solution, 10% NaOH, and 10% NH₃.H₂O, the BFRP modulus did not change while the strength is reduced by 40% (Wang et al. 2008). Other researchers conducted studies on the durability of both the basalt fibers without resin protection and BFRP bars under hygrothermal and alkaline environment (Li et al. 2012). They also investigated the fatigue performances of BFRP reinforcing bars. The scanning electron microscopy revealed serious corrosion of the bare basalt fibers and tests indicated serious degradation in their tensile properties. However, the basalt fiber reinforcing bars showed better durability performance. Another study investigated the degradation of tensile properties of BFRP and the related hybrid FRP tendons under salt solution (Wang et al. 2012). The researchers concluded that the degradation of tensile strength of BFRP tendons was proportional to the increase of stress level, while the corresponding modulus was relatively constant. The study also indicated that the BFRP tendons under the stress level of 0.6fu after 63 days aging maintained more than 90% of its tensile strength showing good resistance to salts corrosion (Wang et al. 2012).

Brik (Brik 2003) studied the bond strength between the modified basalt bars, and compared the ultimate moment capacity of basalt reinforcement bar reinforced concrete beams, and their calculated ultimate moment capacities according to ACI 440.1R (2001). They concluded that bond between the modified basalt bars and the concrete was extremely good due to following observations; the experimental ultimate moment was much higher than the first cracking moment in all the tested beams, the deflections were considerable indicating adequate ductility, and most of the beams had primary flexural failures and few beams had secondary shear failures. Therefore, they recommended that basalt bars are suitable for use in reinforced concrete structures.

Banibayat and Patnaik (Banibayat and Patnaik, 2013) performed creep rupture tests on (BFRP) reinforcing bars of 4.3 mm (0.167 inch) diameter at load levels between 25 and 80 percent of the corresponding ultimate tensile strength. Simulated environmental conditions using alkaline solution with pH of about 13 was used. The tests were conducted at an elevated temperature of 60°C.

Alkaline environment was used to simulate exposure of the FRP bars to the alkaline environment of concrete, and the elevated temperature was used to accelerate the degradation process. The following creep rupture properties of BFRP bars were established as a result of this study:

- i. A 50-year ultimate creep rupture strength coefficient of 0.18 was found to be suitable for basalt FRP bars. Similar coefficients were also determined for other service life durations. For example, the ultimate creep rupture strength coefficient for 5-year service life is 0.28.
- ii. The ACI 440.1 approach of using one creep rupture strength limit corresponding to 50 year service life regardless of the length of the service life leads to conservative designs for structural elements (such as seawalls) that may be designed for 5 to 10 years of service life.
- iii. ACI 440.1 recommends the use of a single safety factor of 1.67 for all service life durations up to 50 years. If safety factors are determined based on service life duration, a much more efficient use of FRP materials is possible. It is recommended that a safety factor of 1.2 be used in designs of structural elements with service life of 50 years while a safety factor of 1.67 be retained for 5 year service life. A variable set of safety factors are suggested for intermediate service lives.
- iv. The million hour (114 year) creep coefficient of BFRP bars was estimated to be 13%, which is marginally smaller than that for other AFRP materials.

Urbanski et al. (Urbanski et al., 2013) presented some chosen results of pilot research on the series of simply supported beams under flexure, reinforced with BFRP bars, compared to the reference beams with steel reinforcement. The tested beams were made of C30/37 concrete and reinforced with basalt bars with 8 mm diameter having and tensile strength evaluated from the tensile tests. The analysis of the beam deflection and cracking behavior has been presented. The results show the different character of the load-deflection relationship of basalt reinforced beams compared to traditionally steel reinforced beams, as well as the significant influence of the type and quality of anchoring on the process of basalt bars tensile process. They concluded that the bilinear stress-strain dependence for a steel reinforcement, in contrast with basalt reinforcement has a linear dependence until the entire the beam section load capacity has been exhausted. In addition, it was noted that critical load for tested beams reinforced with BFRP bars was much greater than the carrying capacity of beams with conventional steel reinforcement, which arose from the different degrees of mechanical reinforcement in both types of beams. The failure of beams with BFRP reinforcement did not occur suddenly and this effect was a result of transformation of the beam into a tie system because of flexural basalt reinforcement remained unbroken. Deflections of beams with BFRP reinforcement were significantly higher than the reference beam deflection, due to the much lower modulus of BFRP bars compared to steel bars. Deformation of the reinforcement of concrete beams with basalt reinforcement were considerably higher (average of 3 to 4 times) than the beams with steel reinforcement. However, in the final phase of the loading the difference decreased to 40% due to the phenomenon of plasticity in the beams of conventional RC beams. Average width of cracks on the section constant cross-section in beams with basalt reinforcement was 4 times higher than in the reference beams. Since the width of the cracks is primarily a function of the deformation of the reinforcement and the concrete between adjacent cracks, due to the much greater deformations in the reinforcement and the surrounding concrete with reinforcement for basalt beams, relatively to reference beams, the above phenomenon is as expected. Due to the relatively lower elasticity modulus of basalt rods,

compared to steel ones, both: the deflection and width of cracks can be a major factor in the designing the BFRP reinforced concrete beams.

El Refai (El Refai, 2013) presented the test results of a durability study on a novel basalt fiber-reinforced polymer (BFRP) bar-anchor system. The BFRP bars were exposed to saline and alkaline solutions for 10 weeks before being anchored and tested under static and fatigue loading. Unconditioned basalt, glass, and carbon specimens were also tested and served as controls. Regardless of the fiber material, all the bar-anchor systems withstood their ultimate tensile loading with no sign of slippage or premature failure in the anchor zones. Conditioning of the BFRP bars in saline and alkaline solutions resulted in 7 and 9% decreases, respectively, in the systems' tensile capacity. A decrease of 11% in the modulus of the alkali-conditioned BFRP specimens was also observed. The fatigue test results showed that the fatigue life of the bar-anchor system was primarily affected by the applied stress range. In addition, continuous immersion of the BFRP bar in the alkaline solution increased its tendency to fracture prematurely in the anchor zone. The fatigue limit of the BFRP bar-anchor system was determined to be 4% of its ultimate capacity, compared with 3 and 10% for the glass and carbon systems, respectively. Cyclic loading had a minimal effect on the tensile strength and the modulus of the BFRP specimens. However, the fatigue life of the bar-anchor system was significantly affected by the stress range. The life decreased as the applied stress range increased. Log-linear relationships between the fatigue life and the stress range applied were proposed for fatigue design. It is also recommended that a stress range of 80 MPa be considered as the anchor limit of the BFRP bar-anchor system when unconditioned BFRP bars are used. When the bars are subjected to saline or alkaline exposure, this value should be assumed as 60 or 40 MPa, respectively. Although the fatigue performance of the BFRP bar-anchor system is poorer than that of the CFRP system, the former can still be considered in applications involving low to moderate levels of fatigue loading to avoid the high cost of carbon fiber composites. Finally, basalt fibers might be a good alternative to traditional glass fibers in manufacturing fiber-reinforced composites. The durability and fatigue performance of BFRP bar-anchor systems could be enhanced by improving the manufacturing quality of the BFRP bars in terms of the fiber, matrix, and fiber/matrix interface, as observed in this study. In addition, the author recommended to investigate the durability and long-term investigations that consider other types of environmental exposure and other gripping mechanisms for the future.

2.8 Summary

The observations and the outcomes from the different field applications reported in this paper can be summarized as follows: (1) Corrosion resistance is without a doubt the main motive and attraction to use FRP over steel; (2) Applications of FRP reinforcement in different structures have proved to be very successful to date; (3) From the construction point of view it was felt by the construction personnel that the lightweight of the FRP reinforcements were easy to handle and place during construction; (4) Concrete bridges, water tanks, soft eye tunnels and parking garages provides an excellent opportunity to the use FRP.

CHAPTER 3

CHARACTERIZATION AND DURABILITY OF GFRP BARS

3.1 Introduction

Glass fiber-reinforced polymer (GFRP) materials have emerged as a practical alternative material for producing reinforcing bars for concrete structures. This is due to their relatively low cost-to-performance ratio and noncorrosive nature compared to traditional steel reinforcing bars. In addition, GFRP materials exhibit properties, such as high tensile strength, which make them suitable for use as structural reinforcement. However, their durability in an alkaline environment is still of concern and factors that can affect the long-term behavior of GFRP materials have to be investigated. Considerable research has been conducted in the past decade to assess the suitability of FRP reinforcement in reinforced concrete structures. The research work had highlighted on the short-term performance of FRP reinforced concrete structures or on the durability of FRP reinforcing bars subjected to aging in alkaline solution. Some researchers have reported on the durability of FRP bars embedded in moist concrete, which simulate the real conditions of application, and on the adverse effects of the presence of cracks and microcracks in the FRP bars on their long-term durability (Robert et al. 2009). It was recognized that FRP bars, especially GFRP bars, were susceptible to attack under exposures to moisture, alkaline solutions, and elevated temperature. In particular, the alkaline environment of concrete and moisture absorption could affect the durability of GFRP bars. The moisture diffusion into FRP composites could be influenced by anisotropic and heterogeneous character of the material. Along with diffusion into the matrix, wicking through the fiber/matrix interface in the fiber direction could be a predominant mechanism of moisture ingress (Robert and Benmokrane 2010).

This chapter presents the physical and mechanical properties of three different grades of GFRP bars. These bars are named as low modulus (LM); (Grade I-40GPa), standard (STD); (Grade II-50GPa), and high modulus (HM); (Grade III-60GPa), as classified according to (CSA-S807, 2010) “specifications for fiber reinforced polymers”. These bars are sand coated GFRP bars of different bar size (No. 5, No. 6, and No. 8), (No. 5 and No. 6), and (No. 5, No. 6, and No. 8) for LM, STD, and HM, respectively, Figure 3.1. The study is carried out using multiple performance data obtained from mechanical and physical characterization. Tensile, physical, and microstructural properties are investigated after preloading to evaluate the effects of cracking on short-term performance of preloaded GFRP bars. The long-term durability of preloaded bars is also investigated by accelerated aging of preloaded bars in alkali solution at high temperature.



Figure 3.1: Sand coated GFRP bars

3.2 Procedures

3.2.1 Physical Properties

The physical properties of the GFRP bars were determined according to the (ACI-440.6M, 2008) “Specification for Carbon and Glass Fiber-reinforced Polymer Bar Materials for Concrete Reinforcement” (when applicable) and (CSA-S807, 2010) “Specifications for Fiber Reinforced Polymers.” Fiber content, transverse coefficient of thermal expansion, void content, water absorption, cure ratio, and glass transition temperature (T_g) were determined. Table 3.1 shows test method, number of specimens and specified limits of the each property.

a) Fiber content

The glass fiber content was determined by pyrolysis according to (ASTM D 3171, 2011) – “Constituent Content of Composite, Method I: Procedure G”. Three inch-long samples were accurately weighed (W_T) and heated at 600°C for 5 hours. Thereafter, the sand (W_S) and glass fiber (W_F) weights were determined. The fiber content by weight was calculated using Eq 3.1, and shown in Table 3.2.

$$F \text{ (Fiber Content by weight)} = 100 \cdot W_F / (W_T - W_S) \quad (3.1)$$

b) Transverse coefficient of thermal expansion

The transverse coefficient of thermal expansion, α , was determined according to (ASTM E 831, 2012) – “Linear Thermal Expansion of Solids Materials by Thermo-mechanical Analysis

(TMA)”. Three samples, 3 to 6 mm thick, were placed under the probe and the measurements conducted between -30° and 60°C with a heating rate of 3°C/min. The results of all the GFRP specimens were reported in Table 3.2.

c) Void content

For all the GFRP grades the void contents were determined according to (ASTM D 2734, 2009) – “Void Content of Reinforced Plastics”. The void content, V , was obtained by comparing the density of the material measured (ρ_{meas}) and the theoretical density (ρ_{theor}) calculated from the density of the fibers and resin matrix (including all the additives). The fiber content was obtained from Eqn. 3.1. The theoretical density of the material was calculated by using the following equation:

$$\rho_{theor} = 100 / (R/\rho_R + F/\rho_F) \quad (3.2)$$

where R , ρ_R , F , and ρ_F were the weight percentage of resin, the density of the resin, the weight percentage of fiber and the density of fiber, respectively. The V was equal to:

$$V = 100 (\rho_{theor} - \rho_{meas}) / \rho_{theor} \quad (3.3)$$

The measurements obtained from coated bars did not provide good results because of the presence of sand, which modifies the value of densities and leads to poor void content data.

d) Water absorption

The Water/Moisture absorption was determined according to (ASTM D 570, 2010) - “Water Absorption of Plastics”. Three inch- long specimens were cut, dried, and weighed. Then, the specimens were immersed in water at 50°C. After 24 hours, the samples were removed from water and the surface was dried and weighed. Thereafter, the specimens were immersed again and periodically weighed until fully saturated. The samples were considered as saturated when the weight became constant and then the samples were dried at 100°C and weighed. The water content in weight percent, W , was calculated as follows:

$$W = 100 \cdot (P - P_d) / P_d \quad (3.4)$$

where P and P_d were the weights of the bar after 24 h immersion (or full saturation) and in the dry state, respectively.

e) Cure ratio

The cure ratio was determined according to (ASTM D 5028, 1990) – “Standard Test Method for Curing Properties of Pultrusion Resin by Thermal Analysis”. The enthalpy of polymerization of the sample was measured by DSC and compared with the enthalpy of polymerization of the pure resin, taking into account the weight percentage of resin in the matrix. Thirty to fifty milligrams of samples were accurately weighed and placed in an aluminum crucible. Then, the samples were heated from room temperature to 200°C at 20°C/min and the area of the peak of polymerization was calculated. Table 3.2 shows a summary of the curing ratios results.

f) Glass transition temperature

Glass transition temperature, T_g , was determined according to (ASTM D 3418, 2012) – “Transition Temperatures of Polymers by Thermal Analysis”. The T_g was assigned by Differential Scanning Calorimetry (DSC) using ASTM E 1131 test method. Thirty to forty milligrams of composite samples were weighed and placed in an aluminum pan. Then, the samples were heated to 200°C under nitrogen at a heating rate of 20°C/min. The value of T_g was taken at the mid-height of the C_p jump.

3.2.2 Tensile Properties

This section presents the tensile properties of sand-coated (GFRP) bars, as part of the specification of the product to be used as internal reinforcement for concrete structures. The modulus of elasticity for Grade I, Grade II, and Grade III bars were 40 GPa, 50 GPa, and 60 GPa, respectively. The tensile properties were determined according to the (ACI-440.6M, 2008) and (CSA-S807, 2010).

a) Test specimens

A total of 240 GFRP bars specimens Grade I and Grade II (No. 3, 4, 5, 6, and 8) were tested. The test specimens comprised three production lots and the tensile test was performed on eight specimens of each diameter. In addition, 144 specimens of GFRP bars Grade III (No. 3, 4, 5, 6, 8, and 10) were reported herein and performed on three different production lots with at least eight specimens each diameter. The anchorage steel tubes, shown in Figure 3.2, were installed at the ends of the bars and filled with grout. The specimens were tested under tensile test until failure, and the tensile properties [(modulus of elasticity (E_L), ultimate tensile strength (f_u), and ultimate strain (ϵ_u)] were determined.

b) Test method

The GFRP bars were tested in accordance with the (CSA-S806, 2002), Annex C – “Test Method for Tensile Properties of FRP Reinforcement.”, and (ACI-440.3R, 2004), Test Method B2 – “Test Method for Longitudinal Tensile Properties of FRP Bars”. The test specimens were instrumented with two linear-variable-displacement-transducers (LVDTs) to capture the specimen elongation during testing. The tests were carried out using the Baldwin testing machine. The load was increased until the rupture failure occurred. The applied load and bar elongation were recorded during the test using a computerized data acquisition system. Figure 3.2 shows the test setup and specimen during testing.



Figure 3.2: Test setup

3.2.3 Alkali Resistance without Load

This section presents the alkali resistance of GFRP bars (Grade I-40 GPa, Grade II-50 GPa, and Grade III-60 GPa) immersed in high pH solution (without load). A total of 240 specimens with three different lots productions and diameters were placed in alkali solution at 60°C environmental chamber for exposure time of 3 months. Thereafter, the test specimens were removed from the alkali solution and dried then test under tensile test.

a) Test specimens

240 GFRP bars with three different grades (I, II, and III) and diameters were conditioned for three months immersion period and tested to determine the alkaline resistance of those bars. Three sets of 96, 72, and 72 specimens of GFRP bars were tested. The first set was of Grades I with No. 4, No. 5, No. 6, and No. 8. The second set was of Grade II with No. 3, No. 4 and No. 5. The third set was of Grade III with No. 4, No. 6, and No. 8. The test specimens were received with the steel anchorage tubes to be ready for the tensile test. The steel anchorage tubes at the ends of the GFRP bars were protected from the solution using paint and plastic bags. The conditionings were conducted according to the (ACI-440.3R, 2004), Test method B.6, “Accelerated test method for alkali resistance of FRP bars” and (CSA-S806, 2012), Annex O, “Test method of alkali resistance of FRP rods”. The GFRP samples were inserted in a conditioning container filled with an alkaline solution according to (CSA-S807, 2010) while the specimens were placed

at 60°C environmental chamber for 3 months. The level of alkaline solution and pH level were checked periodically and a new solution was added, when necessary.

b) Test method

After three months of conditioning, the samples were removed from the container and tested to determine the tensile properties according to (CSA-S806, 2012), Annex O – “Test Method for Tensile Properties of FRP Reinforcement”, and ACI 440.3R (2004), Test Method B2 - “Test Method for Longitudinal Tensile Properties of FRP Bars” same proceeding as mention previously in section 3.2.2. Figure 3.3 and Figure 3.4 show the test specimen after drying and through the test. The typical mode of failure of the conditioned GFRP bars is shown in Figure 3.3 to Figure 3.10. Besides, the ultimate tensile strength (f_u) and modulus of elasticity (E_L) of GFRP bars are reported in Table 3.3.



Figure 3.3: GFRP bars of No.5 (Grade II) after drying

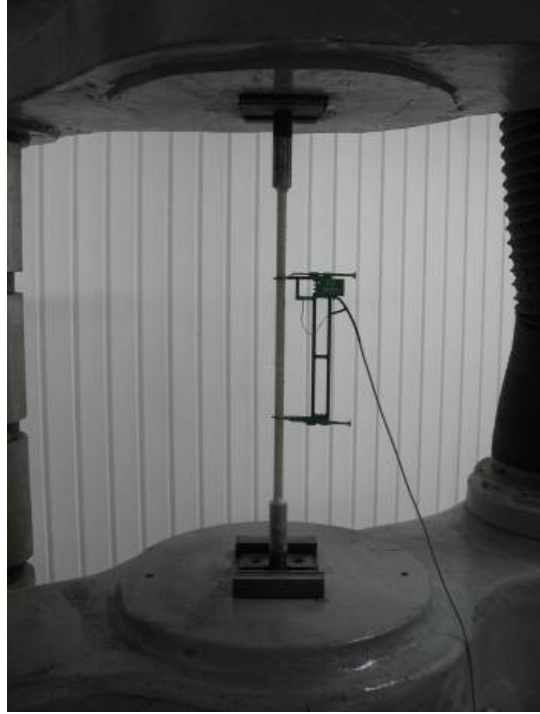


Figure 3.4: Test specimen at testing

3.2.4 Alkali Resistance with Load

This section presents the alkali resistance of GFRP bars (Grade II-50 GPa) diameters No. 4 and 5 set in high pH solution under load. The bars were made of continuous E-glass impregnated in a vinylester resin using the pultrusion process. A total of 23 specimens were placed under load in alkali solution at 60°C environmental chamber for exposure time of 3 months. The residual longitudinal tensile properties (f_u) and (E_L) of the GFRP conditioned specimens in high pH solution with load at elevated temperature, as required by (CSA-S807, 2010), were determined. The residual properties were compared to those of reference GFRP specimens (without conditioning).

a) Test specimens

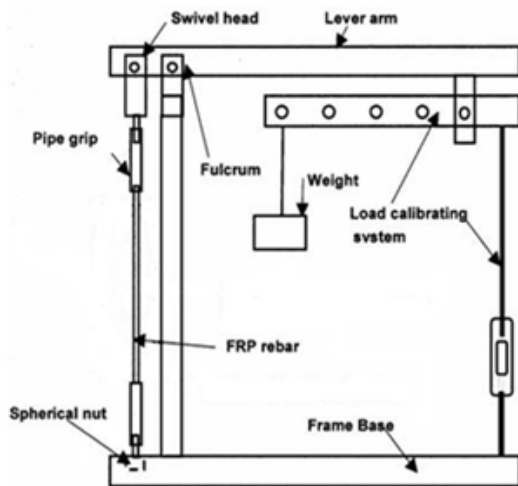
Eight and fifteen GFRP specimens of No.4 and No. 5 bars were tested, respectively. The GFRP bars (No. 5) comprised of three different lots productions (5 specimens each lot). The specimens were received ready for conditioning on creep frame and for testing. The research team installed anchorage tubes and PVC cylinders for the alkaline solution. Figure 3.5 represents the PVC cylinder installed directly on the bar and used for the conditioning in solution.



Figure 3.5: Test specimen with PVC cylinder filled with alkaline solution

b) Conditioning

The conditioning was conducted according to the (CSA-S806, 2002), Annex O-test method (*Test Method for Alkali Resistance of FRP Rods*) for conditioning in alkaline solution. In addition, the conditionings in solution were performed under a sustained tensile load equal to 3,000 micro-strains (according to the (CSA-S807, 2010)) using a load creep frame of durability, Figure 3.6. The 23 GFRP bars were installed in sustained load frame placed in the environmental chamber of Durability Laboratory. The ends of the tubes were sealed with silicone and duct paste and the cylinders got filled with an alkaline solution containing 118.5 g $\text{Ca}(\text{OH})_2$, 4.2 g KOH and 0.9 g NaOH per liter (pH=13). The temperature in the environmental chamber was then set at 60°C for three months (as required by (CSA-S807, 2010) standard). The level of alkaline solution was checked periodically and new solution was added, when necessary.



(a)



(b)

Figure 3.6: Sustained creep load frame: (a) Schematic view, and (b) Specimen under sustained load

c) Test method

At the end of the conditioning, the 23 GFRP bars were tested in accordance with the (CSA-S806, 2002), Annex C – “Test Method for Tensile Properties of FRP Reinforcement”, and ACI 440.3R (2004), Test Method B2 - “Test Method for Longitudinal Tensile Properties of FRP Bars” as previously mentioned in section 3.2.2. The PVC cylinders were removed from the bars before the tests to prevent any damage to the bar (Figure 3.7). The test specimens were then instrumented with an extensometer to capture the specimen elongation during testing (Figure 3.8). The tests were carried out using the Baldwin testing machine (Figure 3.9). The load was increased until tensile failure occurred. The applied load and bar elongation were electronically recorded during the test using a computerized data acquisition system. Through this test, the (f_u) and (E_L) were determined. Typical mode of failure of the bars is shown in Figure 3.17.



Figure 3.7: Removing the PVC tubes



Figure 3.8: LVDT instrumentation



Figure 3.9: Test specimens at testing

3.3 Results

3.3.1 Physical Properties

Physical test results of GFRP bars on the Grade I, II, and III showed that the tested GFRP bars satisfied the ACI and CSA requirements. The test results indicated that the fiber contents in weight of GFRP bars Grade I was 80.8%, 81.4%, and 81.8%, respectively, for No. 5, No. 6, and No. 8. The corresponding values for FGRP bars Grad III were 81.4%, 81.6%, and 82.4%. In addition, the test results indicated that the fiber contents in weight of GFRP bars Grade II were 81.4% and 82.7% for No. 5 and No. 6, respectively. The mass percentages of water uptake after 24 hours were found to be in range 0.21% to 0.34% on average for the three grades. These values within the limits specified in CSA S807 (0.35%). Also, the mass percentage of water uptake at saturation for No. 5, No. 6, and No. 8, Grade I were found to be 0.37%, 0.44% and 0.3%, respectively. The corresponding values for bars Grade III were 0.45%, 0.3%, and 0.29%. The mass percentages of water uptake at saturation for GFRP bars No. 5 and No. 6 Grade II were 0.6% and 0.49%. The water-absorption values obtained are within the limits specified in CSA S807 (0.75%). The material's cure ratio for all the tested GFRP bars is high (close to 100%). The glass transition temperature was visible from the thermograms obtained by Differential Scanning Calorimetry (DSC), and in the range of 125 to 131 °C for the three grades of GFRP bars.

Table 3.1: Properties, test method, number of specimens and specified limits

Property	Method	Number of specimens	Specified limit
Fiber content	ASTM D 3171 – “Constituent content of composite”, Method I; Procedure G.	3	70% (by weight)
Transverse coefficient of thermal expansion	ASTM E 831 – “Linear Thermal Expansion of Solids Materials by Thermo-mechanical Analysis (TMA)”.	3	40·10 ⁻⁶ °C ⁻¹
Void Content	ASTM D 2734 – “Void Content of Reinforced Plastics”	3	1%
Water absorption	ASTM D 570 – “Water Absorption of Plastics”	5	1.00 % (D2) 0.75 % (D1)
Cure ratio	ASTM D 5028 – “Curing Properties of Pultrusion Resin by Thermal Analysis”	5	93 % (D2) 95 % (D1)
Glass transition temperature	ASTM D 3418 – “Transition Temperatures of Polymers by Thermal Analysis”	5	80°C (D2) 100°C (D1)

Table 3.2: Physical properties for GFRP bars

Bar type & Property	Grad I			Grad II		Grad III		
	No.5	No.6	No.8	No.5	No.6	No.5	No.6	No.8
Fiber content (%)	80.8 ± 0.08	81.4 ± 1.50	81.8 ± 0.04	81.4 ± 0.10	82.7 ± 0.09	81.4 ± 0.05	81.6 ± 0.06	82.4 ± 0.14
Transverse CET (10 ⁻⁶ °C ⁻¹)	21.9 ± 0.75	21.5 ± 1.51	21.4 ± 0.72	21.2 ± 1.31	18.7 ± 0.73	22.8 ± 1.03	22.2 ± 0.95	22.8 ± 1.70
Void content (%)	-	-	-	-	-	-	-	-
Water absorption @ 24 h (%)	0.32 ± 0.03	0.25 ± 0.02	0.21 ± 0.02	0.34 ± 0.02	0.28 ± 0.03	0.34 ± 0.03	0.27 ± 0.04	0.21 ± 0.02
Water absorption at saturation (%)	0.37 ± 0.03	0.44 ± 0.03	0.30 ± 0.02	0.60 ± 0.06	0.49 ± 0.06	0.45 ± 0.04	0.37 ± 0.06	0.29 ± 0.02
Cure ratio (%)	99.9 ± 0.16	100 ± 0.00	100 ± 0.00	100 ± 0.00	100 ± 0.00	100 ± 0.00	100 ± 0.00	100 ± 0.00
T _g (°C)	127.4 ± 2.86	125.9 ± 1.60	128.6 ± 1.61	125.5 ± 2.42	127.6 ± 0.97	129.5 ± 2.32	128.0 ± 1.96	131.3 ± 2.96

3.3.2 Tensile Properties

a) Calculations

The tensile strength, f_u , of GFRP bars was calculated according to the following equation:

$$f_u = \frac{F_u}{A} \quad (3.5)$$

where:

f_u = Tensile strength (MPa);

F_u = Tensile capacity (N); and

A = Cross-sectional area of the test bar (mm^2).

The tensile modulus of elasticity, E_L , was calculated from the difference between the load (stress)-strain curve values at 25 and 50% of the tensile capacity according to the following equation:

$$E_L = \frac{F_1 - F_2}{(\varepsilon_1 - \varepsilon_2)A} \quad (3.6)$$

where:

E_L = Longitudinal modulus of elasticity (MPa);

A = Cross-sectional area of the test bar (mm^2);

F_1 and ε_1 = Load and corresponding strain, respectively, at approximately 50% of the ultimate tensile capacity; and

F_2 and ε_2 = Load and corresponding strain, respectively, at approximately 25% of the ultimate tensile capacity, (N and dimensionless, respectively).

b) Test results and discussion

The test results of the tensile properties of the GFRP bars Grade I, Grade II, and Grade III were reported in Table 3.3. The nominal cross-sectional areas provided by the (CSA-S807, 2010) were considered in the calculation of the tensile strength and the elastic modules for all the tested bars. The mode of failure is shown in Figure 3.10. It can be noted that the tested bars failed in the middle of the bar (gauge length). It should be mentioned that the LVDTs were removed at about 80% of the ultimate capacity of the specimens not to break the LVDTs.

The test results of the tensile properties of the GFRP bars Grade I, Grade II, and Grade III were reported in Table 3.3. The nominal cross-sectional areas provided by the (CSA-S807, 2010) were considered in the calculation of the tensile strength and the elastic modules for all the tested bars. The mode of failure is shown in Figure 3.10. It can be noted that the tested bars failed in the middle of the bar (gauge length). It should be mentioned that the LVDTs were removed at about 80% of the ultimate capacity of the specimens not to break the LVDTs. The test results indicated that the average tensile modulus of elasticity values for GFRP bars No. 3, 4, 5, 6, and 8 (Grade I) are 41.2, 44.0, 45.0, 45.0, and 44.0 GPa, respectively. The corresponding ultimate tensile strengths were 907, 866, 999, 868, and 817 MPa, respectively. Also, the corresponding ultimate

strain values were 2.1, 2.3, 2.2, 2.0, and 1.7%. The tensile modulus of elasticity values were over the minimum limit of the ACI 440.6M and CSA-S807-10, 39.3 and 40 GPa, respectively. On the other hand, the ultimate tensile strengths and strains meet the requirements of the ACI 440.6M and CSA-S807-10.

Table 3.3 shows the test results of the tensile modulus of elasticity, ultimate tensile strengths and strains of GFRP bars Grad II. The results indicated that the tensile modulus for the bars of different sizes ranged from 55 to 58 GPa, which it is over the ACI 440.6M and CSA-S807-10 limit (39.3 and 50 GPa, respectively). Also, the ultimate tensile strengths ranged from 1088 to 1129 MPa, which meets the requirements of the ACI 440.6M and CSA-S807-10. GFRP bars of Grade III reached to higher load level at failure. Since, the ultimate tensile strength ranged from 1118 to 1537 MPa. The corresponding tensile modulus of elasticity and ultimate tensile strain were varied from 67 to 66 GPa, and 1.7 to 2.3%, respectively. All these values meet the requirements of the ACI 440.6M and CSA-S807-10.



(a) GFRP before testing



(b) After testing

Figure 3.10: Typical mode of failure of the GFRP bars

3.3.3 Alkali Resistance without Load

a) Calculations

The f_u and E_L of GFRP bars were calculated by Eqns 3.5 and 3.6. The tensile property retention (strength and elastic modulus) was calculated according to the following equations:

$$R_{et} = (f_{u2}/f_{u1}) \times 100\% \quad (3.7)$$

$$E_{et} = (E_2/E_1) \times 100\% \quad (3.8)$$

Where,

R_{et} = Tensile capacity retention, %;

f_{u1} = Average tensile capacity of non-conditioned specimens (reference specimens), kN;

f_{u2} = Average tensile capacity of conditioned specimens, kN;

E_{et} = Elastic modulus retention, %;

E_1 = Average elastic modulus of non-conditioned specimens, GPa, and

E_2 = Average elastic modulus of conditioned specimens, GPa.

b) Test results and discussion

The test results of the tensile properties of the GFRP bars Grade I, Grade II, and Grade III after 3 months conditioning and the retention of the tensile properties were reported in Table 3.4. The nominal cross-sectional areas provided by the (CSA-S807, 2010) were considered in the calculation of the tensile strength and the elastic modulus for all the tested bars. Test observations indicated that there was no significant change noticed at the surface of GFRP bars due to aging high pH solution at 60°C. The mode of failure is shown in Figure 3.11 to Figure 3.16. It can be noted that the tested bars failed in the middle of the bar (gauge length). The failure was accompanied by the delamination of fibers and resin.

The results for reference samples and conditioned specimens were compared in order to measure the effect of potentially high aging on short-term mechanical properties of GFRP bars. It can be noticed that the GFRP bars tested in this study show higher residual strengths compared to required limits of the Canadian codes (S807-10 and S806-12) and ACI 440. The average tensile strength retention of the tested GFRP bars (Grade I, II, and III) conditioned during three months in high pH solution without load at 60°C is over 90% for all the tested GFRP bars of (No.4, 5, 6, and 8), (No. 3, 4, and 5), and (No. 4, 6, and 8), respectively. All the tested GFRP bar (Grade I, II, and III) sizes presented a value greater than the specified limit for high durability (D1) in (the CSA-S807, 2010) Standard (80%). Table 3.4 presents the change in the elastic modulus of aged bars with time of immersion at 60°C. Indeed, it can be seen from the measured results that after 90 days, the loss of elastic modulus is negligible and all aged bars are not affected by the higher temperature or alkaline solution. This result shows that elastic modulus of bars is not affected by aging in an alkali environment. The value of the modulus of elasticity retention for Grade I, II, and III varied between 89 to 103%, 95% to 107%, and 92% to 103%, of the reference elastic modulus value, respectively. The corresponding ultimate tensile strength retentions were varied between 90 to 100%, 93 to 100%, and 87 to 96%, respectively.



(a) Before testing



(b) After testing

Figure 3.11: Mode of failure of GFRP bars No.4 (Grade I)



(a) Before testing

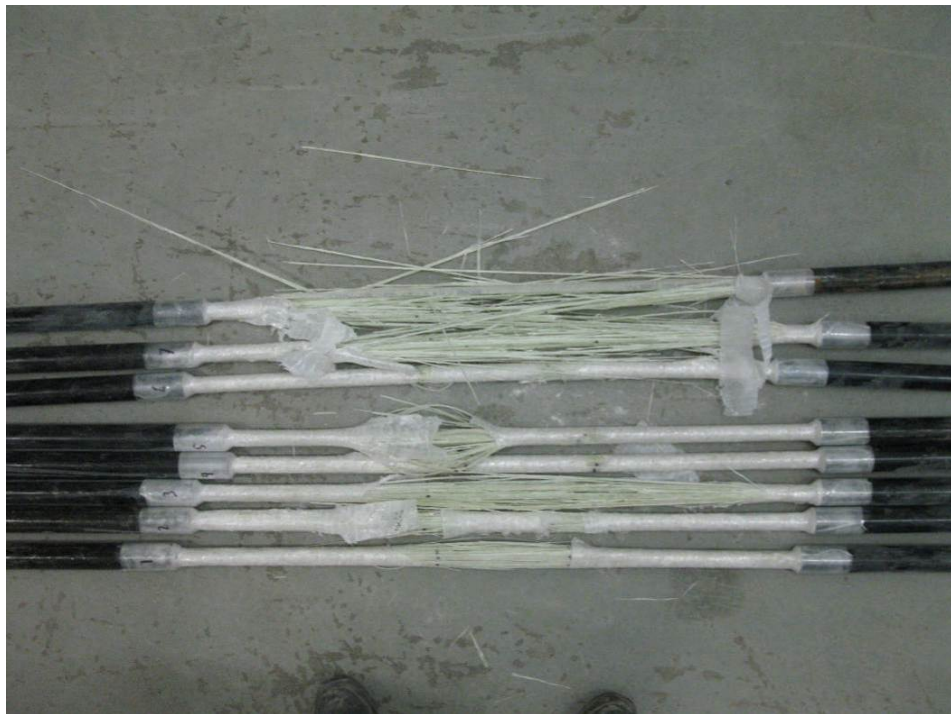


(b) After testing

Figure 3.12: Mode of failure of GFRP bars No.5 (Grade I)



(a) Before testing



(b) After testing

Figure 3.13: Mode of failure of GFRP bars No. 8 (Grade I)



Figure 3.14: Mode of failure of GFRP bars No. 4 (Grade II)

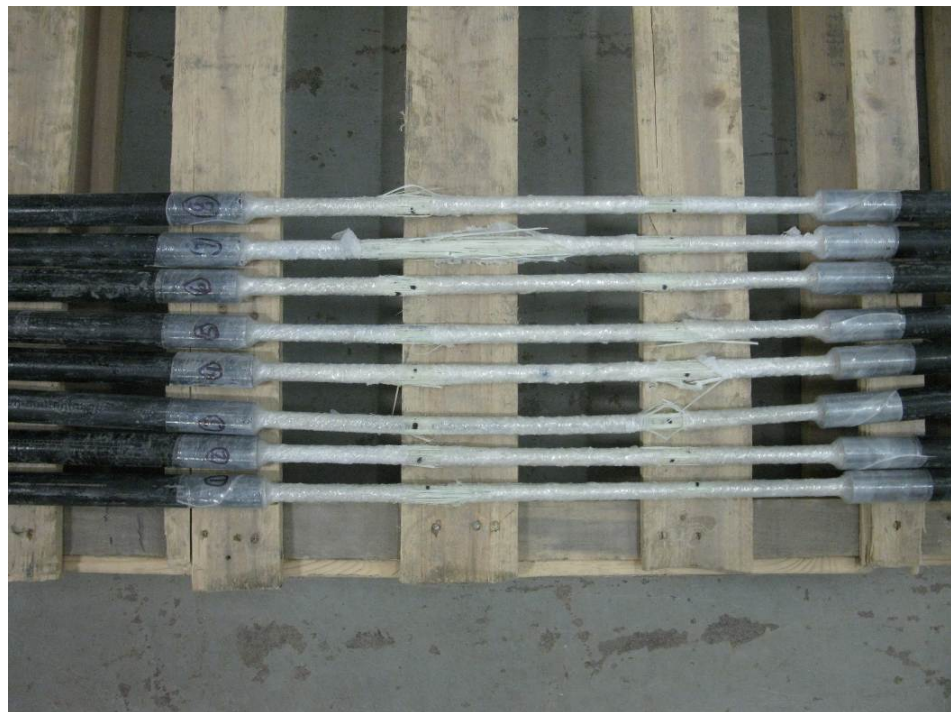


Figure 3.15: Mode of failure of GFRP bars No. 5 (Grade II)



(a) Before testing



(b) After testing

Figure 3.16: Mode of failure of GFRP bars No. 6 (Grade III)

Table 3.3: Tensile properties of GFRP reinforcing bars (reference bars)

Bar Size	d_b (mm)	Area (mm ²)	Modulus of Elastic, E_L (GPa)			Ultimate Tensile Strength, f_u (MPa)			Ultimate Tensile Elongation, ϵ_u (%)		
			I	II	III	I	II	III	I	II	III
No. 3	9.53	71	41.2±0.6	56.0±1.1	81.0±3.5	907±35.0	1129±20.9	1194±128.0	2.2±0.10	2.0±0.10	2.1±0.23
No. 4	12.7	129	44.0±0.8	57.0±0.3	66.0±1.1	866±25.5	1120±28.1	1537±37.0	2.0±0.10	2.0±0.10	2.3±0.10
No. 5	15.9	199	45.0±1.4	55.0±1.1	63.0±0.8	999±55.0	1117±26.2	1421±43.5	2.2 ±0.2	2.0±0.03	2.2±0.10
No. 6	19.1	285	45.0±1.1	57.0±0.7	65.0±1.1	868±20.1	1088±53.6	1343±45.8	2.0±0.10	1.9±0.03	2.0±0.10
No. 8	25.4	510	44.0±0.6	58.0±0.3	67.0±0.7	817±27.4	1125±24.4	1118±38.4	1.8±0.10	1.9±0.07	1.7±0.10
No. 10	31.8	791	-	-	60.1±1.2	-	-	1308±55.2	-	-	2.2±0.10
ACI 440.6M limit			≥ 39.3 GPa			≥ 655 MPa			≥ 1.20 %		
CSA-S807-10 limit			≥ 40 GPa	≥ 50 GPa	≥ 60 GPa	(No.3) ≥ 750 MPa; (No.4, 5) ≥ 650 MPa; (No.6) ≥ 600 MPa; (No.8) ≥ 550 MPa; (No.10) ≥ 500 MPa			≥ 1.20 %		

Table 3.4: Tensile properties of conditioned GFRP reinforcing bars (after 3 months of immersion)

Bar Size	d_b (mm)	Area (mm ²)	Elastic tensile modulus, E_f (GPa)			Ultimate Tensile Strength, f_{fu} (MPa)			Tensile capacity retention (%)			E_f , retention (%)		
			I	II	III	I	II	III	I	II	III	I	II	III
No. 3	9.53	71	-	56.0±1.2	-	-	1128±20.0	-	-	100	-	-	100	-
No. 4	12.7	129	44.9±0.7	60.8±0.3	64.0±2.1	825±18.2	1047±30.7	1308±36.2	95	93	87	102	106	93
No. 5	15.9	199	39.6±0.3	53.0±1.0	-	871±15.5	1055±19.7	-	90	95	-	89	95	-
No. 6	19.1	285	45.0±0.6	-	65.0±0.4	823±16.6	-	1289±24.1	95	-	96	101	-	100
No. 8	25.4	510	44.0±0.5	-	68.3±0.7	824±24.3	-	1272±35.3	100	-	91	99	-	99

3.3.4 Alkali Resistance with Load

a) Test results and discussion

The tensile strengths, f_u , of conditioned GFRP bars (Grade II-50 GPa) diameters No. 4 and 5, were calculated in accordance with Eqs 3.5 and 3.6. The nominal cross-sectional area as indicated in (CSA-S807, 2010) (129 and 199 mm² for No. 4 and No. 5, respectively) was considered in the calculation of the tensile strength and the elastic modulus of the bars. Table 3.5 presents the tensile properties of the conditioned GFRP specimen bars No. 4 and No. 5 and the retention of the tensile capacity compared to their reference counterparts. There was no significant change noticed at the surface of GFRP bars due to aging high pH (pH = 13) solution under sustained load. Figure 3.17 shows that all the bars failed in the middle of the bars, as expected. The failure was accompanied by the delamination of fibers and resin. The tensile test of preloaded specimens showed almost a linear behavior up to failure.

As shown in Table 3.5, the average residual tensile strengths for preloaded GFRP bar No. 4 and 5 were equal to 889 and 1116 MPa, respectively. The corresponding average residual tensile modulus of elasticity was 50.7 and 52.8 GPa, respectively. Indeed, it was seen from the measured results that after the preloading, no significant losses of tensile strength and elastic modulus occurred and the preloaded bars were not affected by stress level used for their preloading. The tested GFRP bars showed a value of tensile capacity retention over 80%. The tested GFRP bars meet the D1 requirement of (CSA-S807, 2010) for the alkali resistance in high pH solution with load (required limit is 70%). Whereas, the average value of the modulus of elasticity retention for GFRP bars No. 4 and 5 were 90 and 95%, respectively. The corresponding tensile capacity retention was 80 and 100%, respectively. These results showed that tensile properties of GFRP bars just after the application of high stress level, leading to the creation of cracks and micro cracks, were not affected by their preloading under a sustained tensile load equal to 3,000 micro-strains.

Table 3.5: Tensile properties of conditioned GFRP bars (Grade II-50 GPa) after 3 months of immersion

Bar Size	Load condition	Elastic tensile modulus, E_f (GPa)	Ultimate Tensile Strength, f_{fu} (MPa)	Tensile capacity retention (%)	E_f , retention (%)
No. 4	With Load	50.7±1.5	889±67.2	80	90
	Without Load	60.8±0.3	1047±30.7	93	106
No. 5	With Load	52.8±1.5	1116±24.1	100	95
	Without Load	53.0±1.0	1055±19.7	95	95



Figure 3.17: Typical mode of failure of the tested GFRP bars

3.4 Summary

a) Physical Properties

Physical properties test results of GFRP bars with the three Grades [I(LM) , II(STD), and III(HM)] showed that the test GFRP bars satisfied the ACI and CSA requirements (when applicable) for: (1) Glass fiber content, (2) Transverse coefficient of thermal expansion, (3) Moisture absorption, (4) Cure ratio, and (5) Glass transition temperature.

b) Tensile properties

Mechanical properties were given for three different grades of GFRP bars named: Grade I, Grade II, and Grade III. Different sizes of GFRP bars were tested for each grade. Tensile tests were conducted to define the mechanical properties of GFRP bars. Tensile strengths, elastic modulus, and ultimate strain are presented for each bar type. The test results indicated that all the tested GFRP bars with different sizes and grades meet the ACI and CSA requirements.

c) Alkali resistant without load under conditioning of 3 months

The average tensile strength retention of the tested GFRP bars (Grade I, II, and III) conditioned during three months in high pH solution without load at 60°C is over 90% for all the tested GFRP bars of (No.4, 5, 6, and 8), (No. 3, 4, and 5), and (No. 4, 6, and 8), respectively. All the tested GFRP bar (Grade I, II, and III) sizes presented a value greater than the specified limit for high durability (D1) in (the CSA-S807, 2010) Standard (80%). The modulus of elasticity of the GFRP bars (Grade I, II, and III) is not significantly affected by the immersion in high pH. The value of the modulus of elasticity retention varied between (89% to 103 %), (95% to 107%), and (92% to 103%), of the reference elastic modulus value, respectively.

d) Alkali resistant with load under conditioning of 3 months

The tested GFRP bars (Grade II-50 GPa) showed a value of tensile capacity retention over 80%. The tested GFRP bars meet the D1 requirement of (CSA-S807, 2010) for the alkali resistance in high pH solution with load (required limit is 70%).

CHAPTER 4

CHARACTERIZATION AND DURABILITY OF TOKYO ROPE CARBON FIBER COMPOSITE CABLES (CFCC)

4.1 Introduction

In recent years, fiber-reinforced-polymer (FRP) materials have emerged as an acceptable construction material for both new construction as well as for the rehabilitation and strengthening of existing structures. The proven track record of superior performance of FRP materials in terms of strength, durability, resistance to corrosion, and versatility of fabrication have made these materials attractive to civil engineers. One use of FRP materials is as reinforcement for reinforced-concrete structures, such as bridges, buildings, highways, pavements, and marine structures. Due to the excellent corrosion resistance of the FRP reinforcing bars and tendons currently on the market, they have been used as reinforcement for many concrete structures, including bridges, parking garages, highway pavements, tunnels, and structures housing MRI equipment. Nowadays, FRP tendons for prestressing applications are emerging worldwide as one of the most promising technologies of this decade. FRP tendons can be manufactured in the form of round or flat bars, ropes, cables, or strands. A variety of surface textures are possible, including smooth, braided, rough, sand coated, dimpled, indented, etc. Commercially available FRP tendons are in the form of Aramid FRP (AFRP), Carbon FRP (CFRP) and Glass FRP (GFRP). This testing program aims to characterize and evaluate the physical, mechanical, and durability characteristics of the Tokyo Rope CFCC 1X7 as shown in Figure 4.1. Carbon-fiber composite cables (CFCCs) manufactured by Tokyo Rope Manufacturing Co. Ltd., Japan, were used in this study. This type of reinforcement is of interest to the Florida Department of Transportation and other DOTs for use as corrosion-resistant reinforcing material for prestressed precast-concrete bridge-deck and pile applications (as a competitive material of stainless-steel prestressing cables). The Tokyo Rope carbon-fiber composite cable is a stranded cable made from a number of individual wires. In general, these cables are made of 7, 19, or 37 twisted carbon wires, with nominal diameters varying from 5 to 40 mm (0.2 to 1.6 in.). The individual wires are made with carbon fibers impregnated with thermosetting resin. In this study, CFCC cables 7.5 mm and 12.5 mm in diameter made with a 7-wire strand are used, as shown in Figure 4.1.



Figure 4.1: Carbon Tokyo Rope (CFCC 1X7)

4.2 Procedures

4.2.1 Physical Properties

This section presents preliminary physical characteristics of Tokyo Rope CFCC 1X7 ($\phi 12.5$ mm) as shown in Figure 4.1. The test procedures and experimental test results of carbon fiber content, water absorption, cure ratio and glass transition temperature were carried out according to the specification of the (ACI-440.6M, 2008) and (CSA-S807, 2010). Moreover, Optical and Scanning Electronic Microscopy (SEM) analyses were performed to investigate the microstructure of the material.

a) Fiber content

Carbon fiber content was determined by thermogravimetry according to (ASTM E 1131, 2008) – “Standard Test Method for Compositional Analysis by Thermogravimetry”. A very small piece of material (a few tenths milligrams) was cut from the center of the cable, placed in a platinum crucible, and then heated up to 550°C under inert atmosphere. The weight loss was recorded as a function of temperature while the weight loss at 550°C (W_L) was measured. Since the material only contains carbon fibers and resin, fiber content by weight was calculated according to the following equation:

$$\text{Fiber Content by weight} = 100 \cdot (W_T - W_L) / W_T \quad (4.1)$$

b) Water absorption

The water absorption was determined according to the (ASTM D 570, 2010) – “Water Absorption of Plastics”. The water content percentage in weight was obtained from three 1-inch long specimens, which were cut, dried, and weighed. After 24 hours, the specimens were immersed in water at 50°C . Then, the specimens were removed from water until the surface was dried and weighed again. Thereafter, the specimens were immersed again and periodically removed from water until the surface was dried and weighed until full saturation. The specimens were considered as saturated when the weight became constant. The water content percentage in weight, W , was calculated from Eqn. 3.4.

c) Cure ratio

The cure ratio was determined according to (ASTM D 5028, 1990) – “Curing Properties of Pultrusion Resin by Thermal Analysis”. The measurements were carried out on three specimens. The enthalpy of polymerization of the samples were measured by Differential Scanning Calorimetry (DSC). Thereafter, the measurements were compared to the enthalpy of polymerization of pure resin, taking into accounts the weight percentage of resin in the matrix. Thirty to fifty milligrams of sample were accurately weighed and placed in aluminum crucible. The samples were heated from the room temperature to 200°C at 20°C/min and the peak area of polymerization was calculated.

d) Glass transition temperature

The Glass transition temperature, T_g , was determined by (DSC) using (ASTM D 3418, 2012) – “Transition Temperatures of Polymers by Thermal Analysis”. Three specimens were used to measure the T_g . Thirty to forty milligrams of composite sample were weighed and placed in an aluminum pan. Then, the sample was heated up to 200°C under nitrogen at a heating rate of 20°C/min. The value of T_g was taken at the mid-height of the C_p jump.

e) Optical microscopy

Micrographs of the sample cross-section were prepared according to the procedure described in Section 4.3. The sample cross-section micrographs were obtained using a Leica Microscope at several magnifications.

f) Scanning Electronic Microscopy (SEM)

Longitudinal and transverse sections of unconditioned cable sample (one-inch-long) were cut and placed in cylindrical molds, where epoxy resin was cast. After 24 hours of curing at room temperature, the samples were removed and cut using a low speed saw equipped with a diamond blade. Thereafter, the specimens were polished using a polishing machine with three diamond pastes (15, 3, and 1 micron) before sputtering them with palladium. Thus, the specimens were ready for analysis with a Hitachi SEM.

4.2.2 Tensile Properties of Long-Term Durability Performance of Tokyo

Rope (CFCC)

The test program aims to characterize and evaluate the long-term durability characteristics of the Tokyo Rope CFCC 7.5 mm-diameter (area of 31.1 mm²). The effect of alkali conditioning on Tokyo Rope CFCC bars at different elevated temperatures (22, 40, 50, 60 °C) for different exposure times 1,000, 3,000, 5,000, and 7,000 hours was addressed. The tests investigated the alkali resistance of the Tokyo Rope cables using the (ACI-440.3R, 2004), test method B.6, “Accelerated test method for alkali resistance of FRP bars” and the (CSA-S806, 2012), Annex M, “Test method of alkali resistance of FRP rods”. After conditioning, the Tokyo Rope samples were tested in tension according to B.2 test method of the (ACI-440.3R, 2004) and (CSA-S806, 2012), Annex C– “Test Method for Tensile Properties of FRP Reinforcement” to determine the tensile properties.

The CFCC cable having 7.5 mm in diameter (area of 31.1 mm²) with a 7-rod strand was received as one coil. All bars prepared for tensile preloading were cut into lengths of 1600 mm as specified in ASTM D7205 [2]. The CFCC specimens were completely immersed in alkaline solution inside different PVC tubes. The alkaline solution was prepared using calcium hydroxide, potassium hydroxide, and sodium hydroxide (118.5 g of Ca(OH)₂ + 0.9 g of NaOH + 4.2 g of KOH in 1 L of deionized water) according to CSA S806 [3] and ACI 440 [4]. The alkali solution's pH was 12.8. The tubes and CFCC specimens were kept at four different temperatures (22°C, 40°C, 50°C, and 60°C).

a) Test specimens

Tokyo Rope cable was received as one coil (Figure 4.2). A total of 52 specimens were cut into a length of 1600 mm (Figure 4.3), as specified in ASTM D7205.]. The bars were divided into two series: (1) the unconditioned control samples and (2) the conditioned samples immersed in alkaline solution. The test specimens were conditioned and then tested under tensile test method. The CFCC specimens were completely immersed in alkaline solution inside different PVC tubes. The alkaline solution was prepared using calcium hydroxide, potassium hydroxide, and sodium hydroxide (118.5 g of Ca(OH)₂ + 0.9 g of NaOH + 4.2 g of KOH in 1 L of deionized water) according to CSA S806-12 and ACI 440, Figure 4.4. Sixteen ABS tubes were used to keep the test specimens in the solution, which has 40 mm diameter, 1700 mm length and plugged at their ends by plastic caps, Figure 4.5.]. The alkali solution's pH was 12.8. Each tube had 3 bars. The tubes were filled with the alkaline solution from the upper end in vertical position (Figure 4.6). After that, the tubes were sealed at top end by plastic caps.



Figure 4.2: CFCC Tokyo Rope cable as received



Figure 4.3: Test specimens (length 1600 mm)



(a) Alkali powder bag

(b) Alkaline solution after mixed with water

Figure 4.4: Preparing the alkaline solution



(a) ABS plastic caps



(b) ABS tubes

Figure 4.5: The ABS tubes and plastic caps



Figure 4.6: Filling the tubes with alkaline solution

b) Conditioning

The tubes and CFCC specimens were kept at four different temperatures (22°C, 40°C, 50°C, and 60°C). In addition, 22°C was performed by immersing the CFCC bars in a wooden containers filled with alkaline solution at room temperature. The wood containers were covered with polyethylene sheeting to avoid water evaporation during conditioning. Three environmental chambers were used to accelerate the degradation of the CFCC specimens at 40°C, 50°C, and 60°C. The immersion temperatures were chosen to accelerate the degradation effect of aging, but they were not high enough to produce any thermal-degradation mechanisms. The specimens were removed from the alkaline solution after 1000, 3000, 5000, and 7000 hours. Figure 4.7 and Figure 4.8 show different environmental chambers (40, 50, and 60 °C) and the tubes inside the environmental room. Table 4.1 presents the test matrix of the intended experimental program.

Table 4.1: Test matrix for the specimens

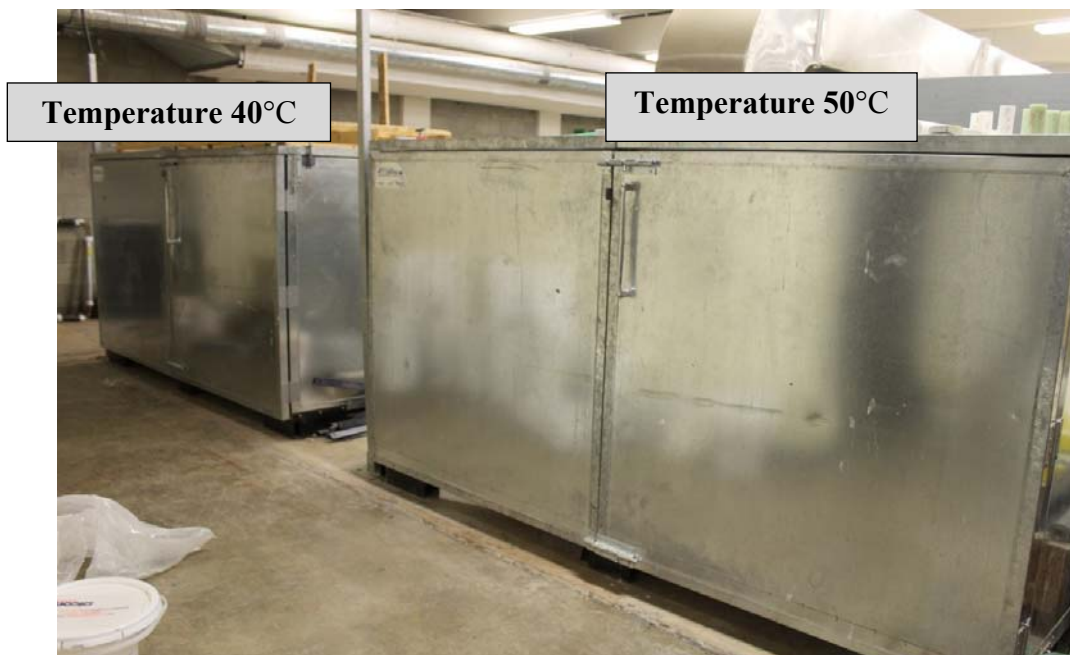
Phase no.	No. of specimens	Immersion duration (h)	Temperature (°C)
I	3	1,000	22
	3	1,000	40
	3	1,000	50
	3	1,000	60
II	3	3,000	22
	3	3,000	40
	3	3,000	50
	3	3,000	60
III	3	5,000	22
	3	5,000	40
	3	5,000	50
	3	5,000	60
IV	3	7,000	22
	3	7,000	40
	3	7,000	50
	3	7,000	60

c) Testing method

All of the control and conditioned CFCC specimens were tested under tension according to ASTM D7205. The CFCC specimen length as well as the length and diameter of the anchor to be used for the tensile test were calculated according to ASTM D7205. The test specimens were instrumented with one linear variable differential transformer (LVDT, 200 mm length) to capture specimen elongation during testing. The tests were carried out with the Baldwin testing machine in the structures laboratory of the Department of Civil Engineering at the University of Sherbrooke. The rate of loading ranged from 250 to 500 MPa/min. The applied load and bar elongation were recorded during the test with a computer data-acquisition system. Figure 4.9 and Figure 4.11 showed the test specimens. The ultimate tensile strength (f_u) and modulus of elasticity (E_L) of Tokyo bars under different exposure time and environmental temperature chambers were determined.



(a) Environmental chamber of (60°C)



(b) Environmental chamber of (temperature 40°C and 50°C)

Figure 4.7: Different environmental chambers



(a) Specimens inside the environmental chamber of 60 °C



(b) Specimens inside the environmental chamber of 50 °C

Figure 4.8: Specimens inside environmental chambers

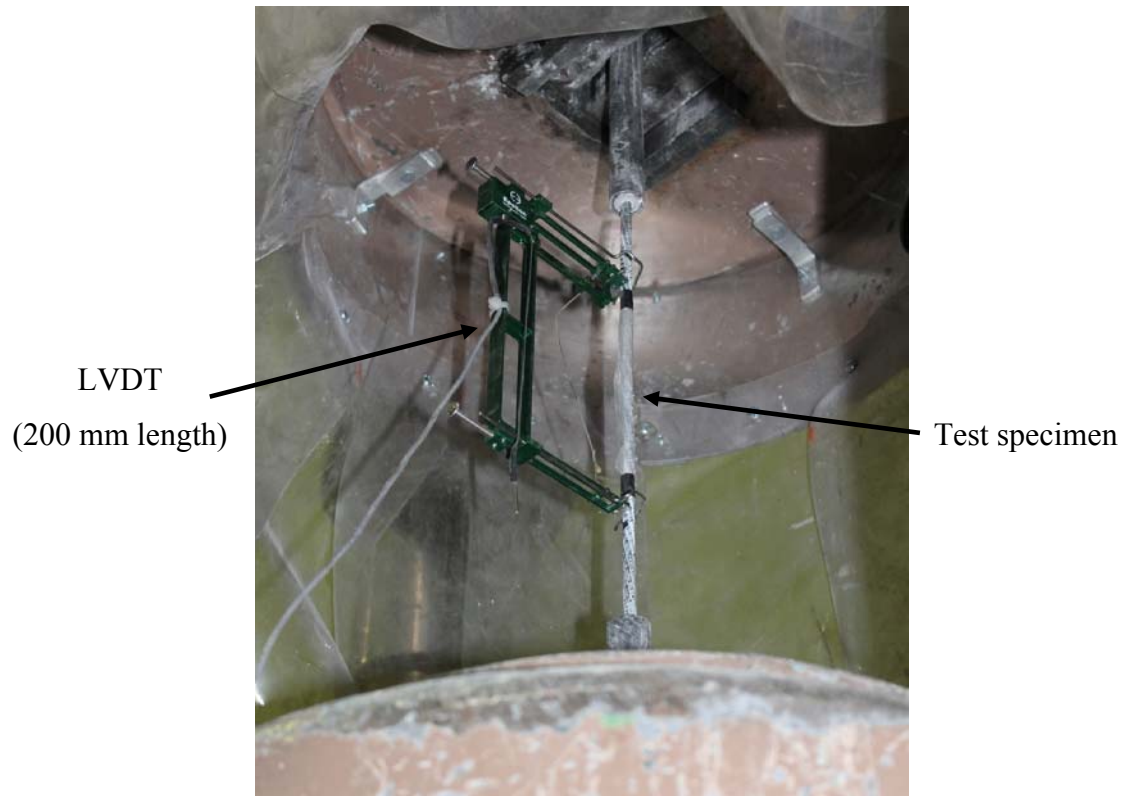


Figure 4.9: Test specimen under testing

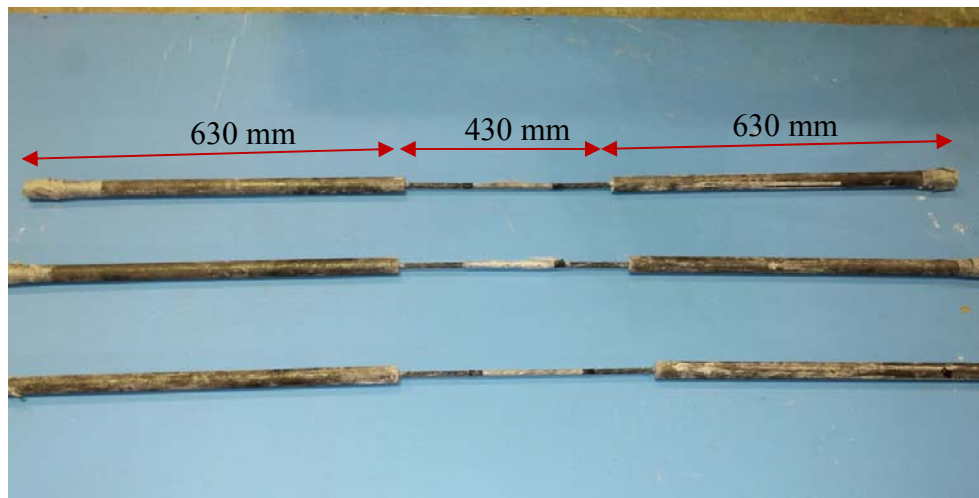


Figure 4.10: Specimens dimensions



Figure 4.11: Test specimens attached with steel tubes

4.3 Results

4.3.1 Physical Properties

The test results indicated that the carbon-fiber content in weight of CFCC was 82.0%. The mass percentage of water uptake after 24 hours and at saturation were found to be 0.8% and 1.1% on average. The mass percentage of water uptake after 24 hours and at saturation was reported in

. The water-absorption values obtained exceed the limits specified in CSA S807 (0.35%). The material's cure ratio is high (close to 100%). Figure 4.12 presents the three thermograms obtained by Differential Scanning Calorimetry. The glass transition temperature was not clearly visible from the thermograms obtained by Differential Scanning Calorimetry (DSC), two very small shifts were observed at 110° and 150°C, approximately. However, it was not possible to confirm that these shifts are caused by the glass transition of one, two or a blend of resins.

Figure 4.13 to Figure 4.17 presented the optical micrographs of the cross-section of a cable sample. Figure 4.13 showed a general view of the seven strands. Each strand was covered with a layer of carbon fibers. The intersection between the central and outer strands creates six voids along the cable. Some pores were visible at various degrees. Strands 1, 4 and 5 were porous (Figure 4.14), whereas strands 2 and 7 are less porous (Figure 4.15). Strands 3 and 6 did not present any significant porosity (Figure 4.16). Voids generally were occurred at the interface between carbon fiber bundles, which corresponded to the white lines clearly visible in strand 2. Figure 4.17 was a close-up of the intersection between two strands. A few voids were visible in the coating.

Table 4.2: Water absorption after 24 hr and at saturation (%)

Sample	24 hr	saturation
1	0.7	1.1
2	0.7	1.1
3	0.8	1.1
Average	0.8	1.1
S.D	0.01	0.0

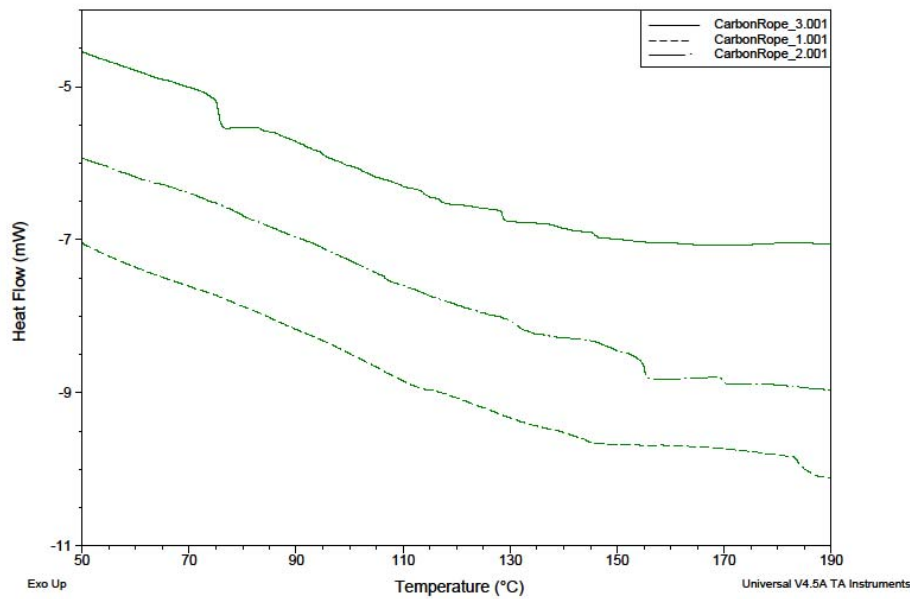


Figure 4.12: DSC curves of carbon Rope cable.

SEM micrographs of transverse and longitudinal section of the sample were presented in Figure 4.18 to Figure 4.24. Figure 4.18 shows a general view of the cross-section at the intersection between two strands. . A large pore was found in a SEM micrograph at the intersection of two strands. Figure 4.19 clearly shows the tubular shape of the pores in the cross section of a strand. Figure 4.20 and Figure 4.21 present the cross-section of a strand at medium and high magnification, respectively. The bonding between the fibers and the resin is good since no free gap was detected at the interface. Figure 4.22 and Figure 4.23 show the longitudinal section of the material at medium and higher magnification, respectively. Besides, the adhesion between the fibers and the resin matrix seems appropriate, in spite of the presence of a slight debonding around certain fibers (Figure 4.22). However, debonding may occur here and there, such as observed in the micrograph of Figure 4.24 obtained at very high magnification. The level of debonding seemed very low and should not affect the properties and durability of the FRP cable.



Figure 4.13: General view of carbon Rope cross-section by optical microscopy

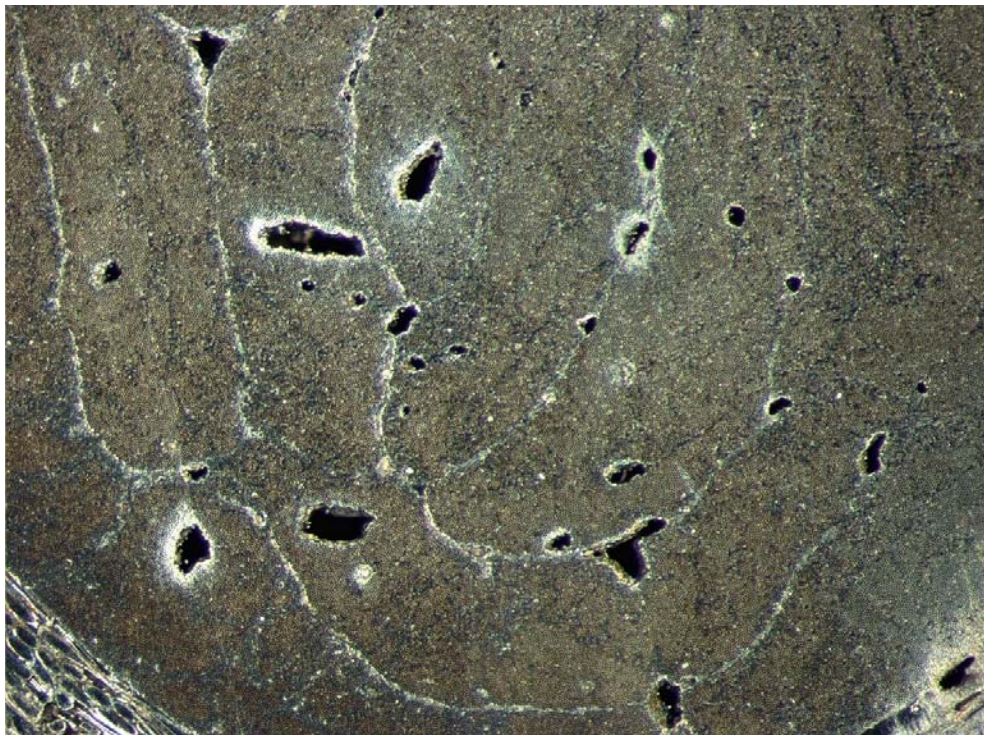


Figure 4.14: View of strand 1

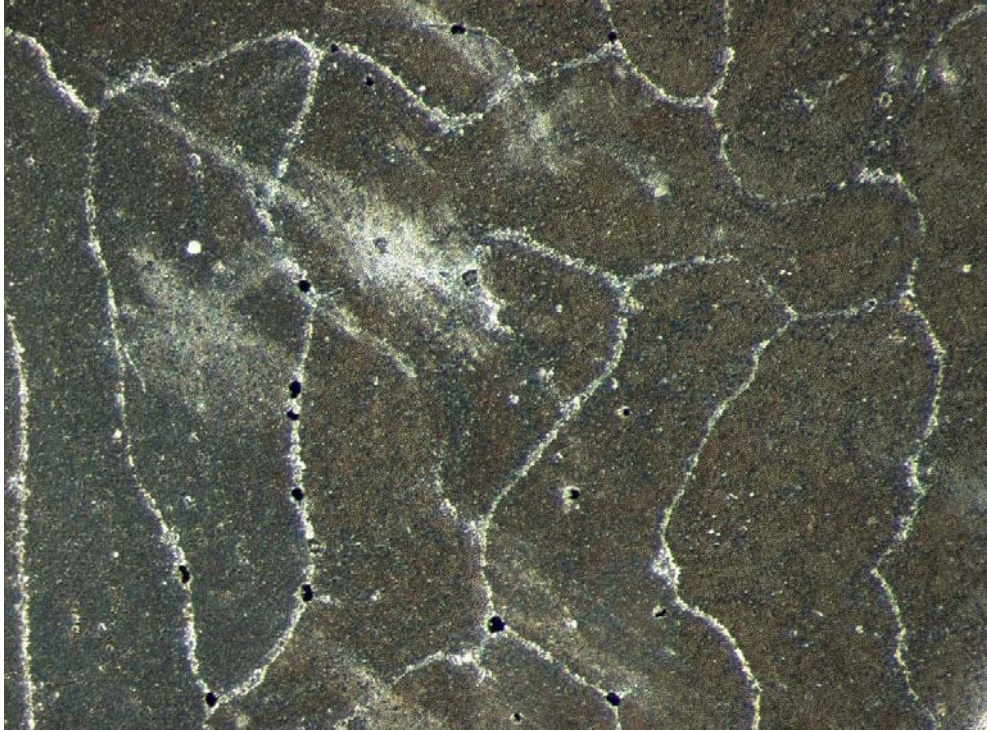


Figure 4.15: View of strand 2

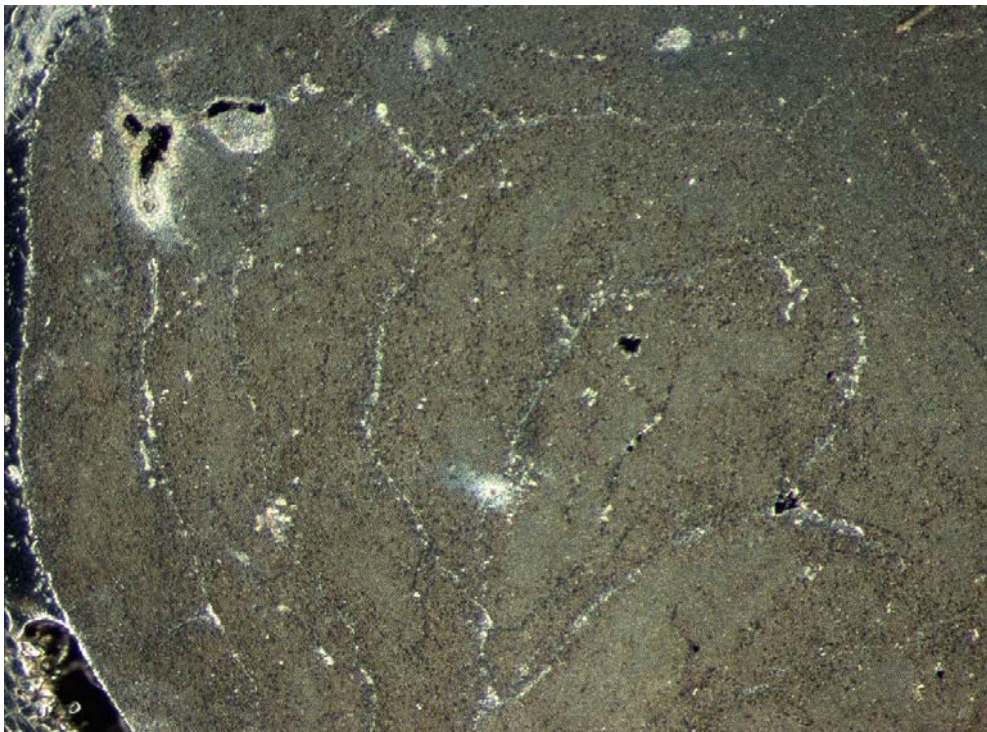


Figure 4.16: View of strand 3

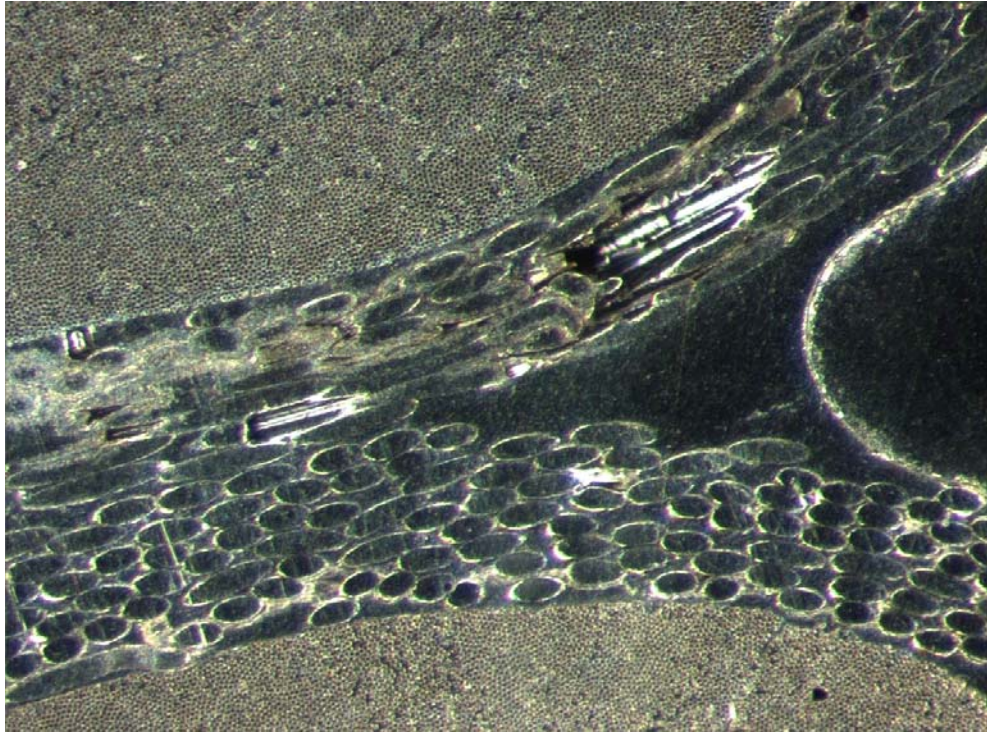


Figure 4.17: Intersection between two strands showing two pores

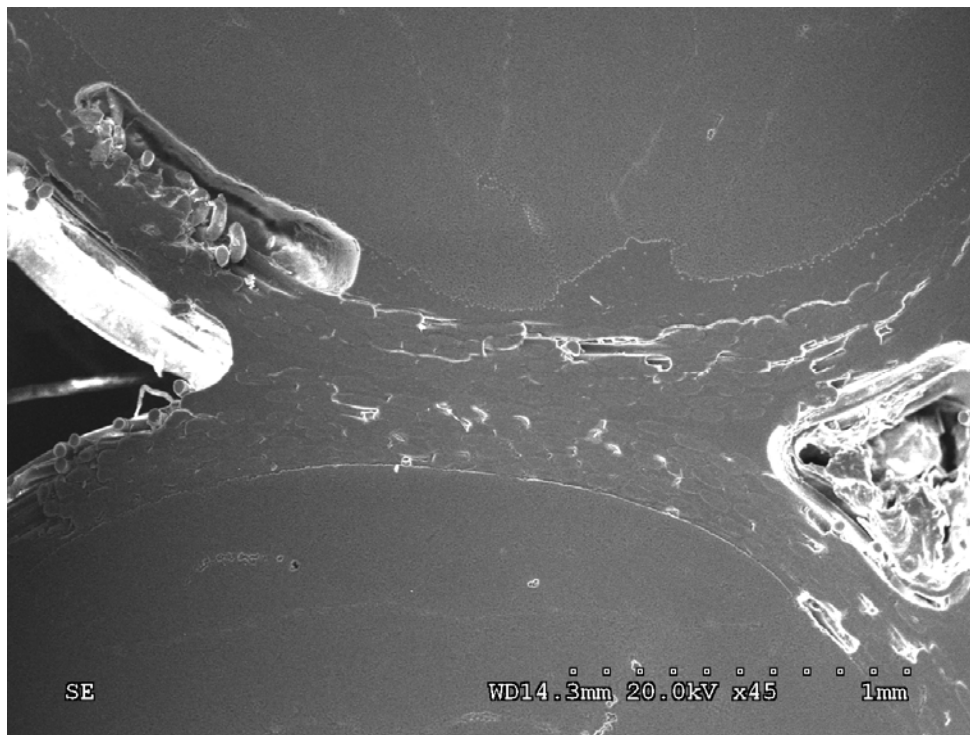


Figure 4.18: SEM micrograph of two strands of carbon Rope at low magnification

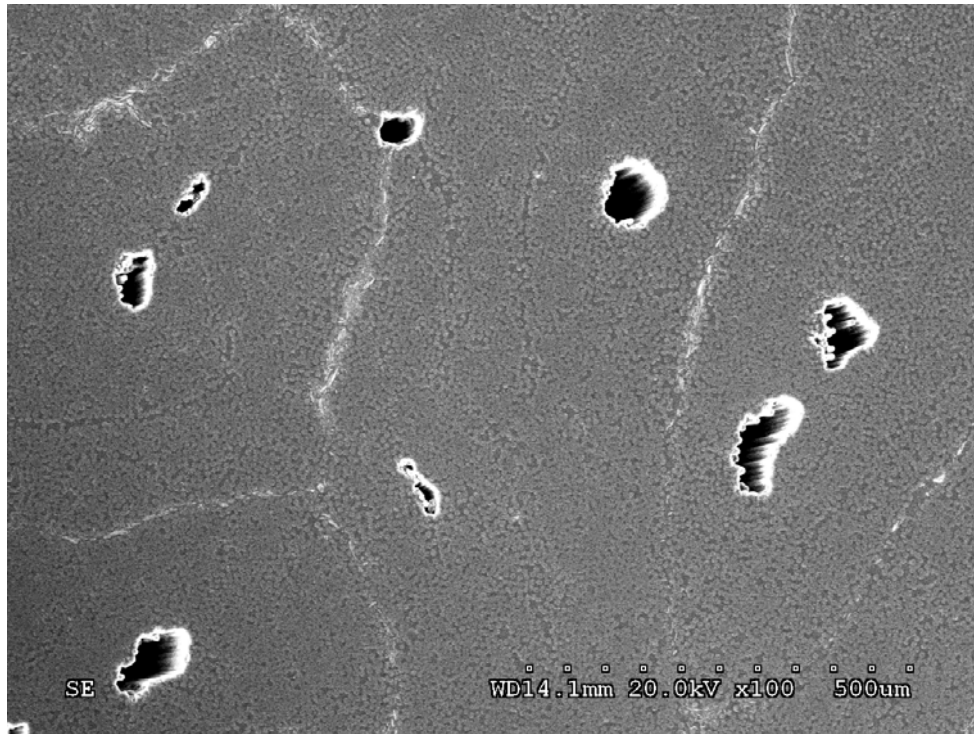


Figure 4.19: Tubular pores in a strand

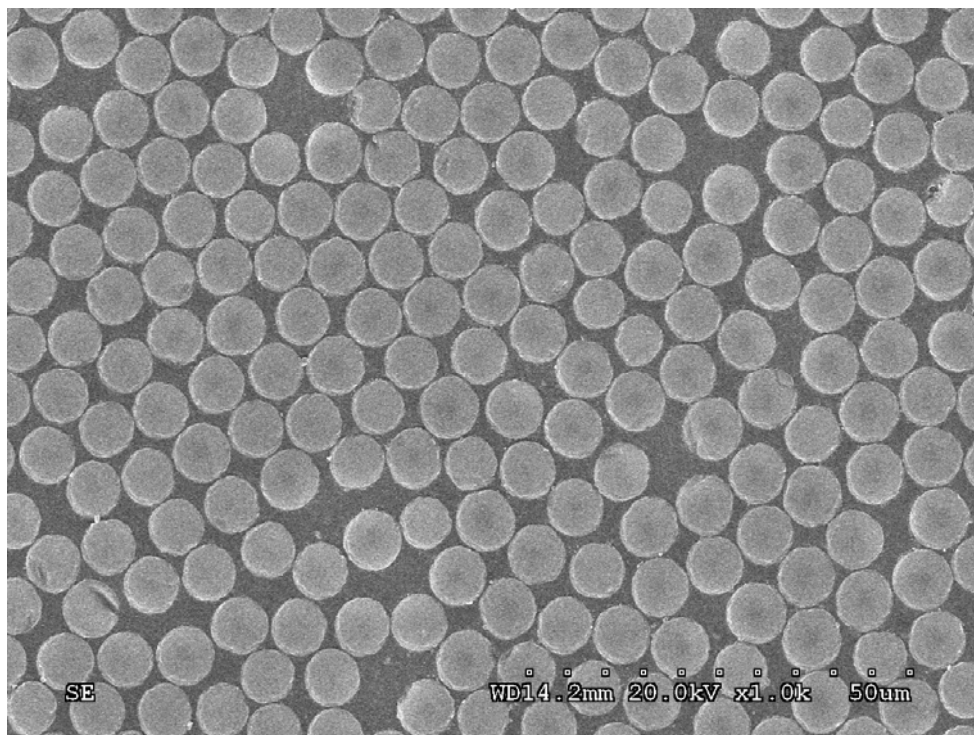


Figure 4.20: Cross-section showing carbon fibers at medium magnification

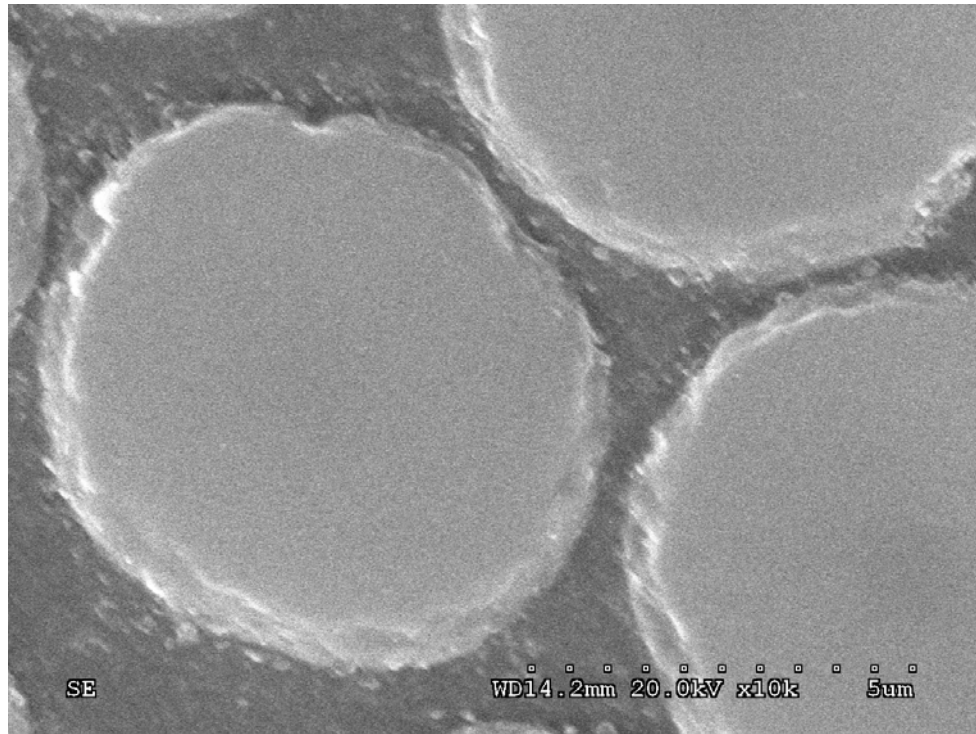


Figure 4.21: Carbon fibers at high magnification

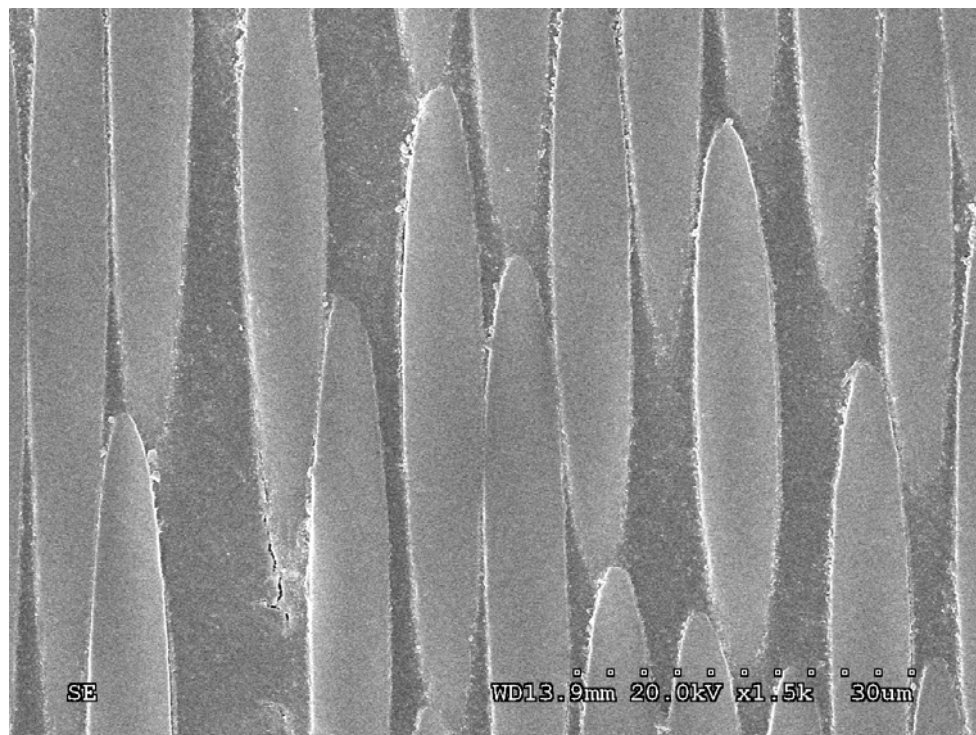


Figure 4.22: Longitudinal section of carbon Rope at medium magnification

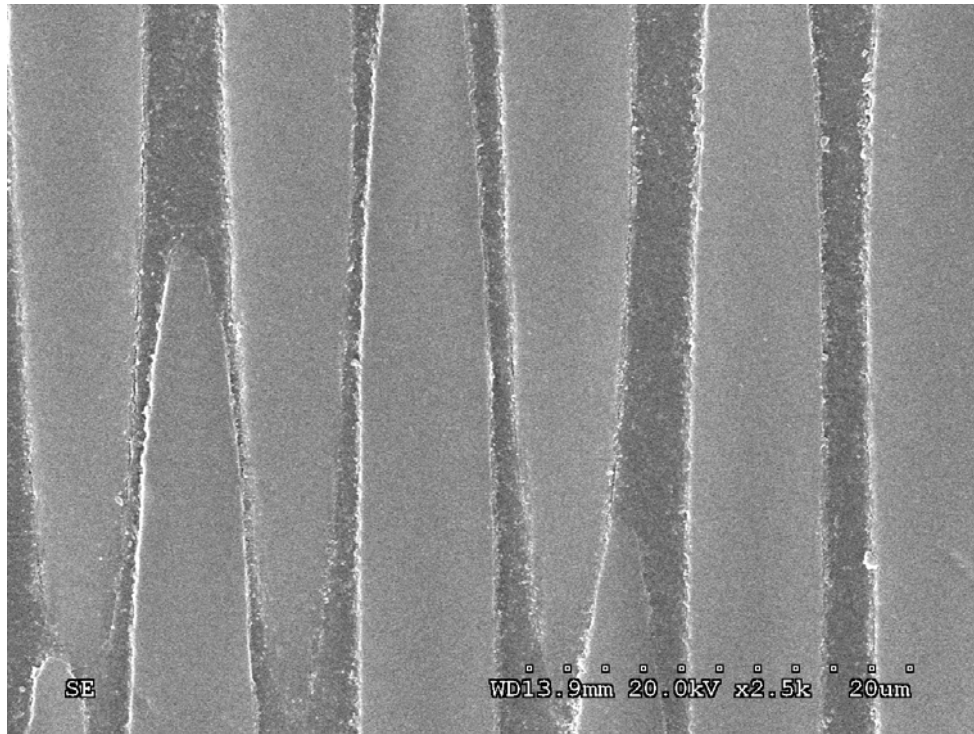


Figure 4.23: Longitudinal section of Tokyo Rope CFCC at higher magnification

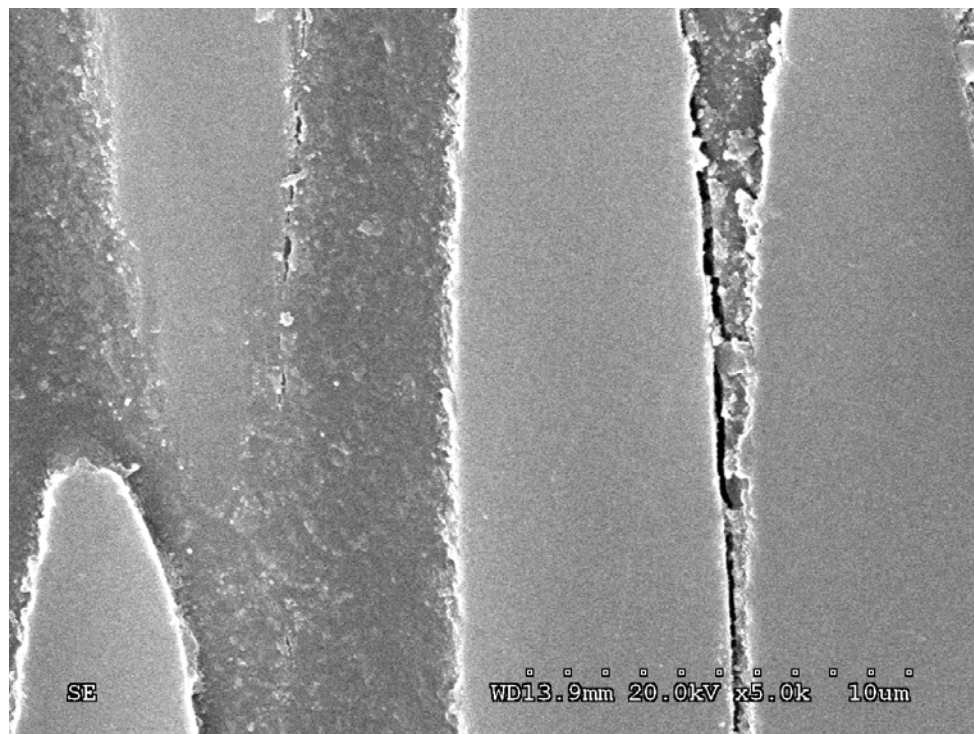


Figure 4.24: Debonding at the interface between a carbon fiber and resin

4.3.2 Tensile Properties of Long-Term Durability of Tokyo Rope (CFCC)

a) Calculations

The tensile strength, f_u and modulus of elasticity of GFRP bars were calculated by Eqns 3.5 and 3.6 while the tensile property retention strength and elastic modulus were calculated according to the Eqns 3.7 and 3.8. The “Guaranteed Tensile Strength”, (f_u^*), the “Guaranteed Modulus of Elasticity”, E_f^* , and the guaranteed strain (ε_u^*) were calculated as described in the (ACI-440.1R, 2006) as follows:

$$f_u^* = f_{u,ave} - 3 SD \quad (4.2)$$

$$E_f^* = E_{f,ave} \quad (4.3)$$

$$\varepsilon_u^* = \varepsilon_{u,ave} - 3SD \quad (4.4)$$

Where: (SD) is the standard deviation.

b) Test results and discussion

Table 4.3 presents the test results of the tensile properties of the reference and conditioned specimens after immersion period 1,000, 3,000, 5,000, 7000 hours. This table gives the average, standard deviation, coefficient of variation, and retention values for the tensile strength, Young's modulus, and strain in the control and conditioned CFCC specimens. The average tensile strength and modulus of elasticity of the control specimens were 3154 ± 72.4 MPa, and 151 ± 0.9 GPa, respectively. The guaranteed tensile strength and modulus of elasticity of the tested CFCC cable are set by the manufacturer to 2440 MPa, and 155 GPa, respectively. The results of average tensile strength obtained after conditioning for 1000 hours at 22°C, 40°C, 50°C, and 60°C were 3145, 3118, 3092, and 3077 MPa, respectively. The corresponding values after conditioning for 3000 hours were 3122, 3099, 3086, and 3067 MPa. The results of average tensile strength obtained after conditioning for 5000 hours at 50°C, and 60°C were 3083, and 3014 MPa, respectively. The corresponding values after conditioning for 7000 hours were 2958, and 2928 MPa. Evidently, the tensile strength of the conditioned CFCC specimens decreased as the immersion time and temperature increased. The tensile-strength retentions were about 97.6%, 97.2%, 95.6%, and 92.8% after 1000, 3000, 5000, and 7000 hours of immersion at 60°C, respectively. These results indicate that the strengths of the CFCC specimens were affected by increasing the immersion duration at higher temperature levels.

On the other hand, the environmental conditioning slightly affected the elastic modulus of the aged CFCC specimens. The results of average elastic modulus obtained after conditioning for 1000 hours at 22°C, 40°C, 50°C, and 60°C were 150.3, 150, 150.4, and 150.0 GPa, respectively. The corresponding values after conditioning for 3000 hours were 146.9, 148.2, 148.5, and 148.1 GPa. The results of average elastic modulus obtained after conditioning for 5000 hours at 50°C, and 60°C were 147.6, and 147.0 GPa, respectively. Also, the corresponding values after conditioning for 7000 hours were at 50°C, and 60°C were 147.4, and 147.1 GPa, respectively. Increasing the duration of immersion time from 1000 to 7000 hours decreased the elastic modulus ap-

proximately from 150 GPa to 147 GPa, respectively, at different temperatures (40°C, 50°C, and 60°C).

The test result indicated that the environmental conditioning slightly affected the ultimate strain of the aged CFCC specimens. The results of average ultimate strain obtained after conditioning for 1000 hours at 22°C, 40°C, 50°C, and 60°C were 2.09 %, 2.08 %, 2.06 %, and 2.05 %, respectively. The corresponding values after conditioning for 3000 hours were 2.12 %, 2.09 %, 2.07 %, and 2.07%. The results of average ultimate strain obtained after conditioning for 5000 hours at 50°C, and 60°C were 2.09 %, and 2.05 %, respectively. In addition, the corresponding values after conditioning for 7000 hours were at 50°C, and 60°C were 2.01 %, and 1.99 %, respectively. Increasing the duration of immersion time from 1000 to 7000 hours decreased the ultimate strain approximately from 2.09 % to 1.99 %, respectively, at different temperatures (40°C, 50°C, and 60°C).

Finally, the test results shown in Table 4.3 indicate that the strain-retention capacities were 98.6%, 99.5%, 98.6%, and 95.7%, after 1000, 3000, 5000, and 7000 hours of immersion at 60°C, respectively. The reduction of the tensile properties of the CFCC conditioned specimens could not be attributed to any significant degradation or reduction in carbon fibers tensile properties. The strength reduction of the conditioned specimens is instead due the interface fiber/resin degradation that reduces the stress transfer. Other studies such as SEM analyses are on-going to investigate the interface degradation of the Tokyo Rope Carbon Cable in alkaline environment and elevated temperatures.

Due to the brittle nature of CFCC, no yielding occurred and the stress–strain behavior was linear. Figure 4.25 to Figure 4.33 show the typical failure mode of the CFCC. It can be noted that the tested bars failed in the middle of the bar (gauge length). The test results indicate that the aging condition with alkaline solution at different temperatures did not affect the failure mode significantly.



Figure 4.25: Mode of failure of conditioned specimens at 22°C after 1,000 hr



Figure 4.26: Mode of failure of conditioned specimens at 40°C after 1,000 hr



Figure 4.27: Mode of failure of conditioned specimens at 50°C with 1,000 hr



Figure 4.28: Mode of failure of conditioned specimens at 60°C with 1,000 hr



Figure 4.29: Mode of failure of reference specimens



Figure 4.30: Mode of failure of conditioned specimens at 22° C with 3,000 hr



Figure 4.31: Mode of failure of conditioned specimens at 40° C with 3,000 hr



Figure 4.32: Mode of failure of conditioned specimens at 50° C with 3,000 hr



Figure 4.33: Mode of failure of conditioned specimens at 60°C with 3,000 hr



Figure 4.34: Mode of failure of conditioned specimens

Table 4.3: Tensile properties (reference and conditioned at 22, 40, 50, and 60 °C) after 1,000, 3,000, 5,000, and 7,000 hrs

Time of Immersion (hour)	Temperature (°C)	Number of Specimens	Tensile Strength (MPa)			Modulus of Elasticity (GPa)			Ultimate Strain (%)		
			Average	COV (%)	Retention (%)	Avg.	COV (%)	Retention (%)	Avg.	COV (%)	Retention (%)
0	22	5	3154±72.4	2.30	100	151.0±0.9	0.61	100	2.08±0.04	1.71	100
1000	22	3	3145±81.6	2.63	99.7	150.3±1.8	1.18	99.5	2.09±0.04	2.18	100
	40	3	3118±65.3	2.15	98.9	150.0±0.3	0.17	99.3	2.08±0.04	2.16	100
	50	3	3092±27.3	0.88	98.0	150.4±1.2	0.80	99.6	2.06±0.03	1.49	99.1
	60	3	3077±89.4	2.97	97.6	150.0±1.4	0.91	99.3	2.05±0.04	2.06	98.6
3000	22	3	3122±47.2	1.50	98.9	146.9±0.5	1.50	97.3	2.12±0.04	1.97	100
	40	3	3099±53.7	1.74	98.3	148.2±1.3	0.87	98.2	2.09±0.05	2.59	100
	50	3	3086±90.9	4.12	97.8	148.5±1.8	1.21	98.4	2.07±0.06	2.88	99.5
	60	3	3067±94.4	2.99	97.2	148.1±1.6	1.09	98.1	2.07±0.08	3.68	99.5
5000	22	3	3060±44.2	1.44	97.02	148.23±0.4	0.3	98.16	2.06±0.03	1.45	99.03
	40	3	3100±54.0	1.74	98.28	149.39±0.8	0.54	98.93	2.08±0.03	1.25	100
	50	3	3083±59.4	1.93	97.7	147.6±1.0	0.68	97.8	2.09±0.03	1.25	100
	60	3	3014±112.6	3.74	95.6	147.0±0.7	0.48	97.3	2.05±0.07	3.27	98.6
7000	22	3	3129±14.96	0.48	99.2	147.4±0.36	0.24	97.6	2.12±0.01	0.52	100
	40	3	3054±156.6	5.13	96.8	148.4±1.17	0.79	98.3	2.06±0.09	4.58	99
	50	3	2958±213.7	7.23	93.8	147.4±0.4	0.30	97.6	2.01±0.14	6.95	96.6
	60	3	2928±10.3	0.35	92.8	147.1±0.3	0.22	97.4	1.99±0.01	0.54	95.7

Table 4.4: Guaranteed tensile strength and modulus of elasticity retentions after 1,000, 3,000, 5,000 and 7,000 hrs

Bar Status	Elastic tensile modulus, E_t (GPa)				Guaranteed Tensile Strength, f_u^* (MPa)			
	1000 hrs	3000 hrs	5000 hrs	7000 hrs	1000 hrs	3000 hrs	5000 hrs	7000 hrs
Conditioned at 22 °C	150.3	146.9	148.7	147.4	2900.2	2980.4	2927.4	3084
Conditioned at 40 °C	150	148.2	149.4	148.4	2922.1	2937.9	2937.8	2584
Conditioned at 50 °C	150.4	148.5	147.6	147.4	3010.1	2813.3	2904.8	2317
Conditioned at 60 °C	150	148.1	147.0	147.1	2808.8	2783.8	2676.2	2897

Usually, a normal (Gaussian) distribution is assumed to represent the strength of a population of bar specimens (ACI-440.1R 06). ACI-440.1R 06 stated that the manufacturers should report the guaranteed tensile properties of FRP reinforcement as mentioned in Equations 4.2, 4.3, and 4.4 for strength, modulus, and strain, respectively, to be used in any design calculation. These guaranteed values of strength and strain provide a 99.87% probability that the indicated values are exceeded by similar FRP bars, provided that at least 25 specimens are tested (ACI-440.1R, 2006). Also, ACI 440.1R 06 stated that the material properties provided for FRP reinforcement, such as the guaranteed tensile strength, should be considered as initial properties that do not include the effects of long-term exposure to the environment. Because long-term exposure to various types of environments can reduce the tensile strength and creep rupture and fatigue endurance of FRP bars, the material properties used in design equations should be reduced based on the type and level of environmental exposure. Therefore, one of the main objectives of this study is to determine the effect of alkaline conditioning on the guaranteed tensile strength of the investigated CFCC Tokyo Rope cables to be used as reinforcement for prestressed concrete structures.

Table 4.43 presents the average tensile capacity retention properties of the conditioned specimens after conditioning to 1,000, 3,000, 5,000, and 7,000 hours at different temperature levels. Also, Figure 4.355 to Figure 4.388 represent the guaranteed tensile strength versus impression time for the four different temperature exposure levels used in this report (22, 40, 50, and 60°C). In addition, the test results of the 1,000, 3,000, 5,000 and 7,000 hours of the guaranteed tensile capacity are presented in Figure 4.439 to Figure 4.442. The guaranteed tensile strength retention of the tested Tokyo Rope CFCC conditioned for 1,000, 3,000, 5,000, and 7,000 hours in alkaline solution (pH of 12.8) without load at 22, 40, 50 and 60°C is over 95.6%. Moreover, the value of the elastic modulus retention in the three conditioning phases (1,000, 3,000, and 5,000 hours) at 60°C varied between 99.3% at 1000 hrs and 97.3% at 5000 hrs of the reference elastic modulus value. Figure 4.4342 to Figure 4.446 show tensile capacity retention in a logarithmic scale for the different exposure condition. Presentation of data on a logarithmic scale was intended to predict the tensile properties of tested specimens for long-term exposure using the regression analysis. The long-term predictions revealed that the effect of alkaline conditioning on the average tensile capacity retention at 60°C could be slightly higher than that at 22°C, 40°C and 50°C. As presented in Table 4.3, the average tensile capacity retention after 5,000 hours of exposure ranged between ~97% and ~98% for temperature levels 22°C, 40°C and 50°C; yet it dropped to 95.6% at 60°C. After 7,000 hours of exposure, the average tensile capacity retention at 60°C dropped to 92.8%. In addition, the modulus of elasticity retention after 7,000 hours of exposure at 60 °C was 97.4%. Table 4.4 presents the guaranteed tensile capacity properties of the conditioned specimens without load after conditioning to 1,000, 3,000, 5,000, and 7,000 hours at different temperature levels. However, the manufacturer conducted testing of specimens after conditioning in a alkaline solution with pH of 13 for 2,500 hours at 60°C under 70% of guaranteed breaking load. The reported losses of tensile strength and modulus of elasticity were 2.4% and 1.0%, respectively.

In our research, after 7,000 hours of exposure at 60°C without loading, the tensile capacity retention decreased by about 7%. However, the modulus of elasticity retention after 7,000 hours of exposure at 60 °C only dropped by less than 3%. Figure 4.455 and Figure 4.466 show that the trend lines of the guaranteed tensile capacity retention versus the conditioning exposure time for the specimens conditioned at 50°C and 60°C tend to decrease than that of specimens conditioned at 22°C and 40°C shown in Figure 4.45.43 and Figure 4.4644.

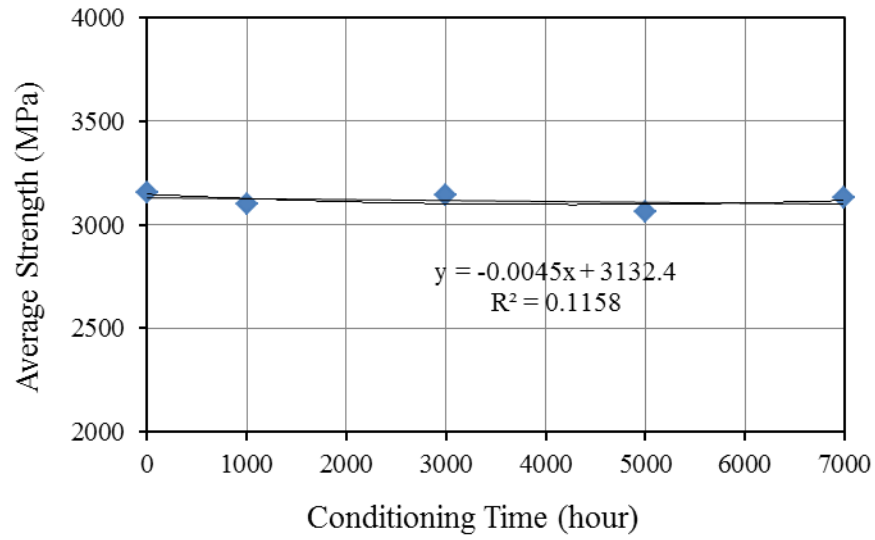


Figure 4.35: Average tensile strength versus conditioning time at 22°C

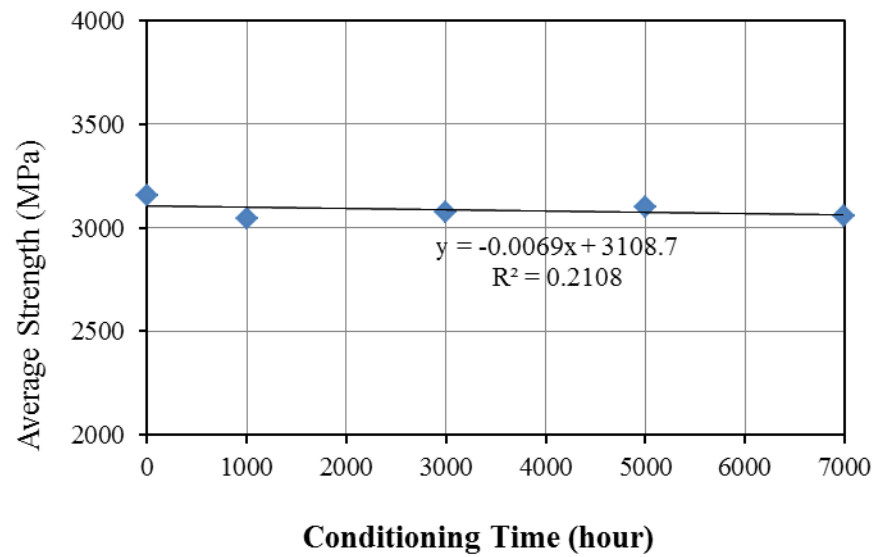


Figure 4.36: Average tensile strength versus conditioning time at 40°C

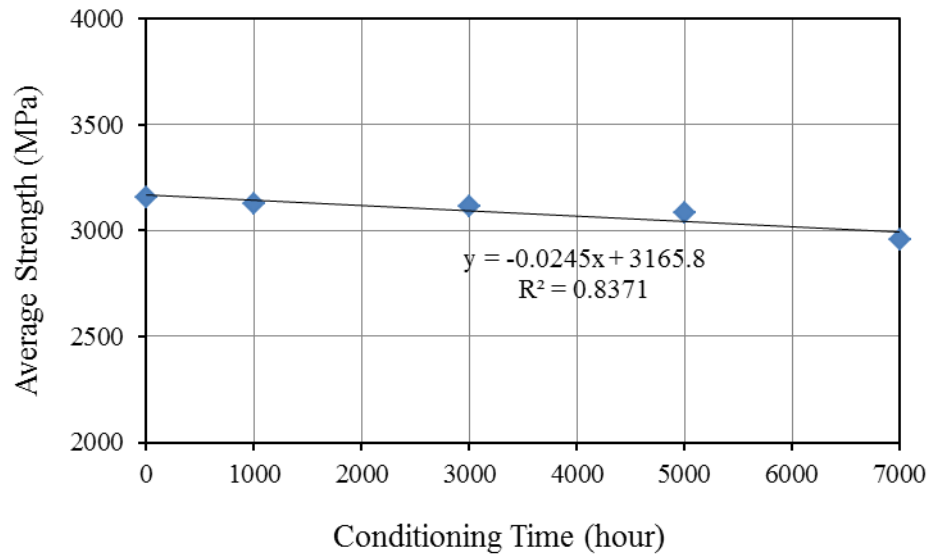


Figure 4.37: Average tensile strength versus conditioning time at 50°C

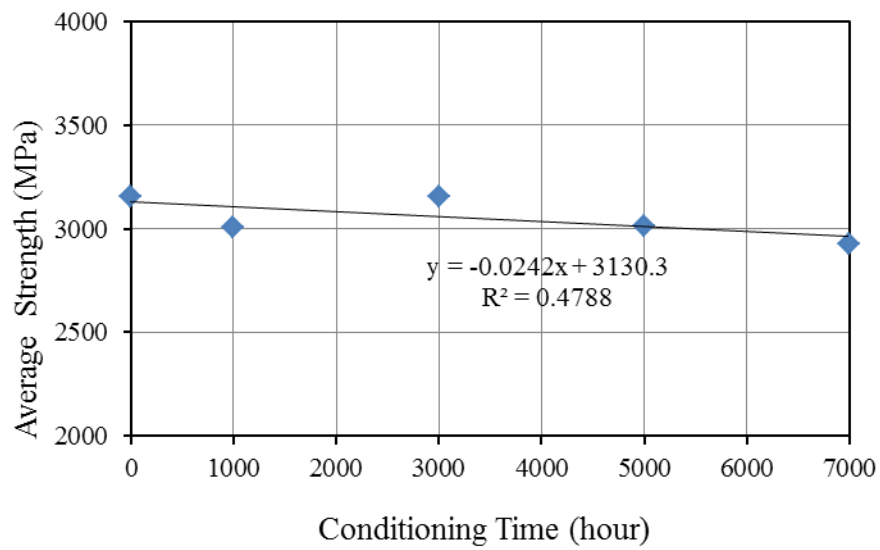


Figure 4.38: Average tensile strength versus conditioning time at 60°C

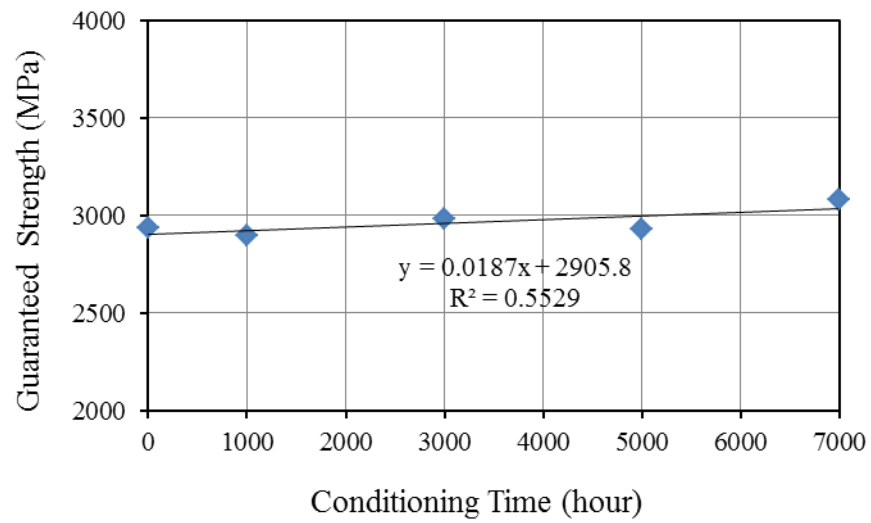


Figure 4.39: Guaranteed tensile strength versus conditioning time at 22°C

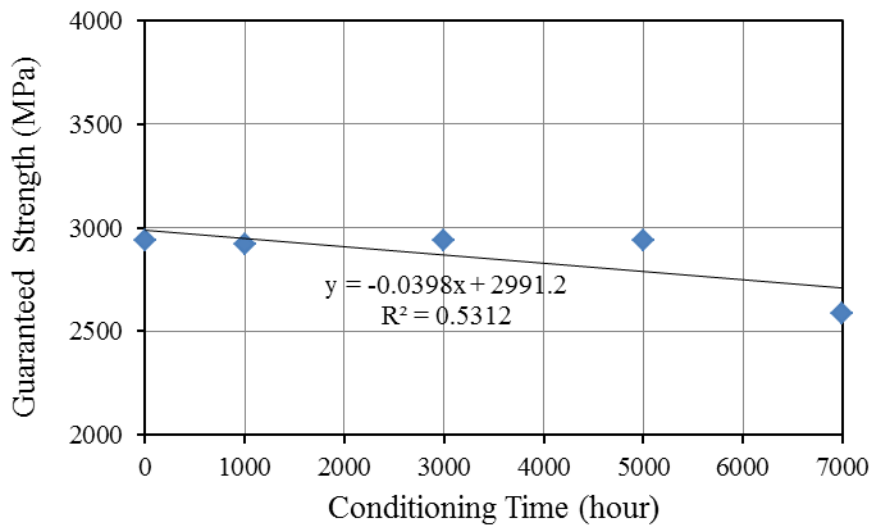


Figure 4.40: Guaranteed tensile strength versus conditioning time at 40°C

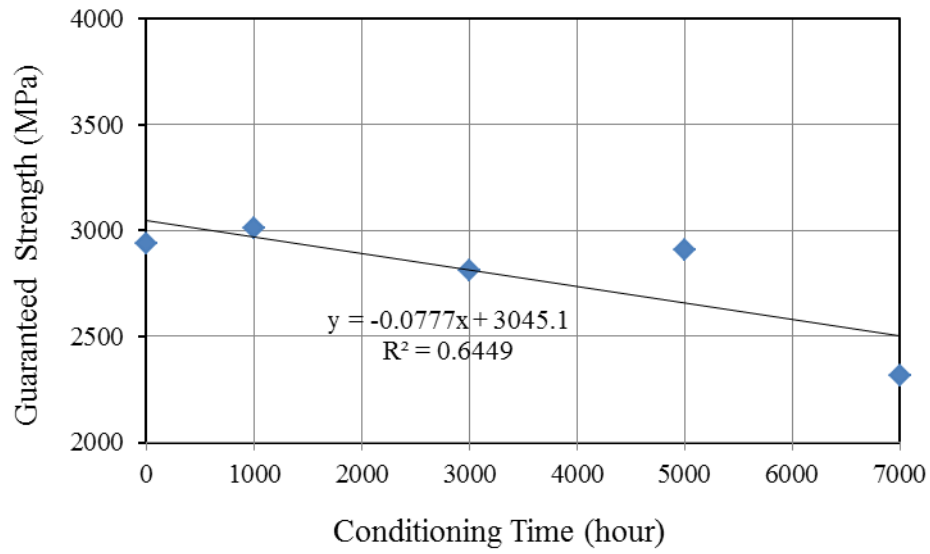


Figure 4.41: Guaranteed tensile strength versus conditioning time at 50°C

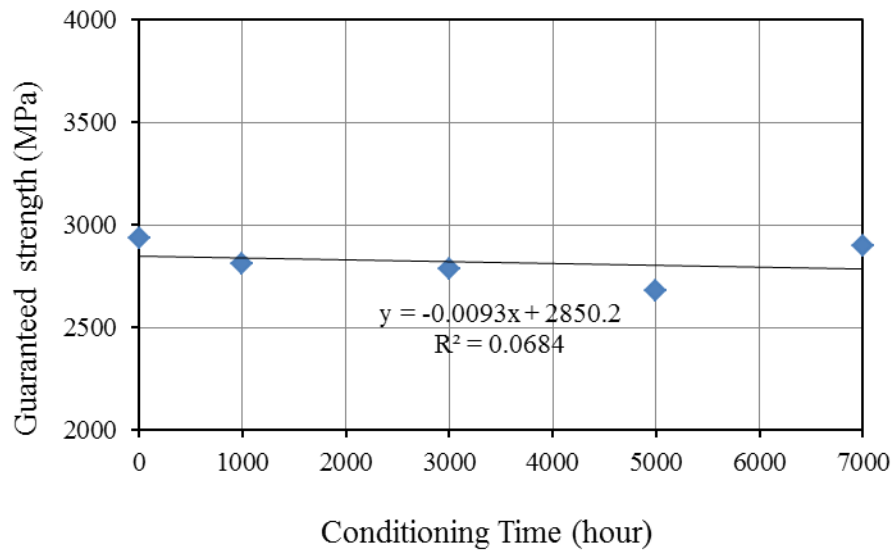


Figure 4.42: Guaranteed tensile strength versus conditioning time at 60°C

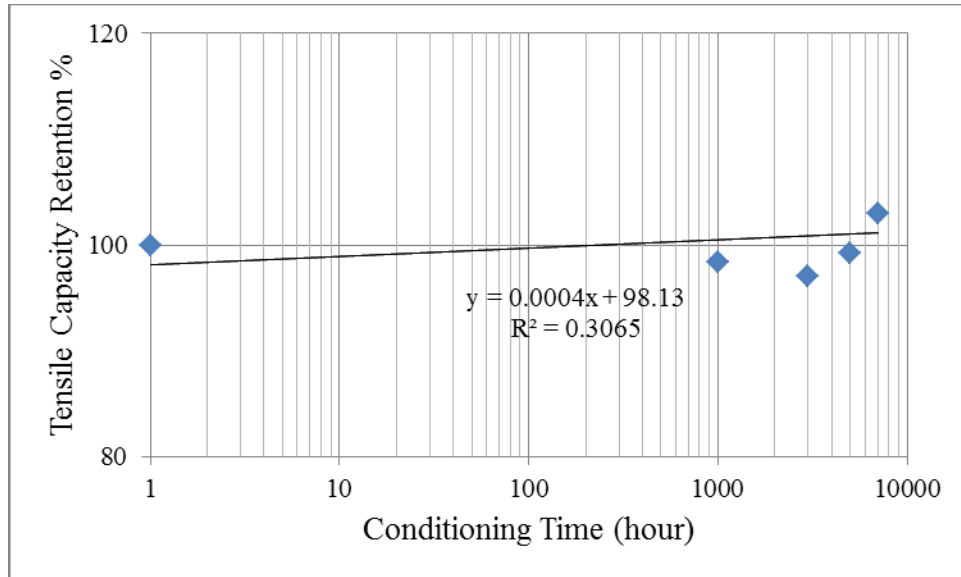


Figure 4.43: Tensile capacity retention with conditioning time at 22°C

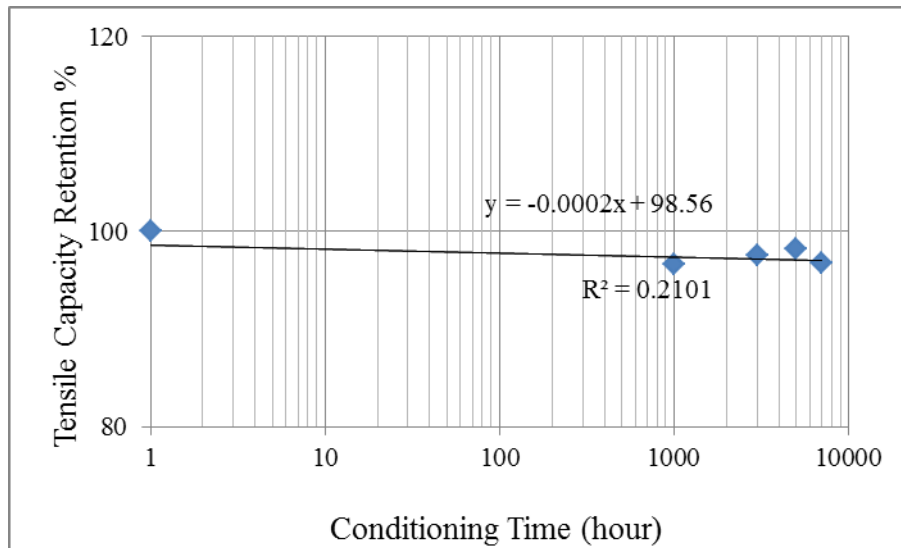


Figure 4.44: Tensile capacity retention versus conditioning time at 40°C

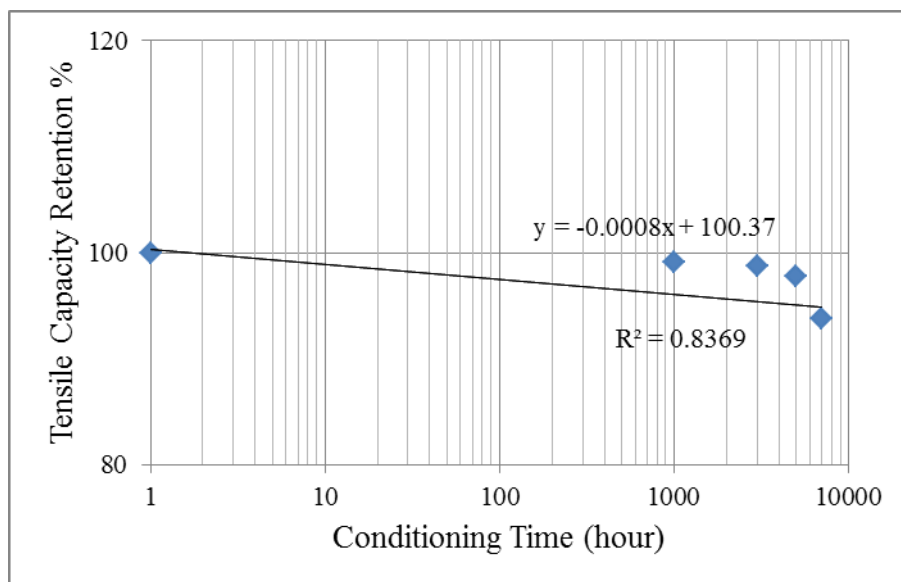


Figure 4.45: Tensile capacity retention versus conditioning time at 50°C

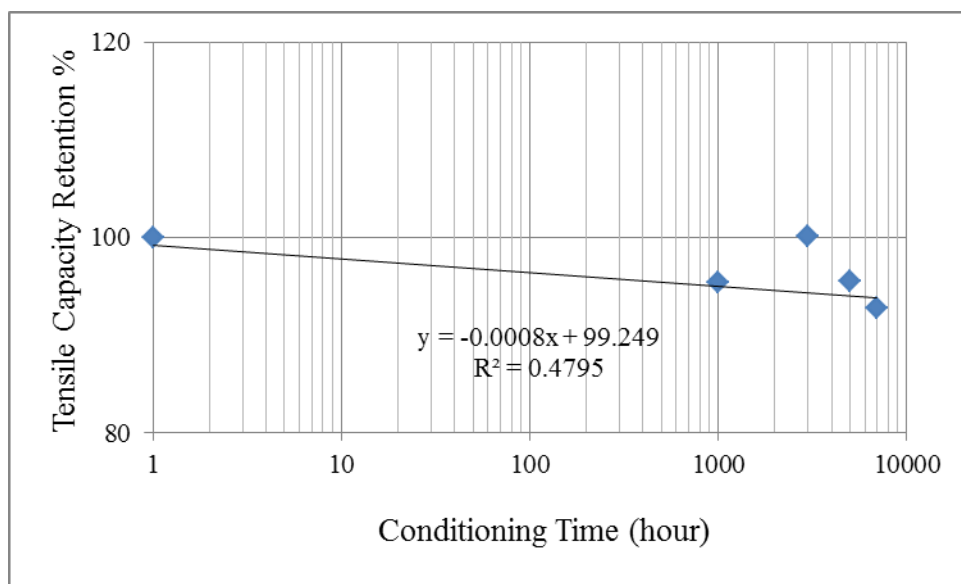


Figure 4.46: Tensile capacity retention versus conditioning time at 60°C

4.4 Conclusions

This study represented an overview on the physical, microstructural analyses, and preliminary durability characterization on CFCC cables. Unstressed CFCC rope specimens (7.5 mm diameter) were immersed for up to 7,000 hours in an alkaline solution with pH of 12.8 accelerated at various temperatures of 22°C, 40°C, 50°C, and 60°C to simulate the effect of alkalinity in concrete. Based on the experimental physical and durability results, the following conclusions were drawn:

1. The test observation indicated that the carbon fiber content is 82% by weight and the water uptake at saturation is equal to 1.1%. The cure ratio of the material is very high (close to 100%) but its glass transition temperature is not clearly visible by DSC.
2. Optical microscopy and electronic scanning microscopy analysis showed that strands presented various levels of porosity. However, no major defect was detected in the tested CFCC material.
3. The results indicate that the strengths of the CFCC specimens were slightly affected by increasing the immersion duration at higher temperature levels. After 7,000 hours of immersion in the alkaline solution at 60°C, test result indicated that almost 7% degradation in the average tensile strength occurred. However, the reduction in the average modulus of elasticity after, 7,000 hours of immersion in the alkaline solution at 60°C, was only 2.6%. The strength reduction could be attributed to an interface degradation that is usually caused by a sizing problem. Further investigations are required on this topic.
4. Long-term durability characterization of CFCC is needed to evaluate the effect of alkaline environment at different temperature exposures and under different sustained tensile load levels (from 30% up to 75% of ultimate tensile load). This study is highly recommended to determine the critical stress (allowable) and safety factors for the use of CFCC as corrosion-resistant reinforcing material for prestressed precast-concrete bridge-deck and pile applications in Florida marine environment.

CHAPTER 5

CHARACTERIZATION AND DURABILITY OF BASALT FIBER-REINFORCED POLYMER BARS

5.1 Introduction

Significant research efforts over the past twenty years have been performed on fiber-reinforced polymer (FRP) materials. These efforts showed that the FRP materials can be used effectively to reinforce and/or strengthen deteriorated or understrength reinforced-concrete (RC) structures. Recently, basalt FRP (BFRP) reinforcing bars have been introduced into the market. However, due to the limited research on BFRP bars for structural applications, further investigations are required to provide confidence in the use of BFRP in RC structures. The physical and mechanical properties, durability, and bond strength to concrete are very important characteristics that should be examined before using them as main reinforcement for concrete structures.

The test program aims at evaluating the physical, mechanical, and durability characteristics of different types of BFRP reinforcing bars provided by different manufactures. This chapter contains physical and mechanical characterization of BFRP bars having different shape and surface configuration. The physical properties of seven types of BFRP bars, namely (A-B-C-D-E-F-G-H), were investigated, whereas bars Type (A) and (G) having different sizes (6, 8, 10, 12, 16, and 25 mm-in diameters) and (No. 3, 4, and 5), respectively. Figure 5.1 to Figure 5.11 show the surface configuration of each BFRP bar. Besides, the physical properties and SEM analysis, transverse shear, and tensile strength properties of TECH-12 mm (No.4), RWS (No. 4, 5, and 8), I and J basalt bars types were examined, respectively. Materials characterization was investigated for newly developed prestressing BFRP bars 7 mm-diameter and BASA BFRP 8-mm-diameter (codes 43 and 51) for reference and conditioned specimens immersed in an alkaline solution at different environmental temperatures and exposure time. Furthermore, mechanical characterization was conducted for BASA BFRP 12 mm- diameter for reference and conditioned specimens immersed in an alkaline solution at 60°C for 60 days in term of tensile, transverse and interlaminar shear, and flexural strengths. While the tensile properties of BASA BFRP 16 mm-diameter specimens. Table 5.1 shows the different tests performed on different BFRP bars types. Finally, experimental testing studies of RC members (beams and one-way slabs) reinforced with BFRP bars were investigated.

Table 5.1: Different tests performed on BFRP bars

	Test Performed							Remarks
Basalt Specimens Label	Physical Proper- ties	SEM	Mechanical Properties					
			Tensile strength	Transverse Shear strength	Interlaminar Shear strength	Flexural strength	Bond strength	
A (No.2 to No. 8)	✓	✓	-	-	-	-	-	
B to F	✓	✓	-	-	-	-	-	
G (No.3 to No. 5)	✓	✓	-	-	-	-	-	
H	✓	✓	-	-	-	-	-	
I & J	-	-	✓	-	-	-	✓	Reference
RWS (No. 4, 5, & 8)	-	-	-	✓	-	-	-	Reference and con- ditioned (2000 hrs.)
TECH- 12 mm (No.4)	✓	✓	-	-	-	-	-	
Prestressing-7 mm	✓	✓	✓	✓	✓	✓	✓	Reference and con- ditioned (1000 hrs.)
BASA- 8 mm, code 43 and 51	✓	✓	✓	✓	✓	✓	✓	Reference and con- ditioned (720 and 2160 hrs.)
BASA- 12 mm, code 43	-	-	✓	✓	✓	✓	✓	Reference and con- ditioned (1440 hrs.)
BASA- 16 mm	-	-	✓	-	-	-	-	Reference



Figure 5.1: BFRP bars Type A (from left to right 6, 8, 10, 12, 16, and 25 mm-designated diameter)

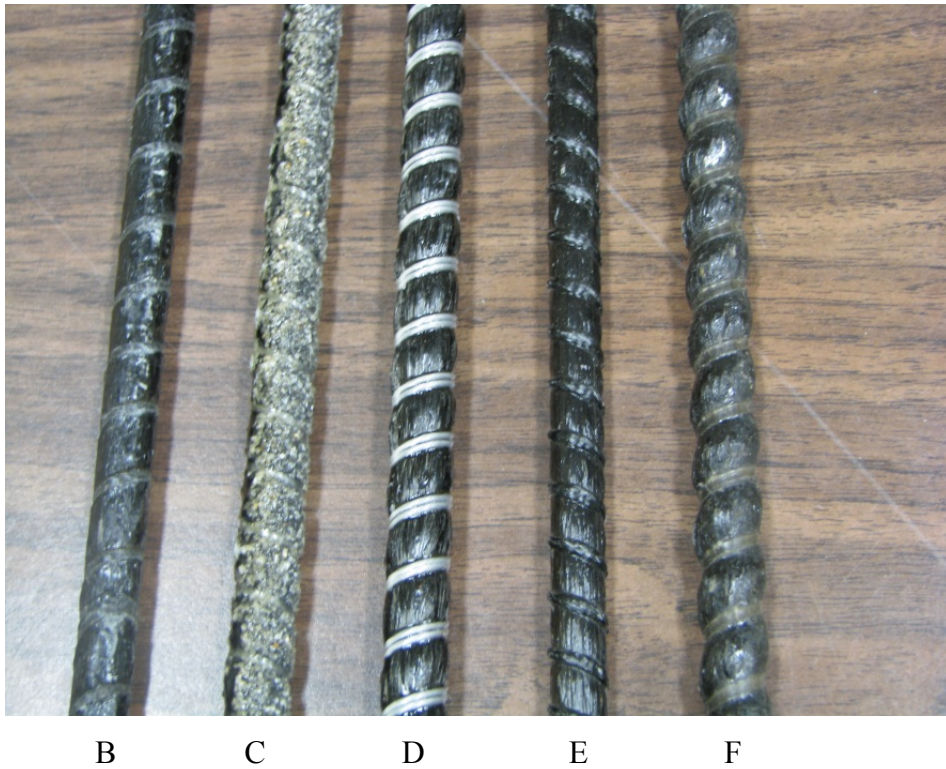


Figure 5.2: BFRP bars Type B to F



Figure 5.3: BFRP bars Type G



Figure 5.4: BFRP bars Type H



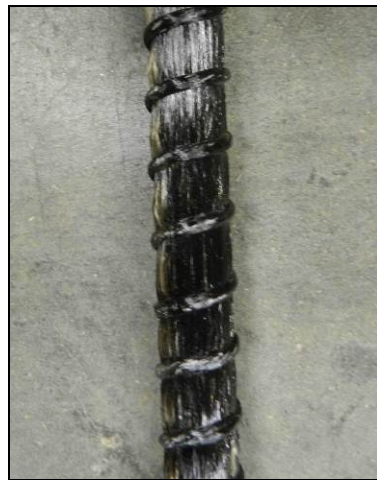
Figure 5.5: TECH BFRP bar of 12 mm- in diameter



Figure 5.6: RWS-BFRP bars No. 8



BFRP type I



BFRP type J

Figure 5.7: BFRP type I and J



Figure 5.8: Prestressing tendon BFRP bar of 7 mm- in diameter

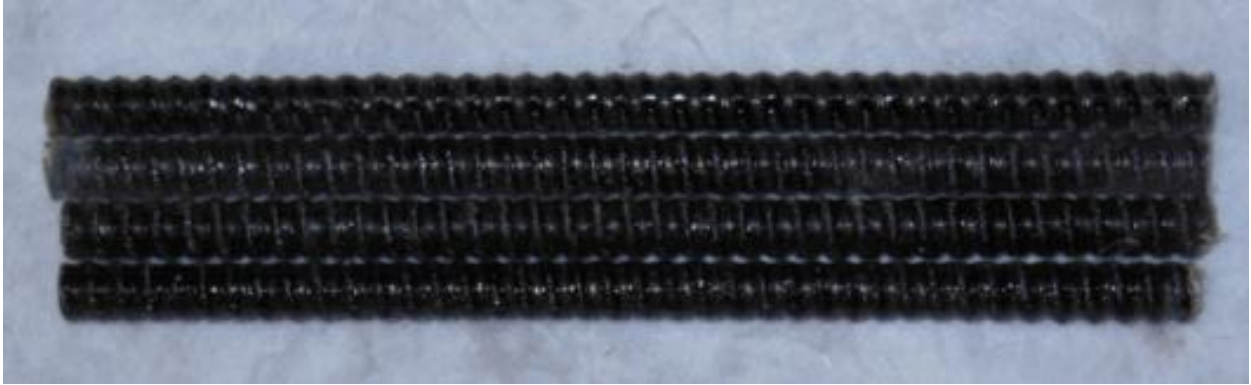


Figure 5.9: BASA-BFRP bars 8 mm- in diameter, codes 43 and 51

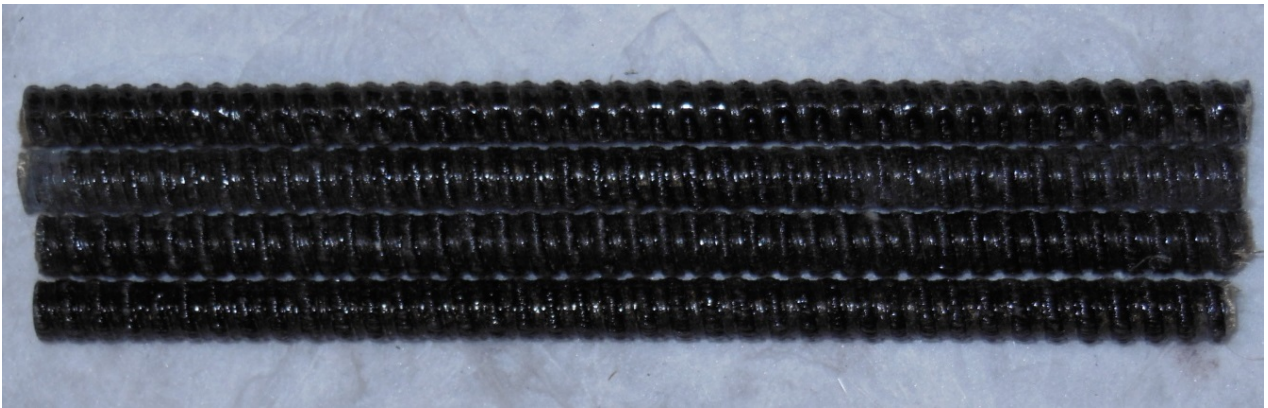


Figure 5.10: BASA-BFRP bars 12 mm- in diameter, code 43



Figure 5.11: BASA-BFRP bars of 16 mm- in diameter

5.2 Procedures

5.2.1 Physical Properties

This section presents test procedures measurements in order to determine the physical properties of the different BFRP bar size and type. The test procedures were carried out according to the specification of the (ACI-440.6M, 2008) and (CSA-S807, 2010). The measurements were presented in term of cross-sectional area, fiber content, transverse and longitudinal coefficient of thermal expansion, void content, water absorption, cure ratio, and glass transition temperature. Moreover, optical and scanning electronic microscopy (SEM) analyses were performed to investigate the microstructure of different BFRP bars size and type.

a) Cross-sectional area

The cross-sectional area of different BFRP bars were determined according to (ASTM D7205, 2011) – “*Standard Test Method for Tensile Properties of Fiber Reinforced Polymer Matrix Composite Bars*”. Five specimens of each bar type were cut to a length (L). The specimens length and weight were determined to calculate the linear mass while their densities were measured by water displacement using a glass cylinder filled with water to the top with precaution that no air bubbles were entrapped. The weight of the cylinder was determined without water and with adding water. Then the volume of added water, V_0 , was calculated. Afterwards, the water was removed and the specimen placed into the cylinder, while the cylinder filled again with water to the same level as previously. The weight of the cylinder was measured and the volume of added water, V_1 , was determined. The average cross-sectional area, A, was calculated as follows:

$$A = 1,000 \times (V_0 - V_1) / L \quad (5.1)$$

b) Fiber content

Fiber content was determined according to (ASTM D 3171, 2011) – “*Standard Test Methods for Constituent Content of Composite*,” Method I; Procedure G. The fiber content by weight, F, was determined by pyrolysis. Three 3-inch-long samples of each bar type were accurately weighed (W_T) and heated at 600°C for five hours. The different components were weighed: sand (W_S) and basalt fiber (W_B) (if any). The fiber content by weight was then calculated according to the following equation:

$$\text{Fiber content by weight} = 100 \cdot W_F / (W_T - W_B - W_S) \quad (5.2)$$

c) Transverse and longitudinal coefficient of thermal expansion

The transverse and longitudinal coefficients of thermal expansion, alpha, were determined according to (ASTM E 831, 2012) - “*Linear Thermal Expansion of Solids Materials by Thermo-mechanical Analysis (TMA)*.” Samples, 5 to 10 mm thick, were placed under the probe and the measurements conducted between -30° and 60°C with a heating rate of 3°C/min. The measurements were conducted with a minimum of three samples.

d) Density

The density (ρ_{meas}) was determined by displacement in water according to (ASTM D 792, 2008) “*Density and Specific Gravity of Plastics by Displacement.*” standard. Three specimens were weighted in air and their weight is (P_S). Each specimen was placed in a cylinder and filled with water. The total weight of the cylinder including the sample and water (P_{S+W}) was measured. Afterwards, the specimen was removed from the cylinder, and then the cylinder was filled with water up to the same level. Thereafter, the weight of the cylinder containing only water was measured (P_W). Considering the density of water equal to 1.00, the density of the sample, ρ , was obtained by using the following equation:

$$\rho_{\text{meas}} = P_S / (P_S + P_W - P_{S+W}) \quad (5.3)$$

e) Void content

The void content was determined according to (ASTM D 2734, 2009) - “*Standard Test Methods for Void Content of Reinforced Plastics.*” The void content was calculated using Eqn. 5.4, whereas (ρ_{theor}) and (ρ_{meas}) are the theoretical density and measured density, respectively. The (ρ_{meas}) was measured in the Eqn. 5.3, while the (ρ_{theor}) was calculated from the density of the fibers, coating sand, if any, and resin matrix (including, fillers and additives). The fiber content (F) was calculated from Eqn. 5.2. Then the void content was calculated as follows:

$$V = 100 (\rho_{\text{theor}} - \rho_{\text{meas}}) / \rho_{\text{theor}} \quad (5.4)$$

$$\rho_{\text{theor}} = 100 / (R / \rho_R + F / \rho_F) \quad (5.5)$$

Where: R, ρ_R , F, and ρ_F are the weight in (%) of resin ($R = 100 - F$), the density of the resin, the weight in (%) of the fiber and the density of fiber, respectively. Since, the specific gravity values for the resin matrix and fiberglass were not available in order to determine ρ_{theor} . Therefore, the obtained values did not provide reliable values.

f) Water absorption

The water absorption (uptake) at saturation was determined according to the (ASTM D 570, 2010) – “*Water Absorption of Plastics.*” Five 4-inches long specimens of each diameter were cut, dried, and weighed. After 24 hours, the specimens were then immersed in water at 50°C during 3 weeks. The samples were periodically removed from water, surface dried and weighted. Because of the high surface porosity of the material, surface entrapped water could not be properly wiped using a paper towel. Only compressed air could adequately dry the surface. The specimens were considered as saturated when the weight became constant. The water content percentage in weight, W, was calculated as follows:

$$W = 100 \cdot (P_s - P_d) / P_d \quad (5.6)$$

where P_s and P_d are the weights of the bar in the saturated and dry state, respectively. The samples are then dried and weighted again to verify that no compound has been extracted.

g) Cure ratio

The cure ratio was obtained according to (ASTM D 5028, 1990) – “Curing Properties of Pultrusion Resin by Thermal Analysis.” The measurements were carried out on five specimens per test. The enthalpy of polymerization of the sample was measured by differential scanning calorimetry (DSC) and compared to the enthalpy of polymerization of the pure resin, taking into account the weight percentage of resin in the matrix. Thirty to fifty milligrams of sample were accurately weighted and placed in aluminum crucible. The samples were then heated from room temperature to 200°C at 20°C/min and the area of the peak of polymerization was calculated.

h) Glass transition temperature

The Glass transition temperature, T_g , was determined by (DSC) using (ASTM D 3418, 2012) – “*Transition Temperatures of Polymers by Thermal Analysis*”. Three specimens were used to measure the T_g . Thirty to forty milligrams of composite sample were weighed and placed in an aluminum pan. Then, the sample was heated up to 200°C under nitrogen at a heating rate of 20°C/min. The value of T_g was taken at the mid-height of the C_p jump.

i) Optical and Scanning Electronic Microscopy (SEM)

The porosity and defects, such as de-bonding between fibers and resin, poor fiber distribution, and micro-cracks, are major parameters affecting the long-term durability of the FRP product. Therefore, a scanning electronic microscopy (SEM) analysis was performed to examine the two components of the bars, resin matrix and fibers, and adhesion between them. Longitudinal and transverse sections of sample (one-inch-long) were cut and placed in cylindrical molds, where epoxy resin was cast. After 24 hours of curing at room temperature, the samples were removed and cut using a low speed saw equipped with a diamond blade. Thereafter, the specimens were polished using a polishing machine with three diamond pastes (15, 3, and 1 micron) before sputtering them with palladium. Thus, the specimens were ready for analysis with a Hitachi SEM.

5.2.2 Mechanical Properties

The mechanical characterization included testing of representative reference and conditioned specimens immersed in an alkaline solution at 60°C for different exposure time. The tests aimed at determining the transverse shear strength (ASTM D7617, 2011), interlaminar shear (ASTM D4475, 2008), tensile strength properties (CSA S806, 2012- Annex C), three-point flexural test (ASTM D4476, 2009), and bond strength (ACI 440, 2004, B.3 test method and CSA S806, 2012- Annex G) and comparing their residual shear and tensile strengths to those of the reference specimens.

a) Tensile strength test

All bars were tested according to the (CSA-S806, 2012), Annex C – “*Test Method for Tensile Properties of FRP Reinforcement*.” and (ACI-440.3R, 2004), Test Method B2 – “*Test Method for Longitudinal Tensile Properties of FRP Bars*.” Each specimen was cut into a certain length as specified in the (CSA-S806, 2012 and ACI 440-R3, 2004) and anchorage with steel tubes at each end. The free length (the length between the two steel anchorages) to bar-diameter ratio was

about $40d$ to avoid the anchorage slippage of the bars. The anchorage steel tubes were filled with cement-based grout. Each specimen was instrumented with a linear variable differential transformer (LVDT) with a gauge length of 200 mm to capture the elongation during testing. The test was carried out using a 2000-kN Baldwin testing machine and the load was increased until failure. The applied load and bar elongations were recorded using a data acquisition system monitored by a computer. The LVDT was removed from the bar at about 70% of the expected ultimate load to avoid damage. The test was continued until the specimen fractured and there was a sudden drop in the load. Only results in which failures occurred in the free-length of the specimen were considered as valid for the determination of the tensile strength. Figure 5.12 to Figure 5.18 show the procedures of preparing the test specimens and typical test setup. The tensile strength, f_u , of BFRP bars was calculated according to the following equation:

$$f_u = \frac{F_u}{A} \quad (5.7)$$

where:

f_u = tensile strength (MPa); F_u = tensile capacity (kN); and A = cross-sectional area of the test bar (mm^2).

While, the tensile modulus of elasticity, E_L , was calculated from the slope of the stress-strain curve at loads between 25 and 50% of the tensile capacity as follows:

$$E_L = \frac{F_1 - F_2}{(\varepsilon_1 - \varepsilon_2)A} \quad (5.8)$$

where:

E_L = Longitudinal modulus of elasticity (MPa); A = Cross-sectional area of the test bar (mm^2); F_1 and ε_1 = Load and corresponding strain at approximately 50% of the ultimate tensile capacity; and F_2 and ε_2 = Load and corresponding strain at approximately 25% of the ultimate tensile capacity, (kN and dimensionless, respectively).



Figure 5.12: Fabrication of plastic rings



Figure 5.13: Installing the plastic rings

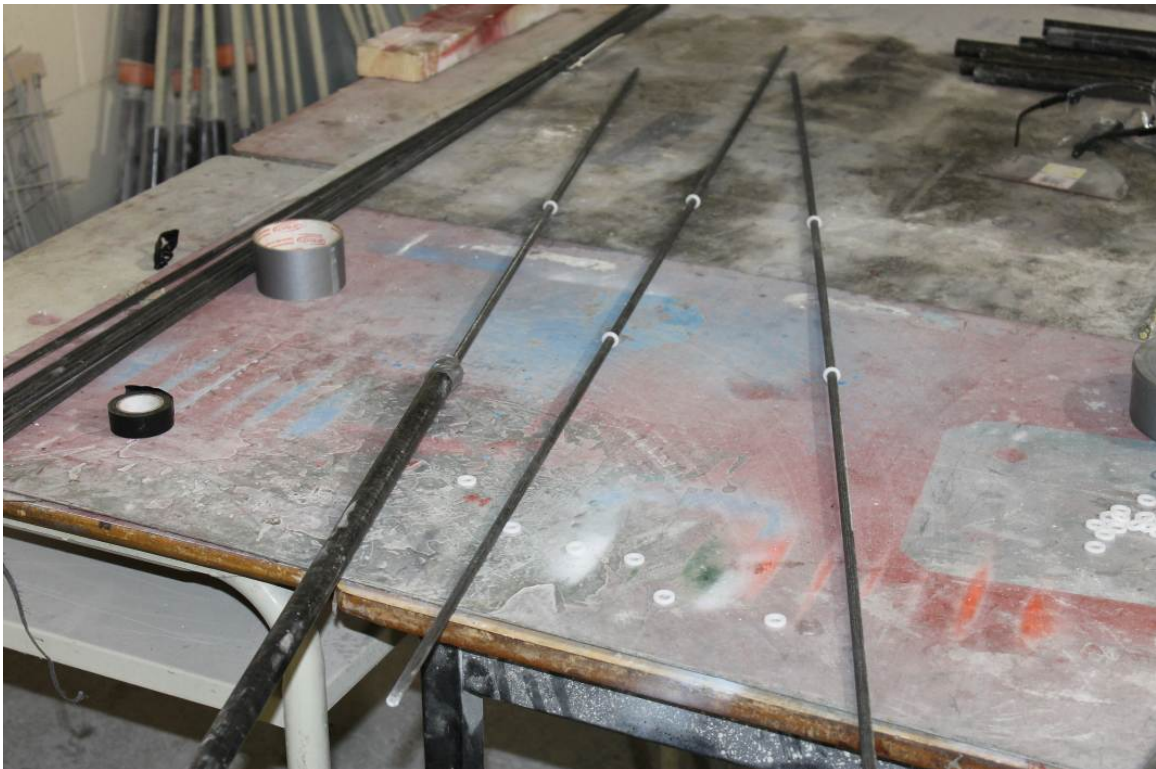


Figure 5.14: Installing the steel tubes

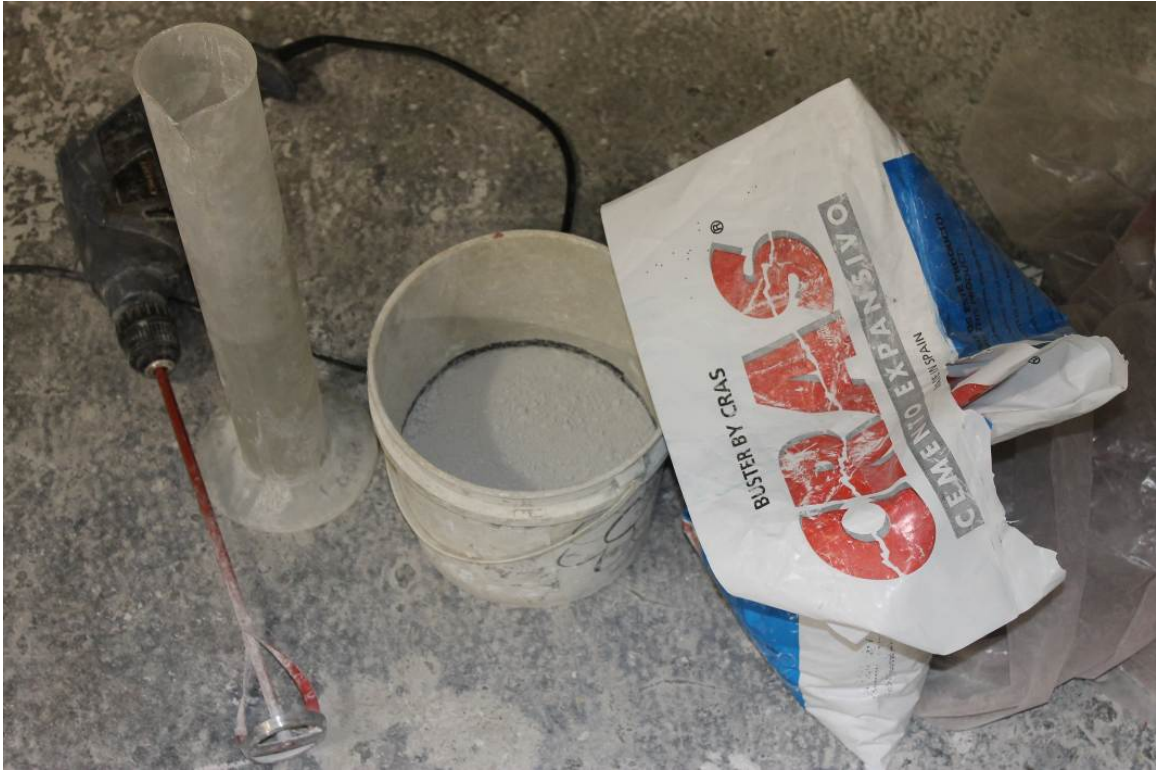


Figure 5.15: Preparing the cement grout



Figure 5.16: Steel tubes after installation



Figure 5.17: Filling the anchorage with cement grout



Figure 5.18: Tensile strength test setup

b) Interlaminar shear test

The interlaminar shear strength test (*short-beam test*) was conducted according to (ASTM D4475, 2008) standard “*Standard Test Method for Apparent Horizontal Shear Strength of Pultruded Reinforced Plastic Rods by The Short-Beam Method*” with a minimum of eight specimens each diameter. This test was conducted to obtain the interlaminar shear strength, which is governed by the fiber–matrix interface. The tests were carried out using a 500-kN MTS 810 testing machine. The specimens were cut to a certain length according to the specification while the distance between supports in the test setup was set to four times the nominal diameter of each BFRP

bar. A displacement-controlled rate of 1.3 mm/min was employed during the test. The applied load was recorded with a computer-monitored data acquisition system. Figure 5.19 and Figure 5.20 show photos of the test setup and test specimen during testing. The interlaminar shear strength, S , of BFRP bars was calculated from Eqn. 5.9 as follows:

$$S = 0.849 \frac{P}{d^2} \quad (5.9)$$

where:

S = interlaminar shear strength (MPa); P = shear failure load (N); and d = bar diameter (mm).

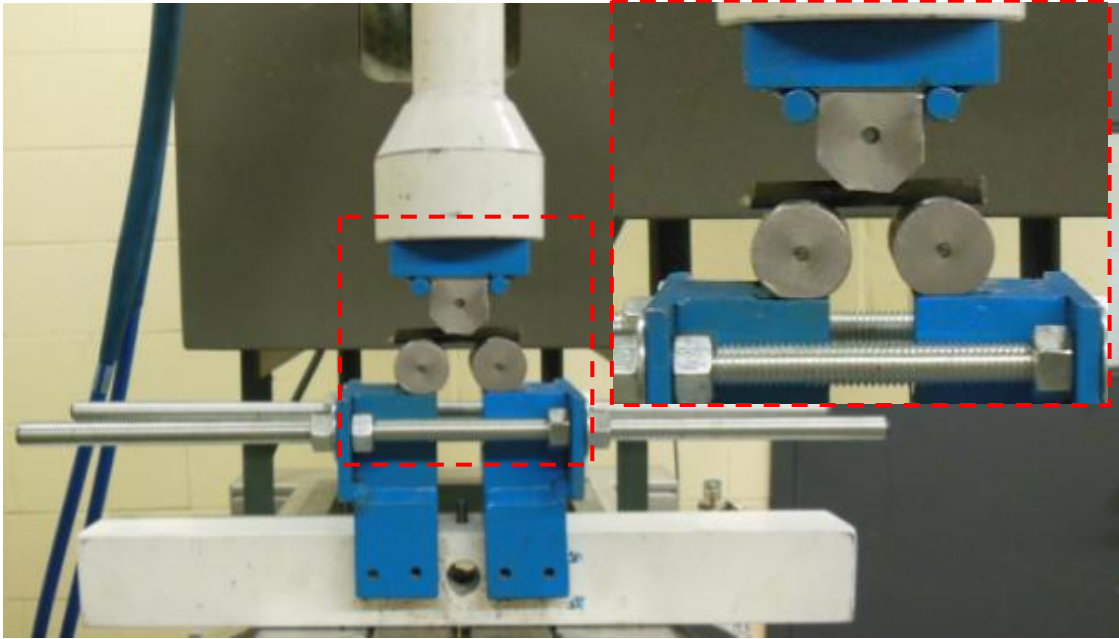


Figure 5.19: Interlaminar shear test setup

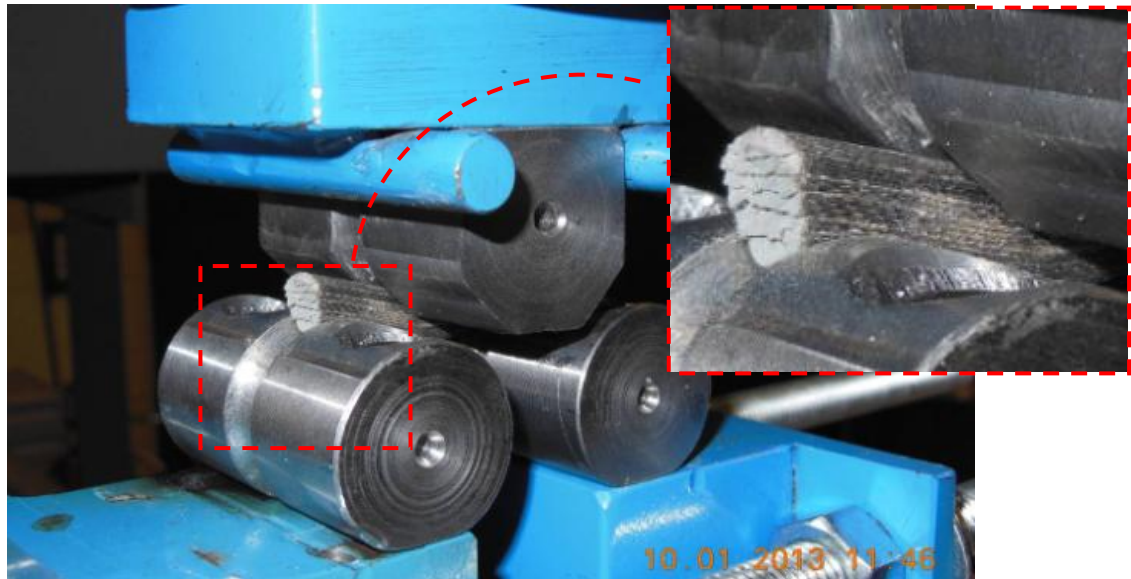


Figure 5.20: Test specimen during testing

c) Transverse shear test

Eight specimens of each diameter 200-mm in length were tested according to (CSA S806, 2012), Annex L – “*Test Method for Shear Properties of FRP Rods.*” using a MTS 810 testing machine equipped with a 500-kN load cell. A displacement-controlled rate of 1.5 mm/min was used during the test, which yielded between 30 and 60 MPa/min until specimen failure. The applied load was recorded during the test using a data acquisition system monitored by a computer. Figure 5.21 shows the transverse shear test setup. The transverse shear strength, τ , of BFRP bars was calculated according to the following equation:

$$\tau = \frac{P}{2A} \quad (5.10)$$

where:

τ = transverse shear strength (MPa); P = shear failure load (N); and A = cross-sectional area of the test bar (mm^2).



(a) Elevation



(b) Side view

Figure 5.21: Transverse shear test setup

d) Three-point flexural test

Flexural testing of BFRP bars was conducted according to the three-point flexural test method and it was consistent with (ASTM D4476, 2009) “*Standard Test Method for Flexural Properties of Fiber Reinforced Pultruded Plastic Rods*.” The tests were performed on approximately 300-mm-long specimens over a simply supported span equal to 16 times the bar diameter. At least eight specimens of each type and diameter were tested under laboratory conditions on a MTS 810 testing machine equipped with a 500-kN load cell. The specimens were tested under a displacement-controlled rate of 1.5 mm/min, which gave between 30 and 60 MPa/min until specimen failure. Figure 5.22 shows the flexural test setup.

The maximum flexure stress, S , of BFRP bars was calculated according to the following equation:

$$S = \frac{P.L.C}{4.I} \quad (5.11)$$

where:

S = Maximum flexure stress in the outer fibers at mid-span (N/mm²);

P = Maximum load at the load-deflection curve (N);

L = Support span (mm);

I = Moment of inertia (mm⁴); and

C = Distance from centroid to extremities (mm).

The modulus of elasticity (E_b) is the ratio, within elastic limit, of stress to corresponding strain. It was calculated according to the following equation:

$$E_b = \frac{P.L^3}{48.I.Y} \quad (5.12)$$

where:

E_b = Modulus of elasticity in bending (N/mm²);

P = Load at given point on the load-deflection curve (N);

L = Support span (mm).

I = Moment of inertia (mm⁴); and

Y = Mid-span deflection at load P (mm).

The maximum strain in the outer fiber at mid span (ε) was calculated according to the following equation:

$$\varepsilon = \frac{12.C.Y}{L^2} \quad (5.13)$$

where:

ε = Maximum strain in the outer fibers at mid-span (mm/mm);

Y = The maximum deflection at load chosen (mm);

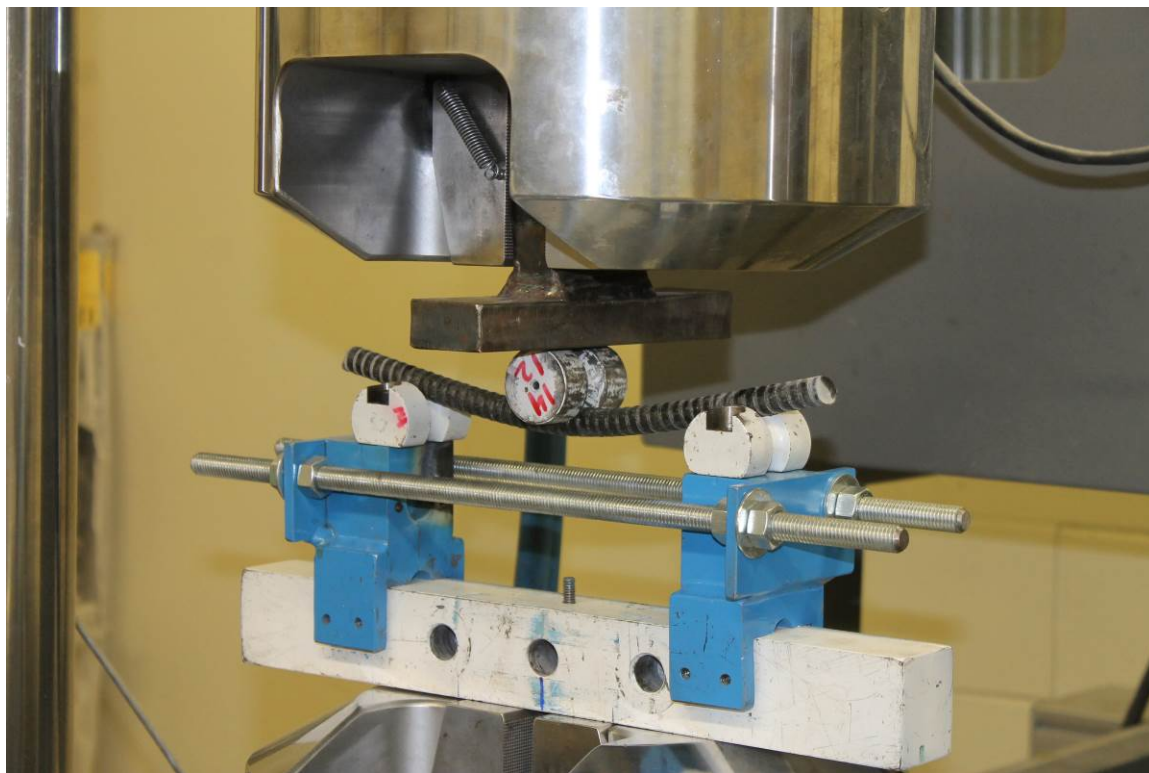
L = Support span (m).

C = Distance from centroid to extremities (mm); and

Y = Mid-span deflection at load P (mm).



(a) Elevation



(b) Side view

Figure 5.22: Flexural test setup

e) Pullout bond test

The pullout tests were carried out according to (ACI 440, 2004-B.3 Test Method- “*Test Method for Bond Strength of FRP Bars by Pullout Testing*”) and (CSA S806, 2012- Annex G-“ *Test Method for Bond Strength of FRP Rods by Pullout Testing*”). A minimum of five specimens were tested under pullout bond. Each block had a section of 200 x 200 x 200 mm, which was cast using compressive strength of 35 MPa after 28 days. The BFRP bar was cut into 1200-mm-long. One end of the bar sample was completely embedded in the concrete block and the other was attached with steel anchorage tube with a length of 400-mm. The embedment length (bond length), namely $5d_b$, was used. The specimens were tested on the BALDWIN machine with a capacity of 2000 kN under a load controlled rate of about 12 to 15 kN per minute until the pullout failure. The slip of the free-end of the BFRP bar was measured using a (LVDT). Figure 5.23 to Figure 5.27 show the test specimens fabrication procedures, instrumentations, and test setup. The average bond stress is calculated using the following formula:

$$u = \frac{F_u}{\pi \times d_b \times l_b} \quad (5.14)$$

where:

u is the average bond strength (MPa);

F_u is the failure load (N);

d_b is the bar diameter (mm); and

l_b is the bonded length (embedment length).

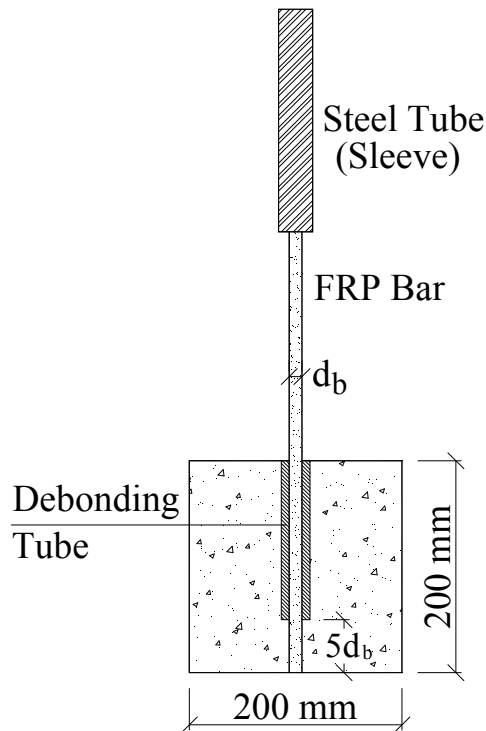


Figure 5.23: Pullout specimens details



Figure 5.24: BFRP bars inside the block formwork



Figure 5.25: Complete formwork



Figure 5.26: Pullout test specimens

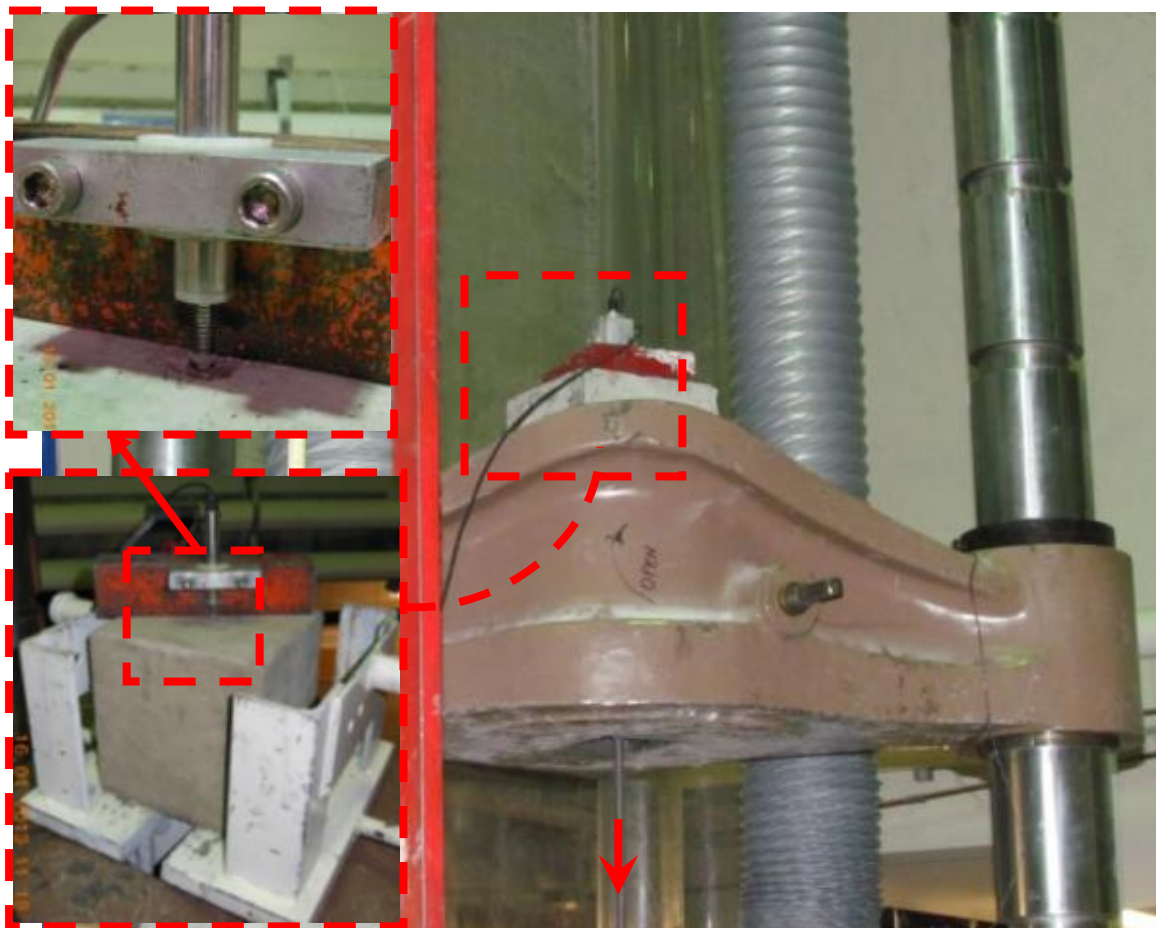


Figure 5.27: Pullout test setup

5.2.3 Test Specimens Conditioning

The long-term performances of the BFRP bars were assessed under harsh alkaline exposure simulating the real life situation. The alkaline exposure was achieved by immersing the BFRP bars in tap water with an alkaline solution at different elevated temperatures to accelerate the effects. Thereafter, the properties were assessed and compared with the unconditioned reference values. The conditioning was conducted according to the (CSA-S806, 2012), Annex O-test method “*Test Method for Alkali Resistance of FRP Rods*” for conditioning in alkaline solution. The alkaline solution compositions were 118.5 g Ca(OH)_2 , 4.2 g KOH and 0.9 g NaOH per liter (pH=13). The embedded BFRP bars were completely immersed in big steel containers specially fabricated for this study. A polyethylene sheet was placed on the top of the container to avoid the evaporation of water during the conditioning. Furthermore, the water level was kept constant throughout the study to avoid the pH, which could be due to concentration of alkaline ions in the solution. Different BFRP specimens of different types and diameters were set at different environmental temperatures (22, 40, 50, and 60°C) under different exposure times ranged between 720 to 2160 hrs. Figure 5.28 shows the BFRP specimens in conditioning.





Figure 5.28: BFRP bars in conditioning at different environmental chambers

5.3 Test Results and Discussion

5.3.1 Physical Properties

a) Cross-sectional area

Table 5.2 to Table 5.4 and Table 5.5 provide the cross-sectional area of the tested BFRP bars (Type A, B, C, D, E, F, G, H), and TECH 12-mm and prestressing BFRP 7-mm and BASA 8-mm (code 43 and 51), respectively.

b) Fiber content

The fiber content in percent by weight for the BFRP bars are listed in Table 5.2 to Table 5.5. As shown from these tables that the fiber content by weight for the BFRP bars types A, B, C, D, E, F, G, H and TECH 12-mm were ranged between 72 to 90.6% (an average of five specimens). The lesser fiber content by weight value, however, depicted in the BFRP type A (6-mm in diameter) which contains less fiber (72%). On the other hand, the prestressing BFRP 7-mm in diameter and BASA 8-mm code 43 and 51 showed an average of 85.1, 77.4, and 81.1, respectively. Hence, the test results were higher than and meet the minimum requirement as specified in the (CSA S807, 2010) and (ACI 440, 2008) (70% fiber content by weight or 55% as fiber content by volume, respectively). Table 5.6 shows the specified limits for physical properties according to the (CSA-S807, 2010) and (ACI 440.6M, 2008).

c) Transverse and longitudinal coefficient of thermal expansion

Table 5.2 to Table 5.5 reported that the transverse coefficient of thermal expansion (CTE) of the BFRP bars of type A to H were ranged from $15.7 \cdot 10^{-6}$ to $25.8 \cdot 10^{-6} \text{ }^{\circ}\text{C}^{-1}$, while the longitudinal

coefficient of thermal expansion were ranged from $5.5.10^{-6}$ to $11.1.10^{-6} \text{ }^{\circ}\text{C}^{-1}$. Besides, the transverse (CTE) of TECH 12-mm, prestressing BFRP 7-mm, and BASA 8-mm code 43 and 51 were $35.6.10^{-6}$, $18.7.10^{-6}$, $26.8.10^{-6}$, and $18.4.10^{-6} \text{ }^{\circ}\text{C}^{-1}$, respectively. The test results indicated that the transverse (CTE) for all the BFRP bars was higher than the specified limit according to the (CSA S807, 2010) which is $40.0.10^{-6} \text{ }^{\circ}\text{C}^{-1}$.

d) Density

Table 5.2 to Table 5.4 and Table 5.5 provide the specific gravity of the tested BFRP bars (Type A, B, C, D, E, F, G, H), and TECH 12-mm and prestressing 7-mm and BASA 8-mm (code 43 and 51), respectively.

e) Water absorption

For the BFRP bars types (A, B, C, D, E, F, G, and H), the test results showed that the water absorption at saturation were ranged between 0.15 to and 1.66% on average of five specimens each diameter while the TECH 12-mm, prestressing 7-mm, and BASA-8-mm code 43 and 51 were 0.27, 0.15, 0.56, and 0.62, respectively. The reported results in Table 5.2 to Table 5.5 indicated that the Water/Moisture absorption at equilibrium (%) for all BFRP bars were satisfied the minimum requirement as specified in the (CSA-S807, 2010) and (ACI 440.6M, 2008) of 0.75% and 1%, respectively, except for the BFRP type B, D, and E which were 0.77, 1.66 and 0.95%, respectively.

f) Cure ratio

A typical value of (218 J/g) for vinyl-ester system was used to estimate the cure ratios for all BFRP bars types. The average cure ratios are reported in Table 5.2 to Table 5.5. As shown from the test results that all BFRP bars were meet the specified limit of the (CSA-S807, 2008) (95%), except the TECH BFRP bar 12-mm in diameter, which is below that limit.

g) Glass transition temperature

The T_g is an important physical property of the matrix and it is not only an indicator of the thermal stability of the material but also an important indicator of the structure of the polymer and its mechanical properties. The BFRP bars type A to H evidenced a T_g ranging from 91.8 to 135.6°C while the T_g of the BFRP TECH-BFRP 12-mm, prestressing BFRP 7-mm, BASA BFRP 8 mm code 43 and 51 were 68.4, 102.2, 117.4, and 126.6 °C, respectively. Table 5.2 to Table 5.5 show the T_g test results while Table 5.6 presents the specified limits of (ACI 440, 2008) and (CSA S807, 2010) (100°C). The comparison between the test results and the specified limits in (ACI 440, 2008) and (CSA S807, 2010) confirms that all BFRP bars tested herein meet the requirements of the specifications, except for the BFRP bar type E and TECH BFRP 12-mm, which showed T_g 91.8 and 68.4°C, respectively. Figure 5.29, Figure 5.30, and Figure 5.31 show photos for some bars after pyrolysis such as, TECH BFRP 12-mm, prestressing BFRP 7-mm, and BASA BFRP bar 8-mm (code 43 and 51), respectively. Furthermore, typical DSC curves of some BFRP samples of type A and H, TECH BFRP 12-mm, prestressing BFRP 7-mm, and BASA BFRP bar 8-mm are shown in Figure 5.32 to Figure 5.35. The DSC curve for TECH BFRP bar 12-mm and prestressing BFRP 7-mm showed a post-curing phenomenon (top) while the second curve obtained after the first heating run and showed no post-curing and a higher T_g .



Figure 5.29: TECH BFRP 12-mm samples after pyrolysis



Figure 5.30: Prestressing BFRP 7-mm in diameter after pyrolysis

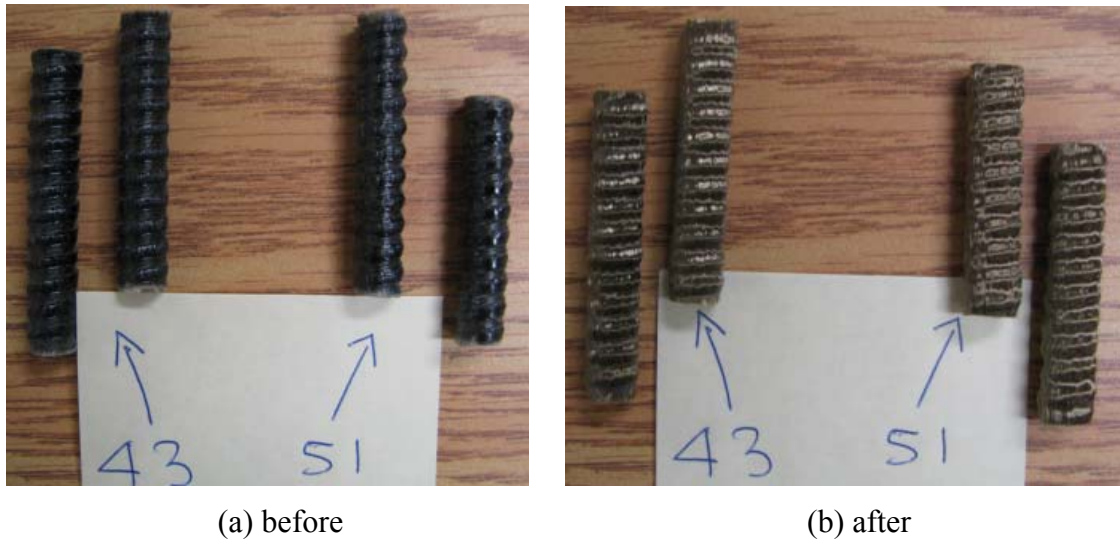


Figure 5.31: BASA BFRP 8-mm (code 43 and 51) samples before and after pyrolysis

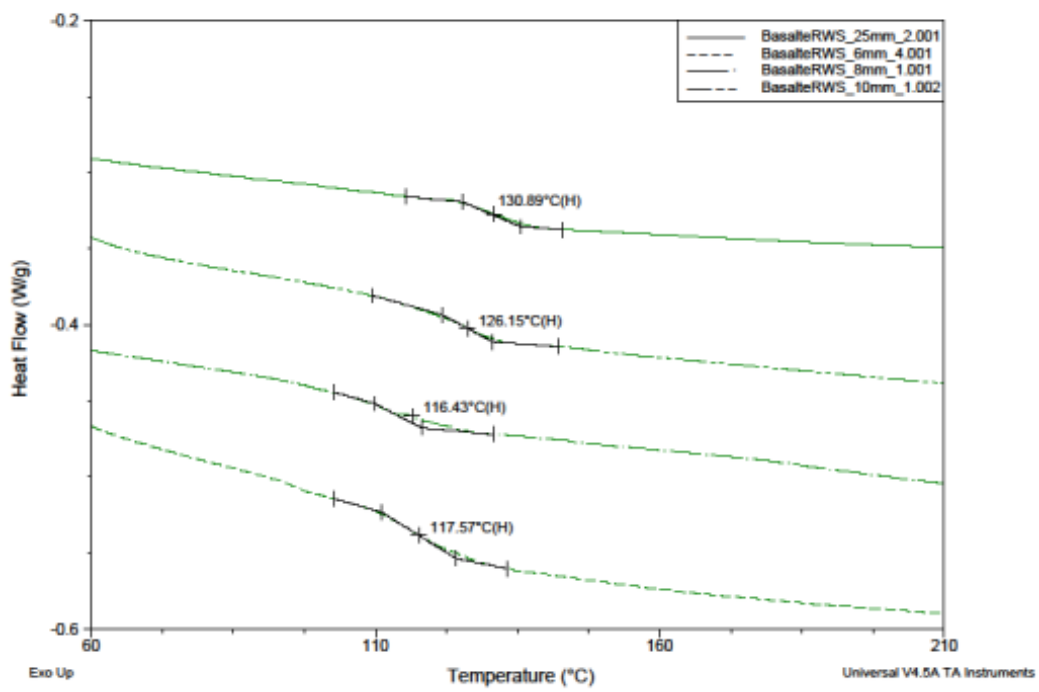


Figure 5.32: Typical DSC curves of BFRP bar samples (Type A)

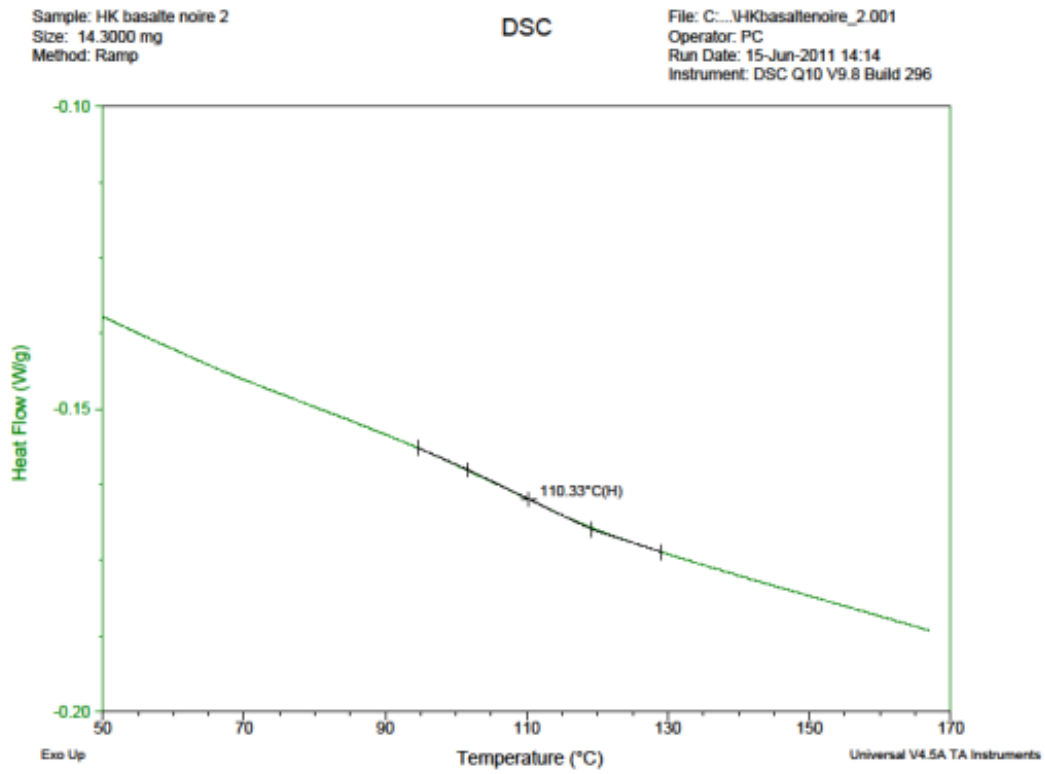


Figure 5.33: Typical DSC curves of BFRP bar samples (Type H)

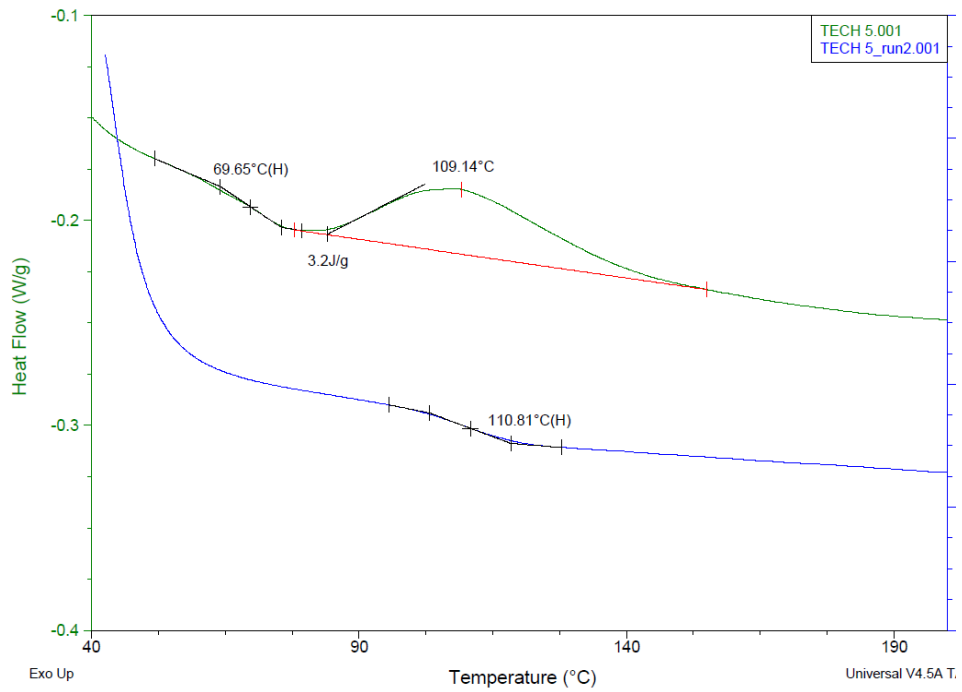


Figure 5.34: Typical DSC curve of TECH BFRP 12-mm in diameter

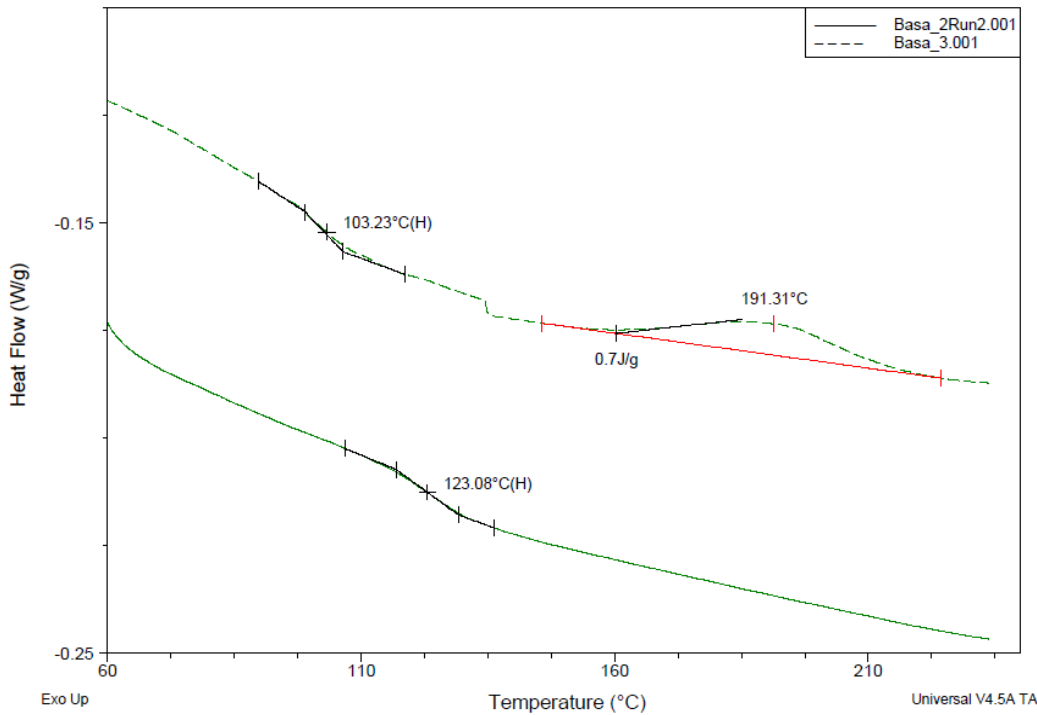


Figure 5.35: Typical DSC curve of prestressing BFRP 7-mm

h) Optical and scanning electronic microscopy (SEM)

A scanning electronic microscopy (SEM) analysis was performed to observe the microstructure of the specimens. Figure 5.36 to Figure 5.142 present the SEM micrographs for all BFRP examined herein. Figure 5.36 to Figure 5.45 show SEM micrographs for transverse and longitudinal sections of some BFRP bar samples of Type A (8, 10, 13, and 16 mm-in diameters). These figures showed that all samples were the same microstructure, regardless their diameter. Besides, it can be observed the presence of many small voids. This issue, however, is often encountered with materials pultruded using open furnace molding process. Air bubbles entrapped between fibers were stretched, giving characteristic tubular pores (Figure 5.38 to Figure 5.41) while the interface between the fiber, and the resin was appropriate since no debonding gaps were appeared (Figure 5.43 to Figure 5.45).

SEM micrographs of cross and longitudinal sections of BFRP bars Type B to F bars are presented in Figure 5.46 to Figure 5.61. Figure 5.46 to Figure 5.49 present the cross-sectional and longitudinal section at low and high magnification for BFRP bar B. It can be observed that the pores were visible between fibers and in rich areas of resin (Figure 5.47 and Figure 5.48). However, the interface was appropriate in spite of the presence of few delaminations, which may be due to a poor wetting (Figure 5.49). Moreover, Figure 5.50 to Figure 5.58 show micrographs of BFRP bars of type C, D, and E. The same observations can be noticed for these three bars: presence of tubular voids and good adhesion of the fibers to the resin matrix. However, a serious debonding of the coating has detected in bar type D (Figure 5.54), which could be an entry for corrosive solution from aggressive environments. Figure 5.59 to Figure 5.61 show cross and longitudinal sections of bar type F. As shown in figures that large pores between fiber bundles were observed.

This phenomenon occurs when the bundles were not sufficiently impregnated and pressed together during the manufacturing process.

The transverse and longitudinal sections SEM micrographs of BFRP bars Type G (No. 3, No. 4, and No. 5) are presented in Figure 5.62 to Figure 5.69. As shown from these figures that the three samples of different diameter were very similar while the micrographs showed the presence of pores having a tubular shape. These pores were more concentrated in the outer area of the bars. Besides, no significant defects, such as microcracking or debonding at the interface between the fibers and resin matrix were appeared. SEM micrographs of transverse sections of the tested BFRP samples of Type H are presented in Figure 5.70 to Figure 5.74. As shown in Figure 5.70 and Figure 5.71 that some tubular pores, having a size of several tenths microns, presented in the FRP material. However, the level of porosity is not very high. Figure 5.72 presents a close-up view of the cross-sectional of basalt fibers embedded in the resin matrix. A small black gap, corresponding to debonding can be observed around some basalt fibers. This debonding was clearly shown in Figure 5.73 (basalt fibers are totally debonded from the matrix), and in Figure 5.74 (where a partial debonding of the basalt fibers can be seen).

Figure 5.75 to Figure 5.81 show micrographs of cross and longitudinal sections for TECH BFRP bar 12 mm at low and high magnifications. Figure 5.75 presents a general view of the cross-section of the bar, where pores and microcracks were visible. The sample characterized by the presence of fine (Figure 5.76) and large microcracks (Figure 5.77). The analysis of a longitudinal section also showed an uneven distribution and poor orientation of the fibers (Figure 5.78). However, micrographs of the cross and longitudinal sections of the sample obtained at high magnification show that the interface between the fibers and resin was appropriate since almost no free gaps were detected around the fibers (Figure 5.79 to Figure 5.81).

Figure 5.82 to Figure 5.87 show the cross and longitudinal sections micrographs of the prestressing BFRP 7-mm at low and high magnifications. Figure 5.82 shows a typical general view of the cross-sectional of the material. It can be observed in the figure that no pores were detected. Figure 5.83 shows the longitudinal section in a region where fibers were not well oriented. However, this phenomenon did not observe on a large-scale which did not affect the properties of the bar. The fiber diameter was not constant but their distribution was appropriate as seen in Figure 5.84. Figure 5.85 to Figure 5.87 show the cross and longitudinal sections at high magnification and the interface between the fibers and resin matrix, which is an important parameter for the mechanical and durability properties of the material. The bonding was appropriate (Figure 5.86). However, free gaps may be observed around some fibers (Figure 5.85 and Figure 5.87), which should not be an issue since their number was limited. Figure 5.88 to Figure 5.90 present micrographs of the fracture surface of a short beam sample. It can be seen from the figures that there was not much residue resin covers the fiber surface, which led the rupture of the sample mainly occurred at the interface. Hence, the strength of the resin fiber bonding was then generally weaker than the resin strength.

Furthermore, SEM micrographs analysis was conducted for prestressing BFRP bars 7-mm immersed in alkaline solution at 60°C for 1,000 hours. After this period, the bars were removed, washed, and dried. While the surface and cross and longitudinal sections were cut and prepared for (SEM) analysis. Figure 5.91 to Figure 5.93 show micrographs at several magnifications of the surface of prestressing BFRP bars 7-mm sample before conditioning (reference) while Figure 5.94 to Figure 5.96 depict the sample surface after conditioning. As shown from these figures

that the surface of the reference sample was covered by a woven fabric. However, after conditioning this fabric was disappeared, leaving the fingerprint of the fibers in the surrounding resin. However, no further degradation was observed and it was very fine layer of fabric. Besides, the resin did not present significant signs of chemical degradation. In addition, Figure 5.97 to Figure 5.99 and Figure 5.100 to Figure 5.102 present micrographs at several magnifications of the cross-section of the sample before and after conditioning, respectively. It can be seen from these figures that the fibers, the resin, and the interface remain unchanged after conditioning. Besides, no additional debonding or other sign of degradation was detected. Figure 5.103 to Figure 5.105 and Figure 5.106 to Figure 5.107 present micrographs at several magnifications of the longitudinal section of the sample before and after conditioning, respectively. It was evidenced from the figures that no corrosion caused by the immersion in alkaline solution.

Optical and scanning electronic microscopy analyses for BASA BFRP bars 8-mm (code 43 and 51) were performed on unconditioned and conditioned specimens for exposure time of 1 month (720 hrs.) in alkaline solution at 22° and 60°C. Figure 5.108 to Figure 5.110 and Figure 5.111 to Figure 5.113 depict optical micrographs of BASA BFRP bars 8-mm (code 43 and 51) at different magnifications, respectively. It can be seen from these figures that some pores were visible, especially in the coating. Besides, it should be noted that the micrographs did not observe any changes after 1 month conditioning at 22° and 60°C and were similar to the reference specimens. Figure 5.114 and Figure 5.115 show general views of the cross and longitudinal sections of BASA BFRP bars 8-mm (code 43) before conditioning. It can be seen that few tubular pores were visible, mainly in the outer section of the bar. However, the fiber distribution was adequate despite the presence of some small resin-rich zones. At high magnification, a certain debonding was observed between the fibers and resin matrix (Figure 5.116 and Figure 5.117), which is common in fiber composites. This is due to an inappropriate sizing agent at the surface of the fiber, manufacturing process or resin shrinkage. Resin shrinkage was similar to concrete shrinkage whereas curing, resin density increases, and free gaps may appear at the surface, which may also induce microcracking. Figure 5.118 shows the cross-section of BASA BFRP bars 8-mm (code 43) after one month conditioning at 22°C. As shown in the figure that white circles around the fibers indicate the presence of a free space at the interface with the resin matrix as observed in the next micrographs obtained at high magnification for the cross and longitudinal sections of the material (Figure 5.119 and Figure 5.120). While the same conclusions can be drawn for the conditioned sample at 60°C (Figure 5.121 and Figure 5.122).

Figure 5.123 and Figure 5.126 show micrographs of the cross and longitudinal sections of reference BASA BFRP bars 8-mm (code 51). BASA BFRP bars 8-mm (code 51) showed similar observations as BASA BFRP bars 8-mm (code 43). However, presence of porosities may be more than in BASA BFRP bars 8-mm (code 43), mainly concentrated in the coating. This is due to air bubble entrapment and fiber debonding at the interface with the resin. Figure 5.127 to Figure 5.129 show typical micrographs of BASA BFRP bars 8-mm (code 51) after 1 month conditioning at 22°C. As shown in these figures that no significant changes were observed with respect to the reference material while the conclusions were the same compared with BASA BFRP bars 8-mm (code 43). However, it seems that the debonding was slightly more pronounced with BASA BFRP bars 8-mm (code 51) than with BASA BFRP bars 8-mm (code 43). The free space appearing at the fiber surface seems generally larger with BASA BFRP bars 8-mm (code 51). The performance of this bar could therefore be slightly lower than for BASA BFRP bars 8-mm (code 43). In addition, increasing of the conditioning temperature to 60°C did not cause any additional

visible changes in the microstructure of BASA BFRP bars 8-mm (code 51) as shown in Figure 5.130 to Figure 5.132.

Further investigations were performed on the BASA BFRP bars 8-mm (code 43 and 51) under conditioning in water or alkaline solution for exposure time of 1,000 hrs., 3,000 hrs., and three months at different temperatures (RT, 50° or 60°C). Several phenomena may induce a reduction of physical properties in which depends on the nature of the components of the material, conditioning may corrode the resin through an alkaline attack (hydrolysis) or the fibers, or affects the bonding of fibers to the resin matrix. Vinylester resin and basalt fibers were fairly resistant to chemicals and properties at the interface between fibers and resin were crucial since fiber debonding was often the cause of degradation of FRP materials. Therefore, more investigations on the microstructural were studied. The following paragraphs presents the test results of (SEM), (T_g) and Fourier transform infrared spectroscopy (FTIR) of the BASA BFRP bars 8-mm (code 43 and 51). Table 5.8 lists the analyses performed on the different conditioned specimens. Typical micrographs obtained at high magnification are presented below. Observations made on the longitudinal sections show that the basalt fiber surface did not affected by the conditioning, considering their fairly good chemical resistance.

For BASA BFRP bars 8-mm (code 43), Sample 1 (1,000 h- Alkaline - 60°C), it can be observed that the sample presented a large crack in its center, which should affect its mechanical properties (Figure 5.133). At high magnification, it was observed that basalt fibers were not bonded to the resin matrix (Figure 5.134). While Sample 2 (3,000 h- Alkaline - 60°C) was similar as previous sample, which contains significant defects and also may affect its performance (Figure 5.135) while debonding was observed at the interface (Figure 5.136). Sample 3 (3,000 h- Water - 60°C) did not depict any major defect and the degree of debonding was less than the two previous samples (Figure 5.137). While Sample 4 (3 months- Alkaline - 50°C) showed less debonding than the conditioned sample at 60°C (Figure 5.138).

For BASA BFRP bars 8-mm (code 51), Sample 5 (1,000 h- Alkaline - 60°C), did not show significant debonding was visible in the sample (Figure 5.139) while Sample 6 (3,000 h- Alkaline - 60°C) observed a significant debonding (Figure 5.140) and Sample 7 (3,000 h- Water - 60°C) was well bonded (Figure 5.141). However, Sample 8 (3 month- Alkaline - 50°C) depicted fibers partly debonded (Figure 5.142). The T_g of samples of BASA BFRP bars 8-mm (code 43 and 51) conditioned in alkaline solution during 3 months at 50°C and 3,000 hours at 60°C was measured by DSC. The values of the T_g of the first and second run and of the references are presented in Table 5.137. The T_g values of conditioned samples were reduced after conditioning (run 1) compared to BASA BFRP bars 8-mm (code 43 and 51) reference samples. However, once heated, these values increase up to a value slightly higher than the reference, which shows that the T_g decrease was reversible and consequently not caused by any chemical degradation of the resin. It can be explained by a plasticizing effect caused by water absorption. An example of thermograms is shown in Figure 5.143.

FTIR spectra presented in Figure 5.144 to Figure 5.147, which was obtained from reference samples of BASA BFRP bars 8-mm (code 43 and 51) and conditioned ones in alkaline solution at 60C during 3,000 hrs. Spectra of reference and conditioned spectra were very similar. A very small increase of the peak related to hydroxyl groups (-OH) around 3400 cm^{-1} was observed.

This could be due to a slight chemical degradation of the resin matrix through a hydrolysis reaction or more probably presence of alkalis or water from the conditioning solution.

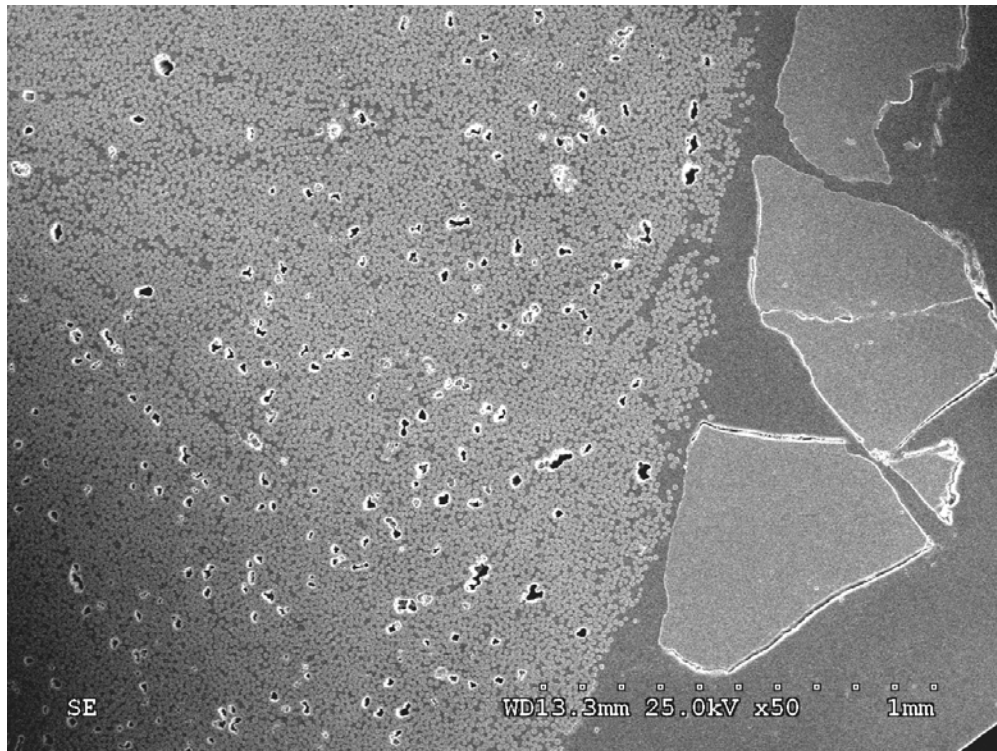


Figure 5.36: General view of the outer part of BFRP bar 10 mm (Type A)

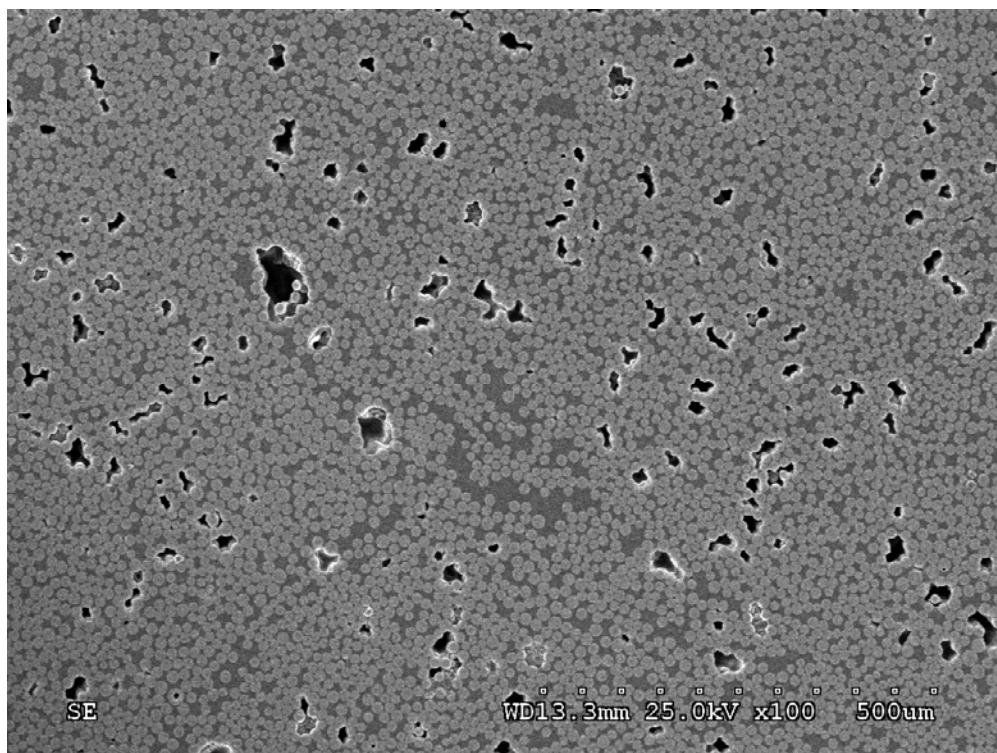


Figure 5.37: General view of the inner part of BFRP bar 16 mm (Type A)

Table 5.2: Physical properties of BFRP bars (Type A) in mm²

Property	BFRP bar (type A)					
	6 mm		10 mm	13 mm	16 mm	25 mm
Cross-sectional area (mm²)	41.0 ± 0.46	54.8 ± 0.50	84.8 ± 0.49	153.8 ± 0.86	224.3 ± 0.22	536.6 ± 1.71
Fiber content by weight (%)	72.0 ± 0.07	78.9 ± 0.12	80.7 ± 0.11	79.2 ± 0.21	80.0 ± 0.17	80.3 ± 0.04
Transverse CET (×10⁻⁶ °C⁻¹)	25.2 ± 3.61	19.4 ± 0.45	22.8 ± 1.20	22.0 ± 0.21	23.8 ± 1.45	21.6 ± 0.25
Longitudinal CTE (×10⁻⁶ °C⁻¹)	11.1 ± 0.25	6.6 ± 0.50	7.5 ± 1.55	6.7 ± 0.46	6.3 ± 0.40	9.4 ± 0.56
Density	2.0 ± 0.03	2.0 ± 0.03	2.1 ± 0.02	2.0 ± 0.02	2.1 ± 0.01	2.1 ± 0.01
Water absorption after 24 h (%)	0.07 ± 0.01	0.09 ± 0.01	0.06 ± 0.02	0.08 ± 0.01	0.05 ± 0.01	0.04 ± 0.01
Water/Moisture absorption at equilibrium (%)	0.27 ± 0.01	0.28 ± 0.02	0.18 ± 0.02	0.35 ± 0.04	0.17 ± 0.01	0.16 ± 0.01
Cure ratio (%)	> 99	> 99	> 99	> 99	> 99	> 99
T_g (°C)	120.2 ± 4.60	111.0 ± 1.58	121.2 ± 3.03	115.0 ± 3.81	127.0 ± 5.34	135.6 ± 3.91

Table 5.3: Physical properties and cross-sectional area of BFRP bars (Type B to F) in mm²

Property	BFRP type				
	B	C	D	E	F
Cross-sectional area (mm²)	97.2 ± 0.84	118.6 ± 3.65	122.2 ± 1.10	88.8 ± 0.84	116.8 ± 1.10
Fiber content by weight (%)	74.7 ± 0.36	79.7 ± 0.21	78.7 ± 0.06	82.4 ± 0.25	81.0 ± 3.91
Transverse CET (×10⁻⁶ °C⁻¹)	23.5 ± 0.46	20.1 ± 1.01	19.5 ± 1.01	18.6 ± 1.20	16.3 ± 0.53
Longitudinal CTE (×10⁻⁶ °C⁻¹)	7.1 ± 1.81	5.9 ± 0.72	6.1 ± 0.71	5.5 ± 0.66	8.5 ± 1.82
Density	1.9 ± 0.01	2.1 ± 0.01	1.9 ± 0.01	2.0 ± 0.01	2.0 ± 0.01
Water absorption after 24 h (%)	0.20 ± 0.02	0.09 ± 0.01	0.32 ± 0.02	0.35 ± 0.04	0.32 ± 0.03
Water/Moisture absorption at equilibrium (%)	0.76 ± 0.03	0.20 ± 0.01	1.66 ± 0.20	0.95 ± 0.11	1.35 ± 0.12
Cure ratio (%)	100.0 ± 0.00	100.0 ± 0.00	99.3 ± 0.27	99.7 ± 0.27	98.5 ± 0.35
T_g (°C)	118.2 ± 5.07	119.4 ± 6.07	108.8 ± 4.02	91.8 ± 1.30	108.6 ± 3.85

Table 5.4: Physical properties of BFRP bars (Type G and H) and TECH 12-mm

Property	BFRP bar (type G)			BFRP bar (type H)	TECH 12-mm
	No. 3	No. 4	No. 5	No. 4	No.4
Cross-sectional area (mm ²)	77.2 ± 0.45	110.8 ± 0.84	220.4 ± 0.89	140.6 ± 0.55	108.2 ± 0.84
Fiber content by weight (%)	87.2 ± 0.84	90.6 ± 0.84	89.9 ± 0.84	84.0 ± 0.00	73.3 ± 0.10
Transverse CET (×10 ⁻⁶ °C ⁻¹)	22.2 ± 0.71	23.0 ± 0.51	25.8 ± 1.10	15.7 ± 0.64	35.6 ± 3.31
Longitudinal CTE (×10 ⁻⁶ °C ⁻¹)	-	-	-	6.6 ± 0.64	-
Density	2.2 ± 0.01	2.1 ± 0.01	2.1 ± 0.00	2.1 ± 0.01	1.92 ± 0.01
Water absorption after 24 h (%)	0.11 ± 0.03	0.06 ± 0.01	0.07 ± 0.02	0.06 ± 0.01	0.10 ± 0.01
Water/Moisture absorption at equilibrium (%)	0.18 ± 0.03	0.16 ± 0.03	0.21 ± 0.03	0.15 ± 0.01	0.27 ± 0.03
Cure ratio (%)	100.0 ± 0.00	100.0 ± 0.00	100.0 ± 0.00	99.0 ± 0.00	93.5 ± 0.50
T _g (°C)	111.4 ± 2.19	103.2 ± 4.32	115.8 ± 3.03	111.0 ± 2.55	68.4 ± 1.14

Table 5.5: Physical properties of prestressing 7-mm and BASA-8-mm (code 43 and 51)

Property	Prestressing 7-mm	BASA-8-mm	
		Code 43	Code 51
Cross-sectional area (mm ²)	48.6 ± 0.55	74.2 ± 0.84	72.6 ± 0.55
Fiber content by weight (%)	85.1 ± 0.01	77.4 ± 0.18	81.1 ± 0.25
Transverse CET (×10 ⁻⁶ °C ⁻¹)	18.7 ± 0.72	26.8 ± 1.23	18.4 ± 2.46
Density	2.2 ± 0.01	2.0 ± 0.02	2.0 ± 0.02
Water absorption after 24 h (%)	0.04 ± 0.03	-	-
Water/Moisture absorption at equilibrium (%)	0.15 ± 0.02	0.56 ± 0.04	0.62 ± 0.03
Cure ratio (%)	97.6 ± 0.65	100.0 ± 0.00	100.0 ± 0.00
T _g (°C)	102.2 ± 2.59	117.4 ± 1.52	126.6 ± 1.82

Table 5.6: Specified limits for physical properties of the (CSA-S807, 2010) and (ACI 440.6M, 2008)

Measured Property	Specified Limits	
	ACI 440.6	CSA S807
Fiber content, wt%	55% by vol.	70
Transverse CET, $\times 10^{-6} \text{ }^{\circ}\text{C}^{-1}$	N.A	40
Void content, %	N.A	1
Water content at saturation, wt%	1	1.0 (D2); 0.75 (D1)*
Cure ratio, %	NA	93 (D2); 95 (D1)*
T _g , $^{\circ}\text{C}$	100	80 (D2); 100 (D1)*

* D1 and D2 can be found elsewhere in the (CSA-S807, 2010).

Table 5.7: Analyses performed on conditioned samples

No	Conditioning	SEM	T _g	FTIR
1	BASA 43- 1,000 h - Alkaline - 60°C	x		
2	BASA 43- 3,000 h -Alkaline - 60°C	x	x	x
3	BASA 43- 3,000 h -Water - 60°C	x		
4	BASA 43- 3 months -Alkaline - 50°C	x	x	
5	BASA 51- 1,000 h - Alkaline - 60°C	x		
6	BASA 51- 3,000 h - Alkaline - 60°C	x	x	
7	BASA 51- 3,000 h -Water - 60°C	x		x
8	BASA 51- 3 months - Alkaline - 50°C	x	x	

Table 5.8: Glass transition temperature, T_g , in °C

Sample	Conditioning	T_g
BASA BFRP 8-mm (code 43)	Reference	118
	3,000 h at 60°C in alkaline sol.-Run 1	108
	3,000 h at 60°C in alkaline sol.-Run 2	121
	3 months at 50°C in alkaline sol.-Run 1	111
	3 months at 50°C in alkaline sol.-Run 2	120
BASA BFRP 8-mm (code 51)	Reference	127
	3,000 h at 60°C in alkaline sol.-Run 1	113
	3,000 h at 60°C in alkaline sol.-Run 2	133
	3 months at 50°C in alkaline sol.-Run 1	116
	3 months at 50°C in alkaline sol.-Run 2	132

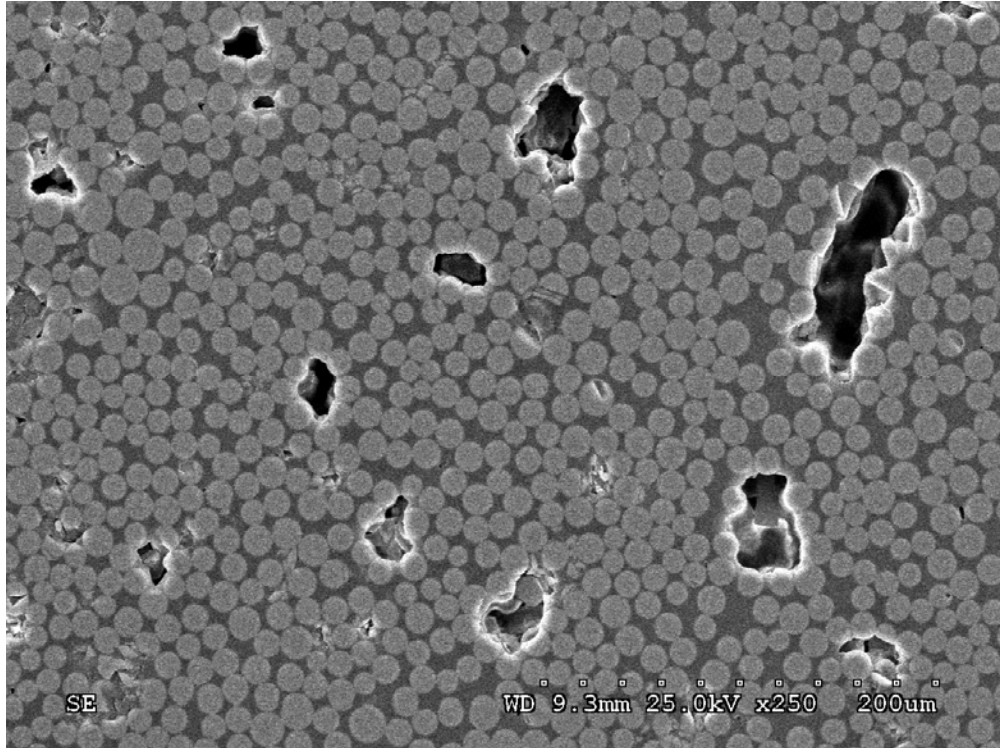


Figure 5.38: Cross section of BFRP bar 13 mm (Type A)

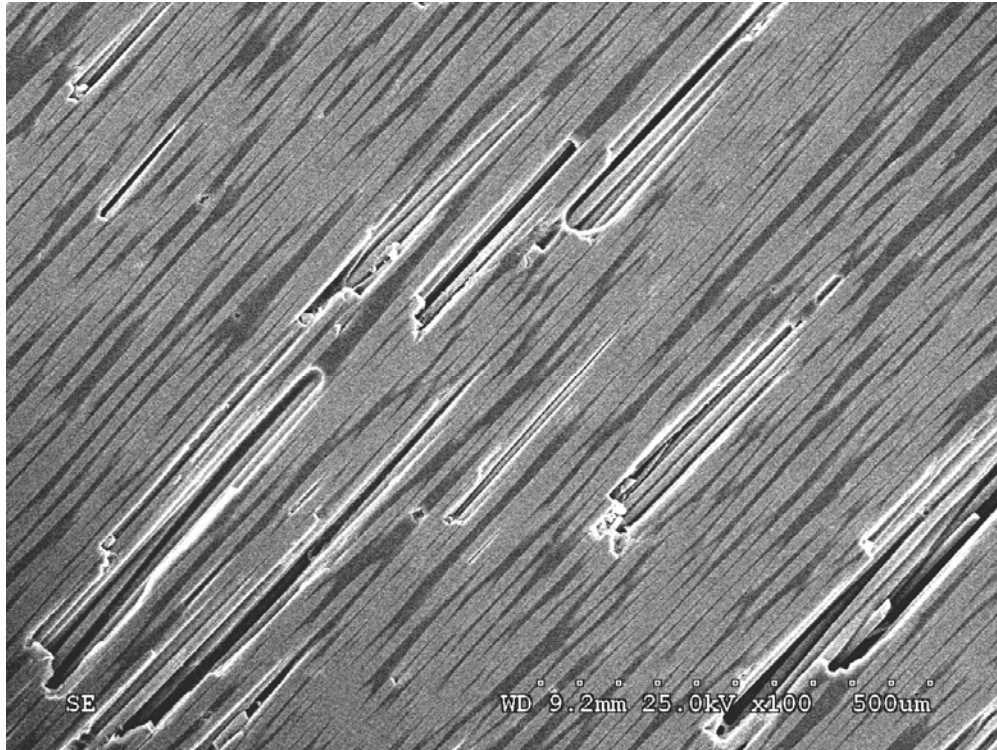


Figure 5.39: Longitudinal section of BFRP bar 13 mm (Type A)

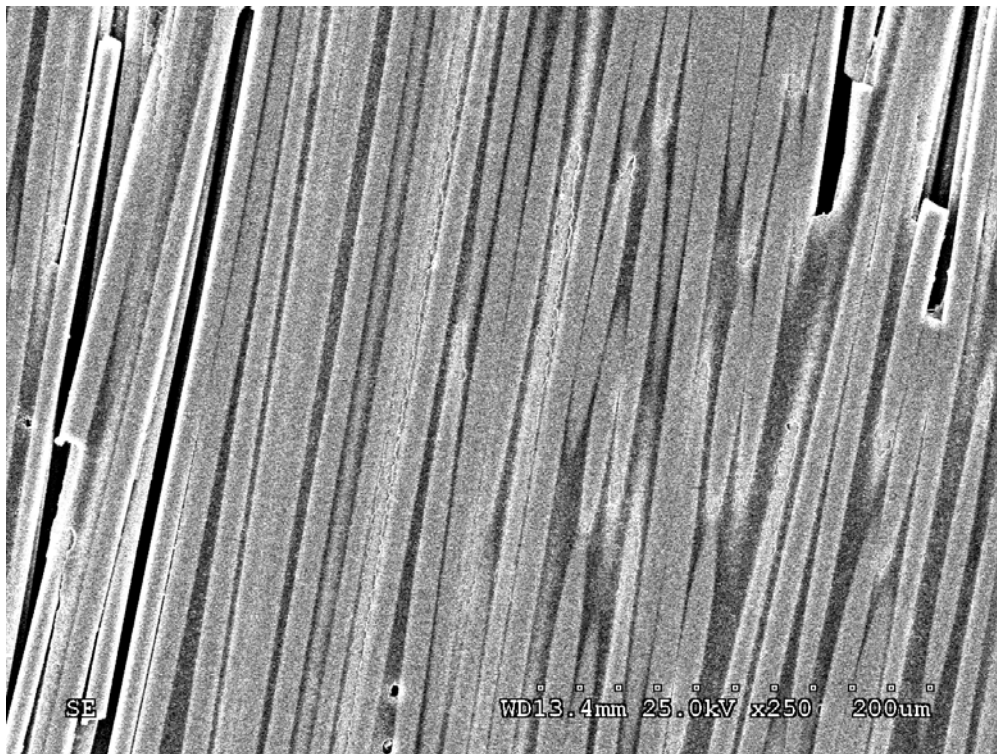


Figure 5.40: Longitudinal section of BFRP bar 8 mm (Type A)

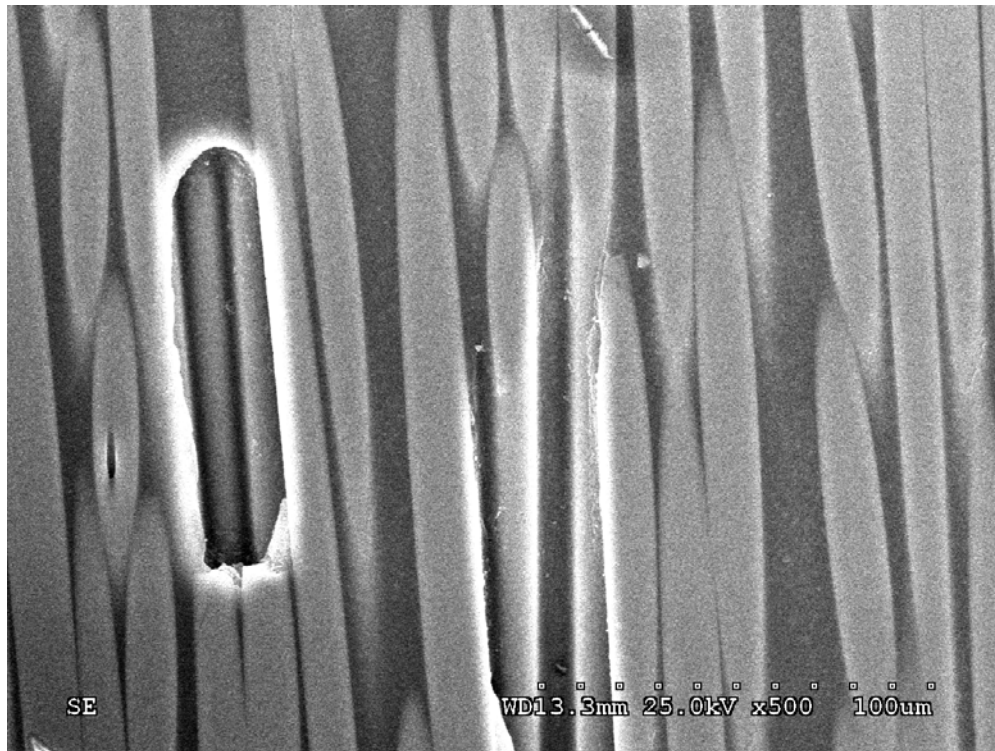


Figure 5.41: Longitudinal section of BFRP bar 10 mm (Type A)

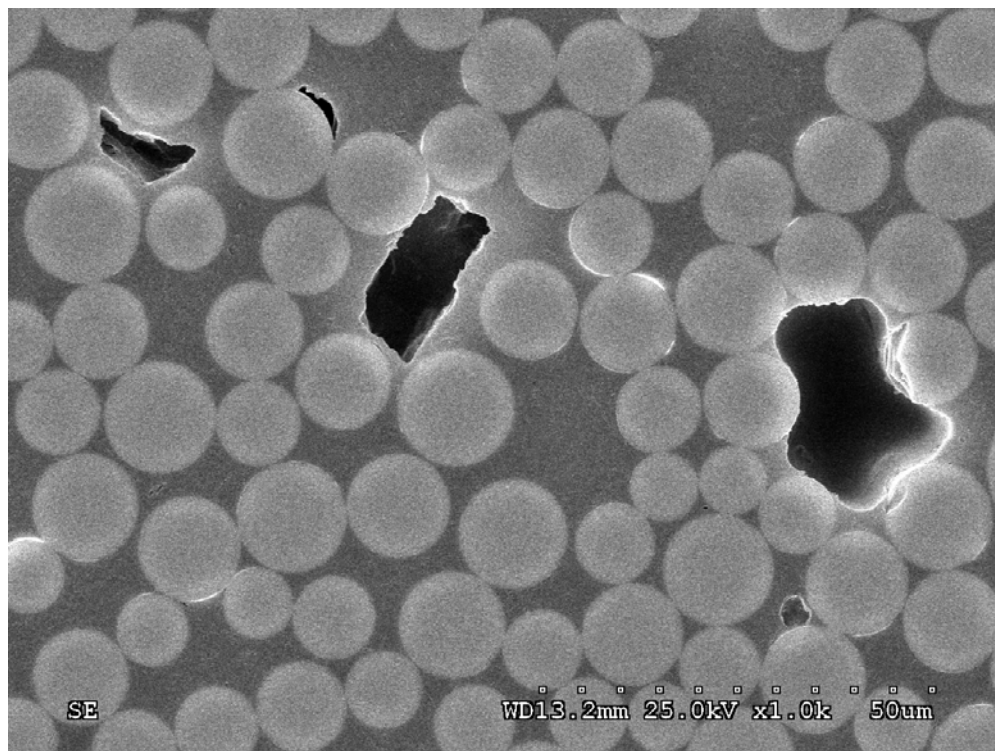


Figure 5.42: Cross section of BFRP bar 16 mm at higher magnification (Type A)

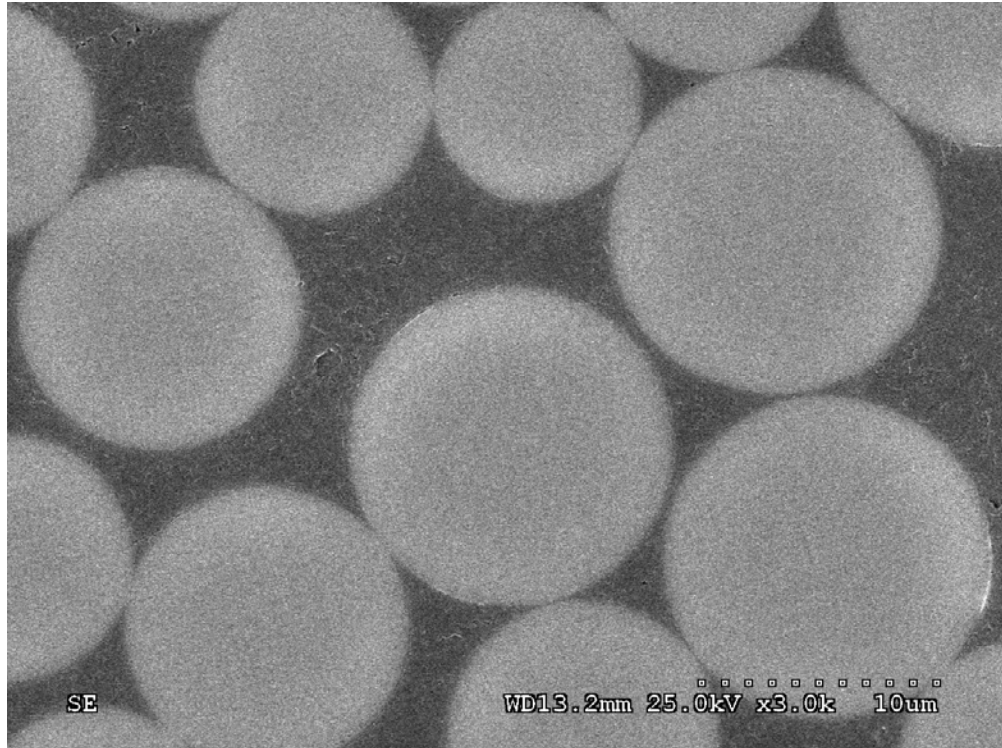


Figure 5.43: Cross section of BFRP bar 16 mm at high magnification (Type A)

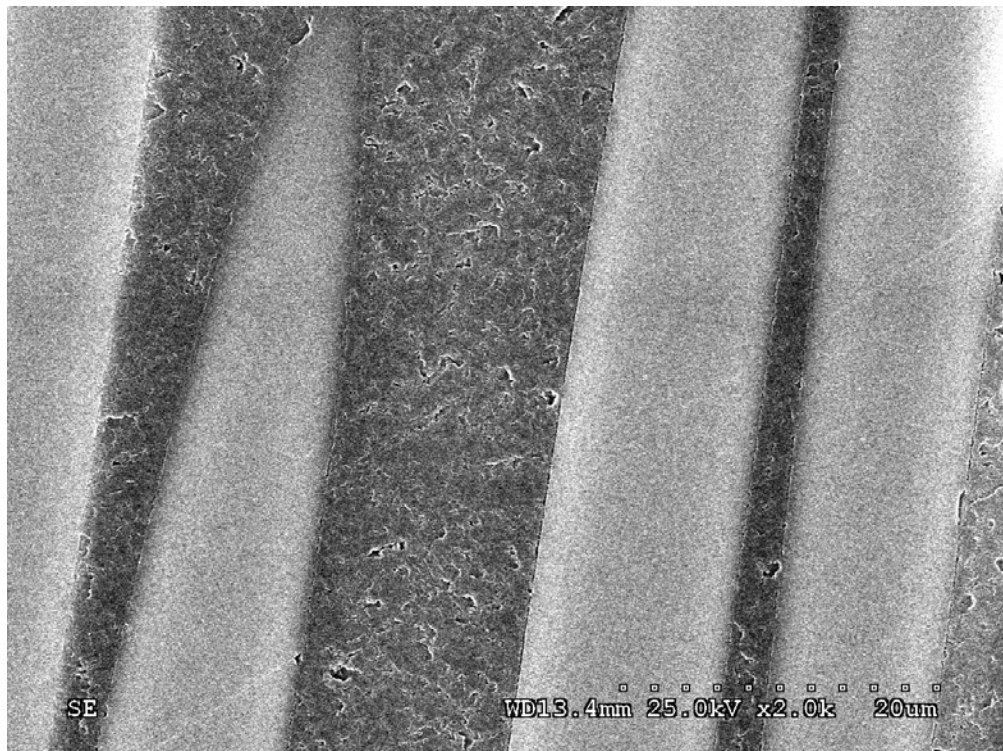


Figure 5.44: Longitudinal section of BFRP bar 8 mm at high magnification (Type A)

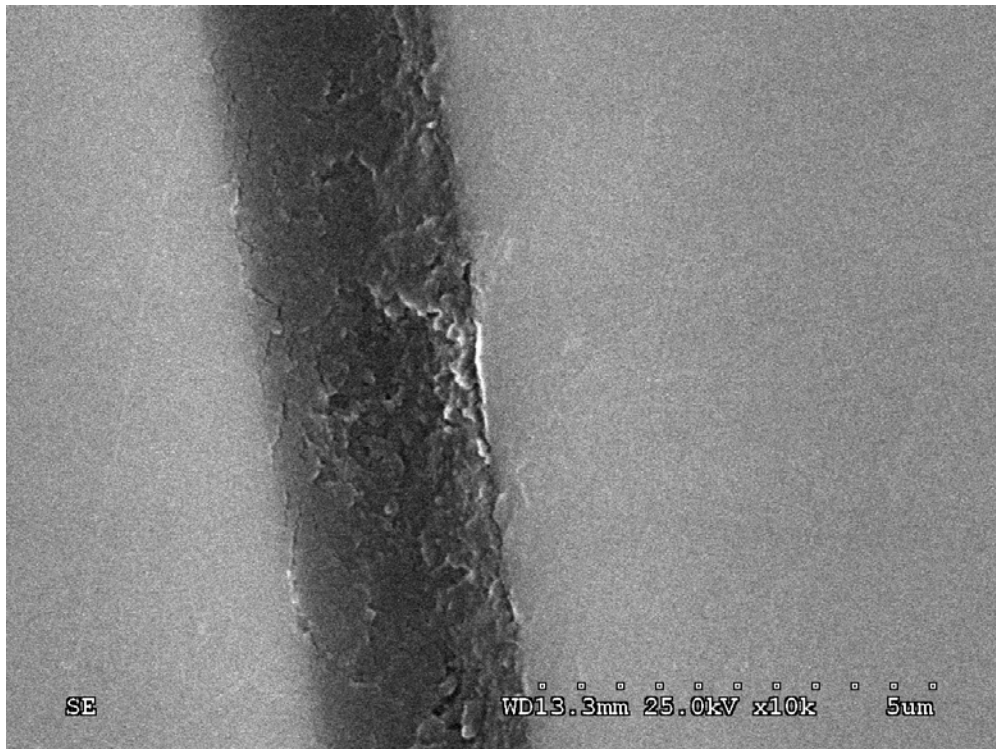


Figure 5.45: Longitudinal section of BFRP bar 10 mm at very high magnification (Type A)

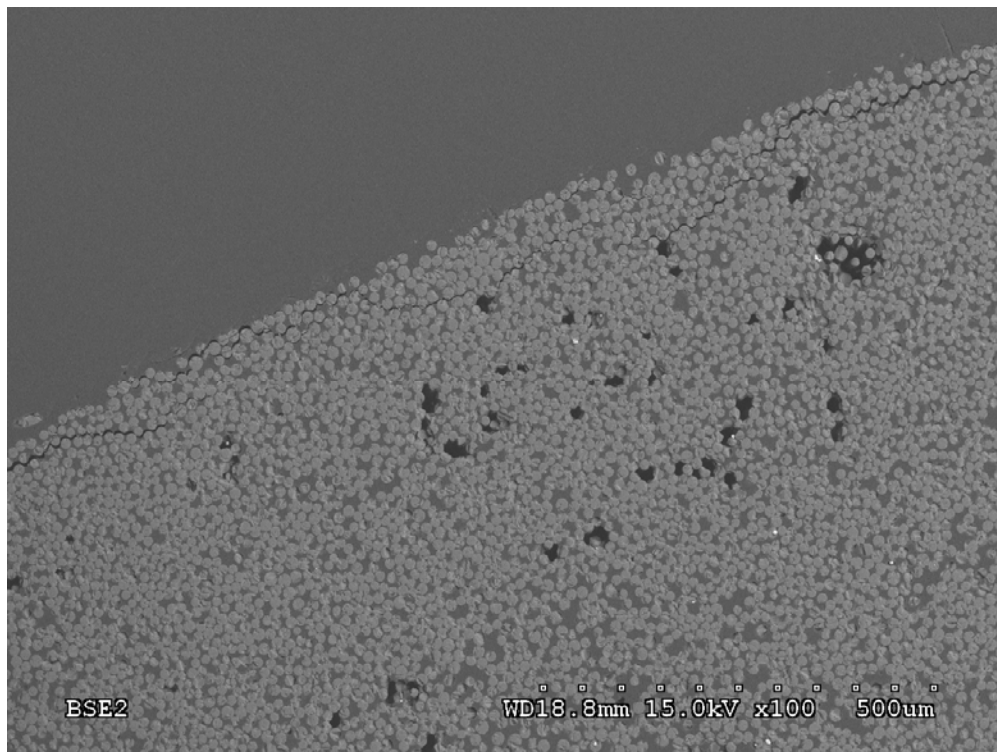


Figure 5.46: Cross section of bar B at low magnification

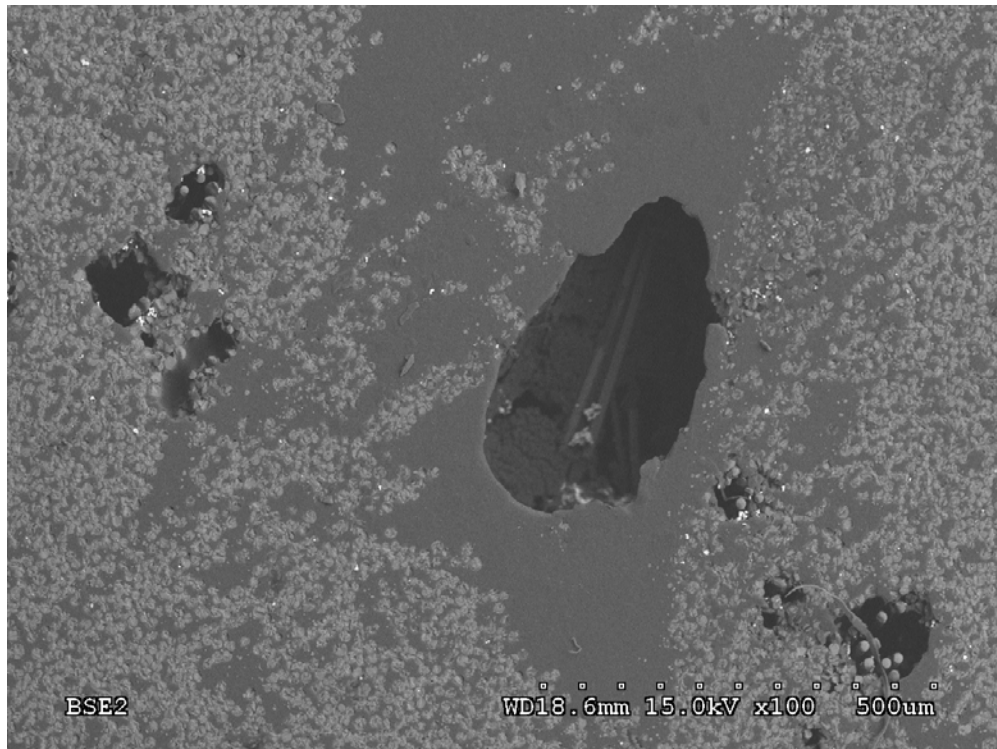


Figure 5.47: Cross section of bar B at low magnification

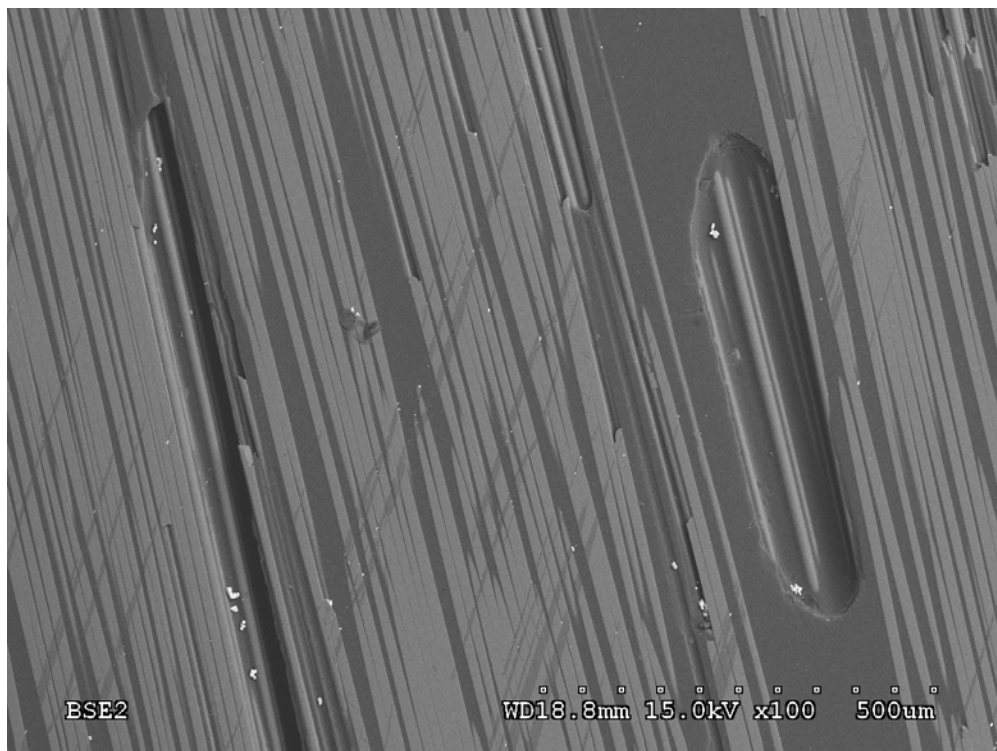


Figure 5.48: Longitudinal section of bar B at low magnification

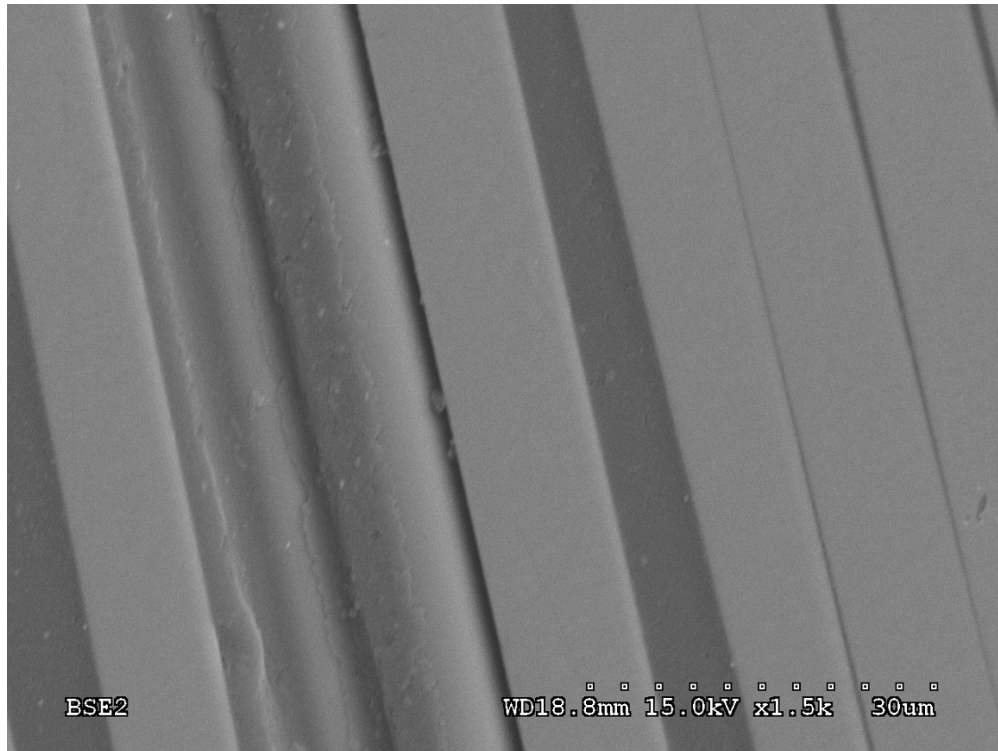


Figure 5.49: Longitudinal section of bar B at high magnification

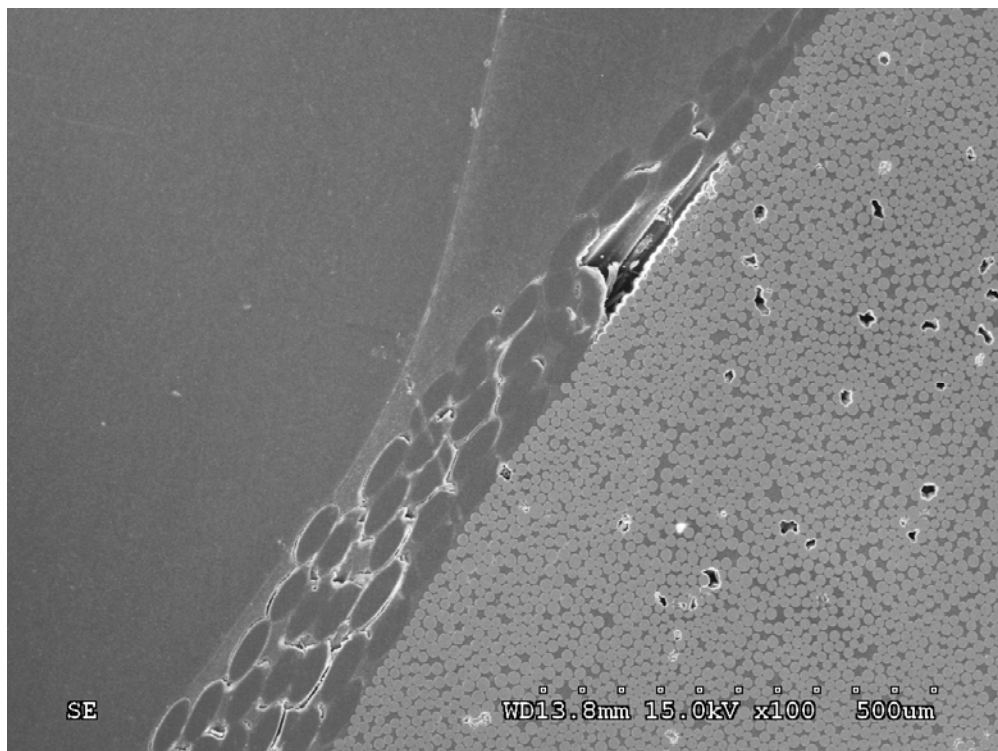


Figure 5.50: Cross section of bar C at low magnification

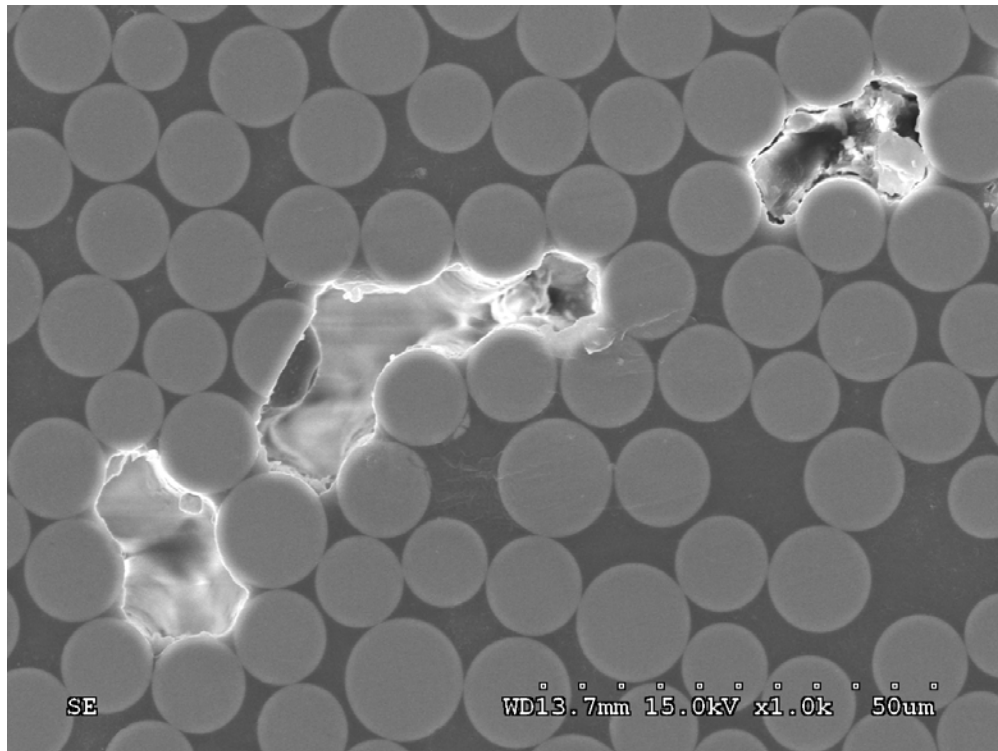


Figure 5.51: Cross section of bar C at high magnification

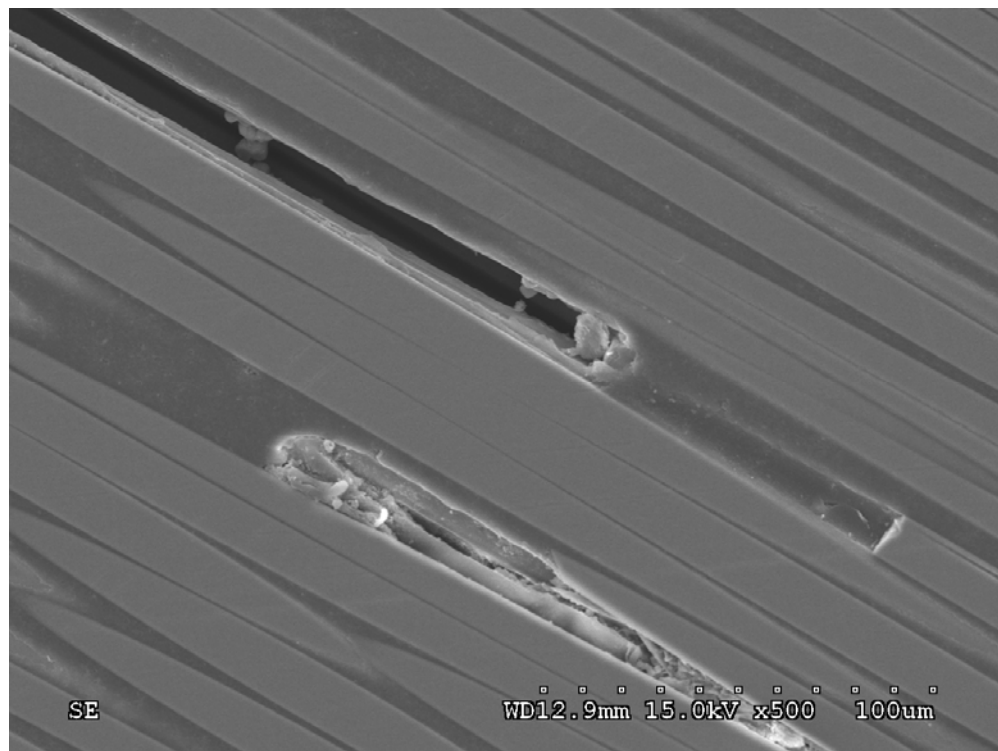


Figure 5.52: Longitudinal section of bar C at high magnification

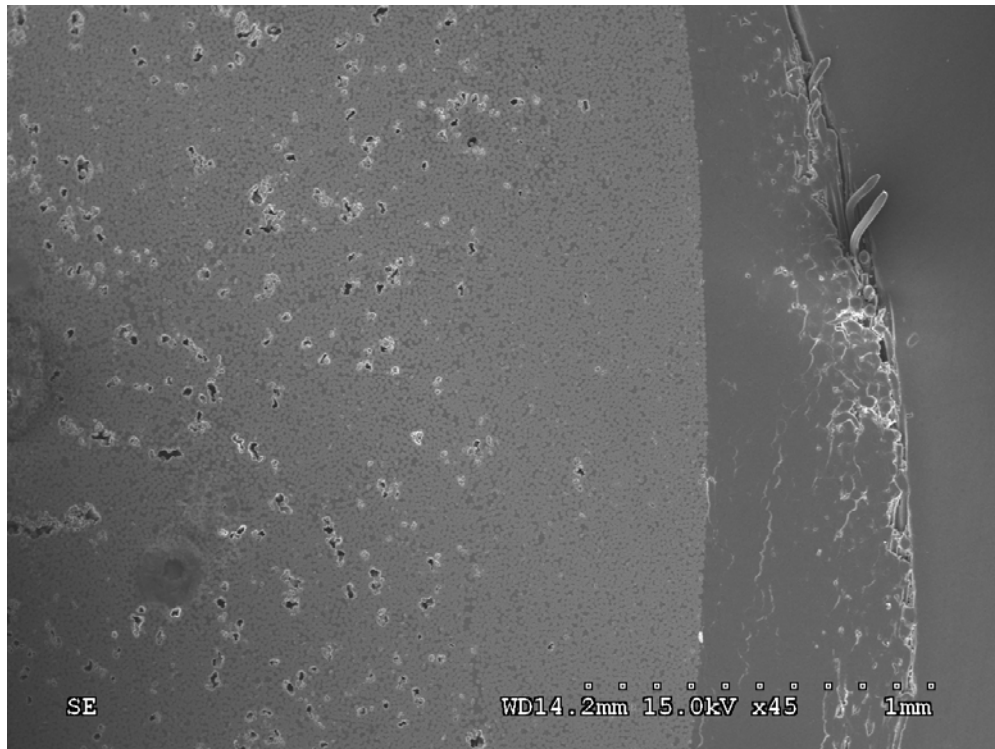


Figure 5.53: Cross section of bar D at low magnification

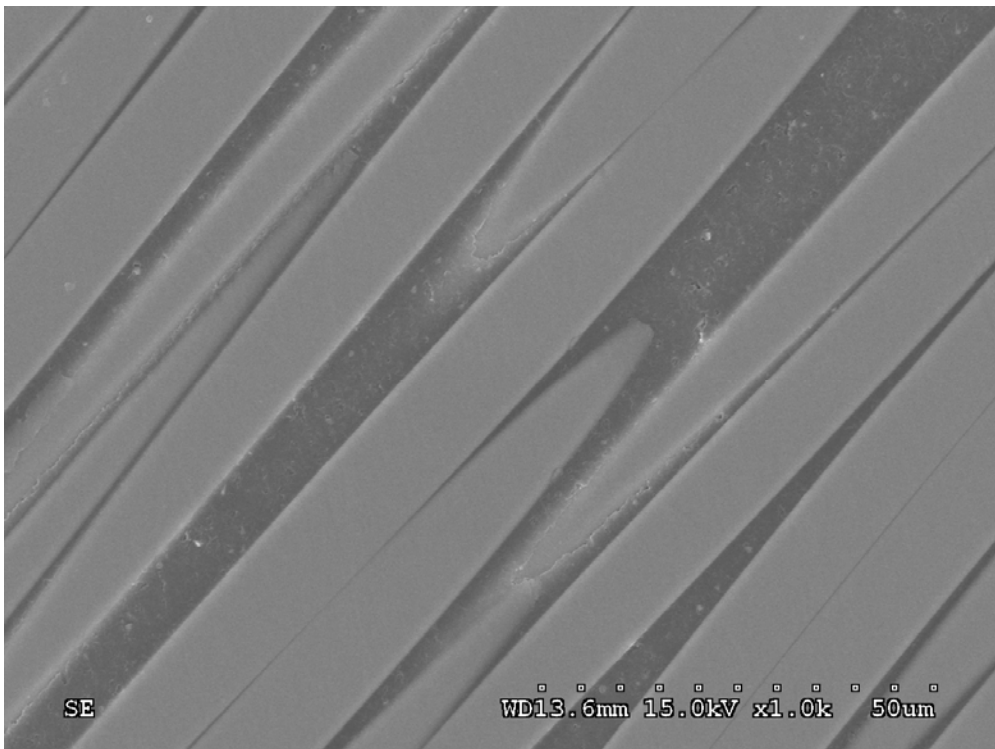


Figure 5.54: Longitudinal section of bar D at high magnification

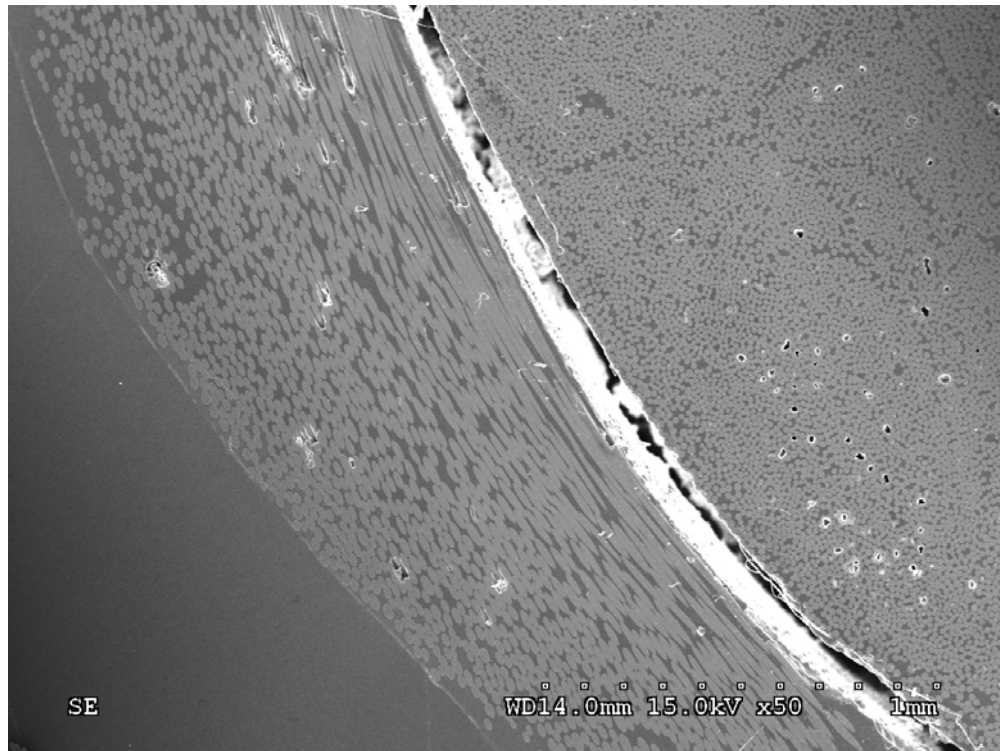


Figure 5.55: Cross section of bar E at low magnification

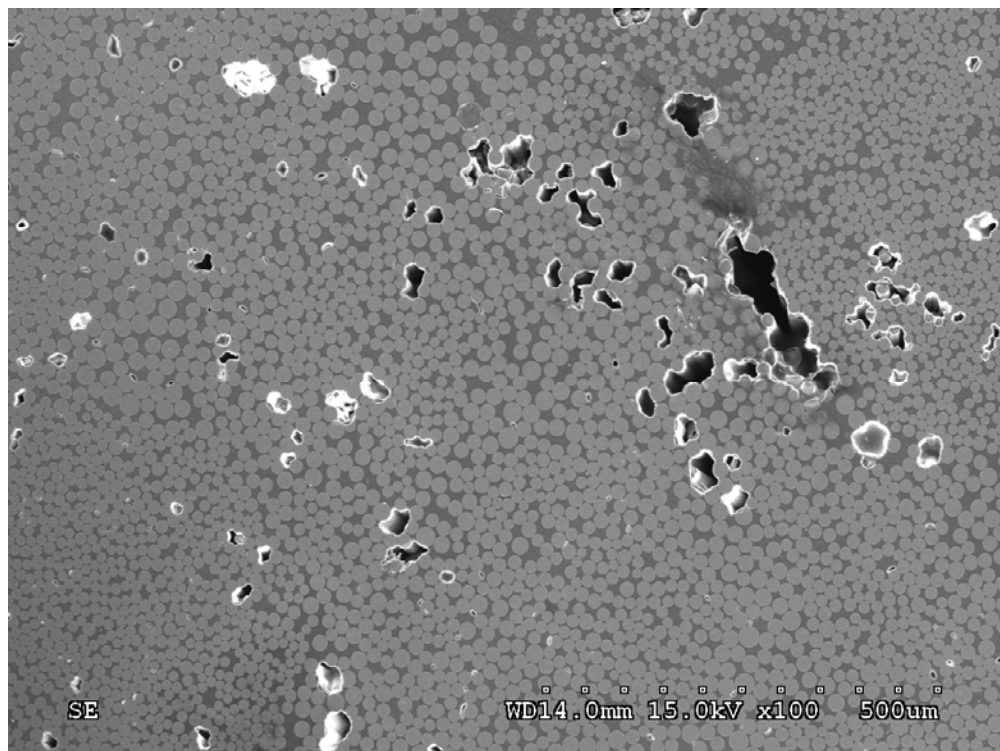


Figure 5.56: Cross section of bar E at higher magnification

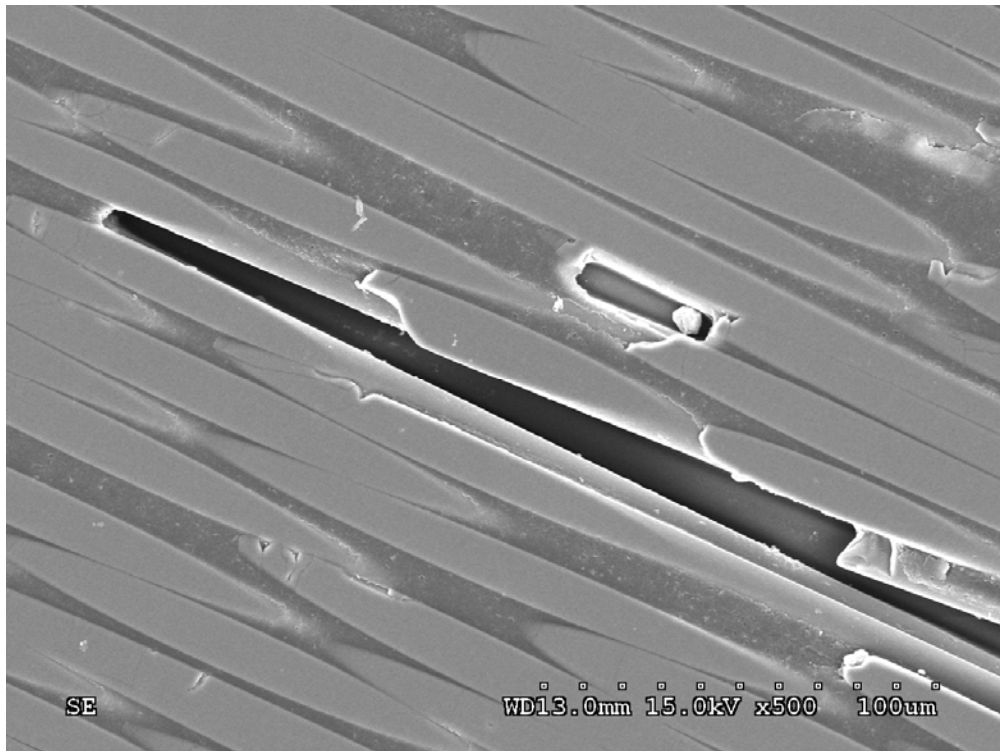


Figure 5.57: Longitudinal section of bar E at high magnification

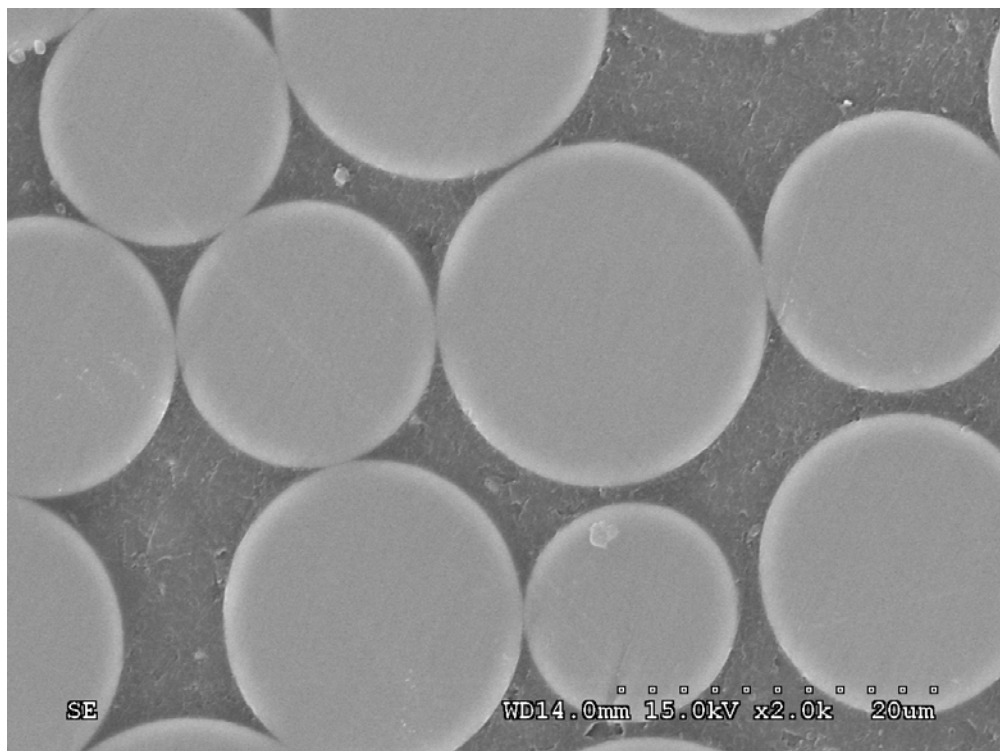


Figure 5.58: Cross section of bar E at high magnification

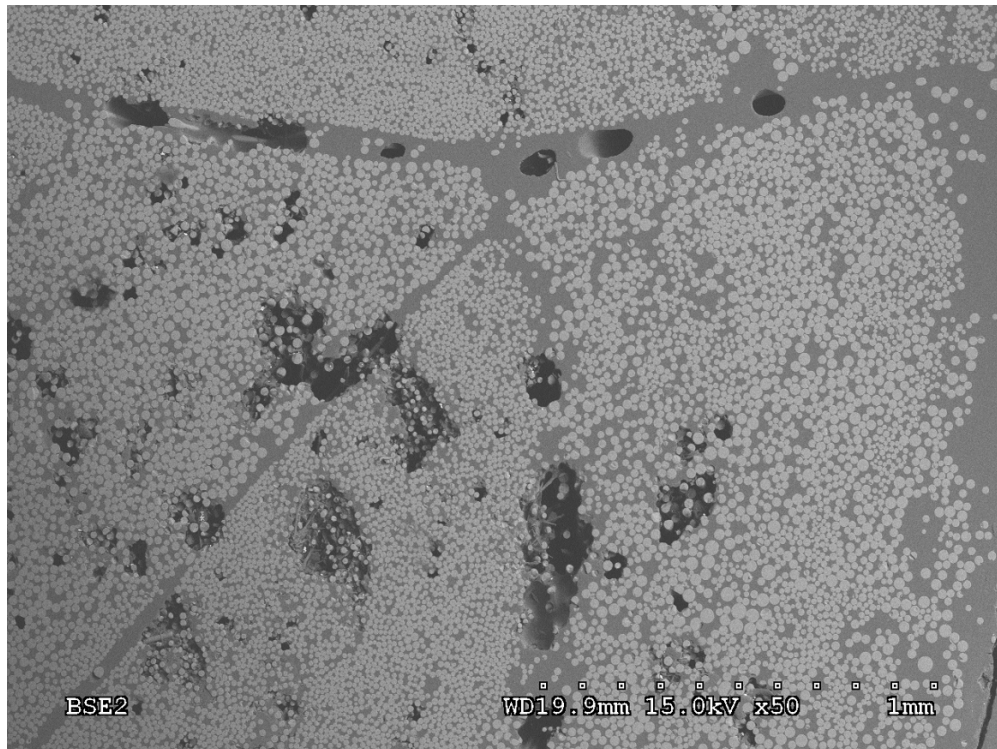


Figure 5.59: Cross section of bar F at low magnification

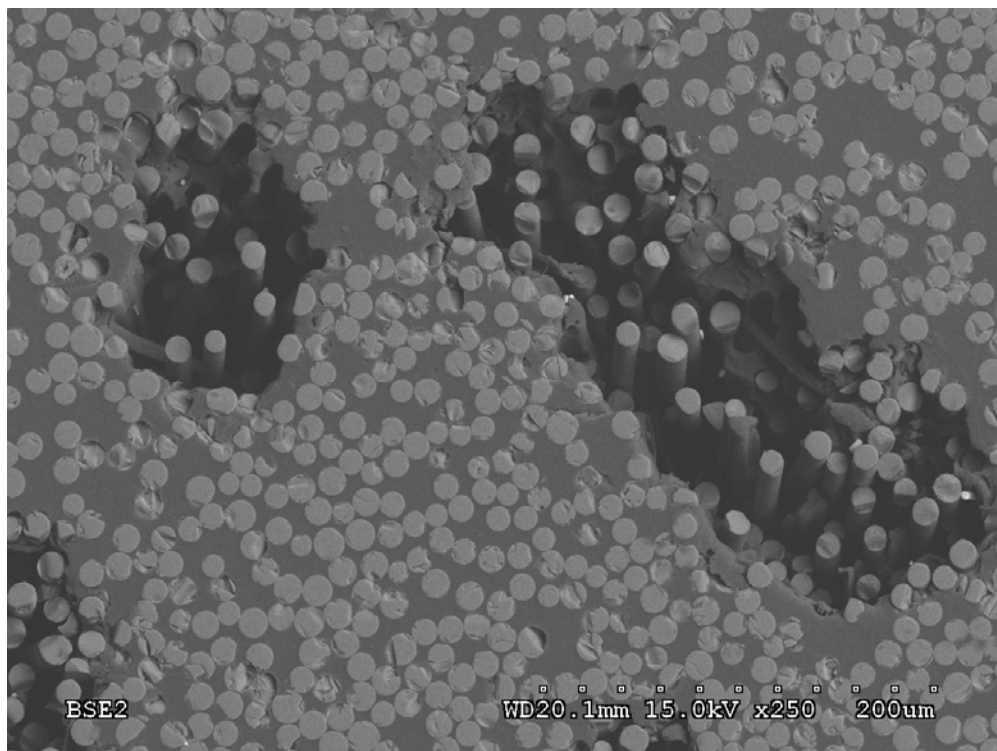


Figure 5.60: Cross section of bar F at higher magnification

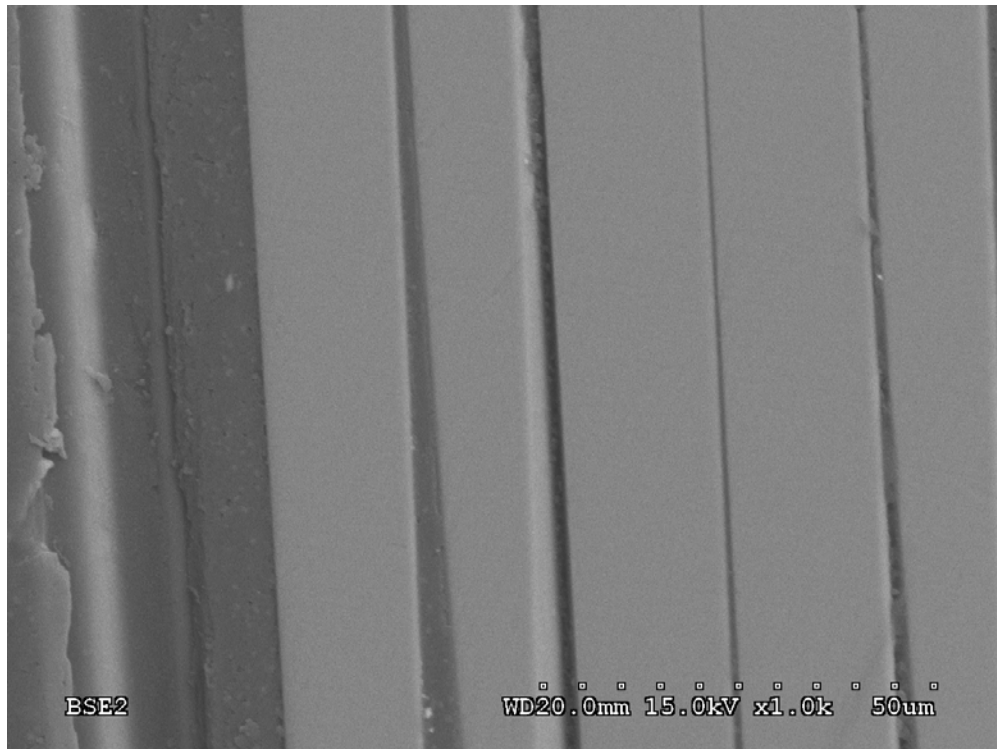


Figure 5.61: Longitudinal section of bar F at high magnification

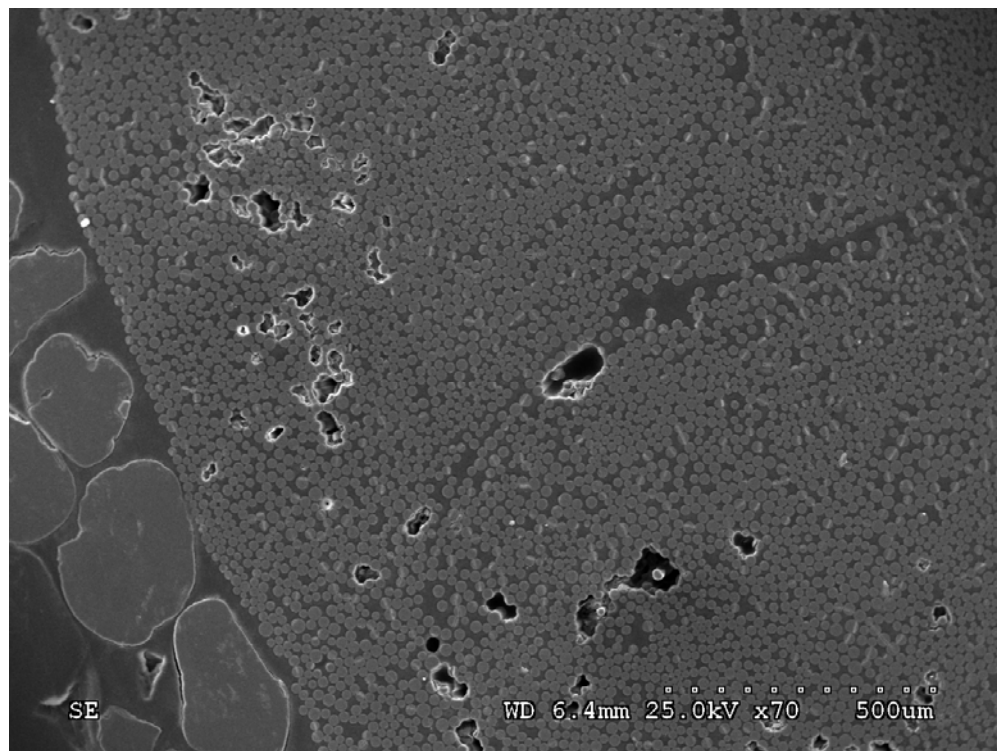


Figure 5.62: Cross section of basalt bar No. 3 showing porosities (Type G)

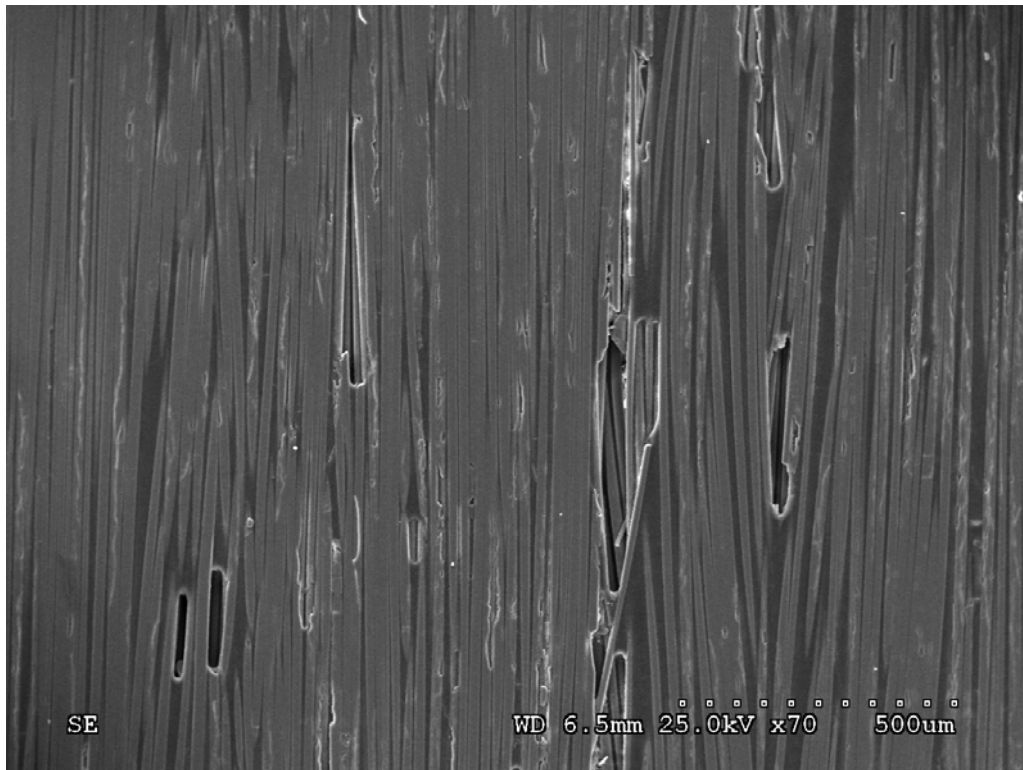


Figure 5.63: Longitudinal section of basalt bar No. 3 (Type G)

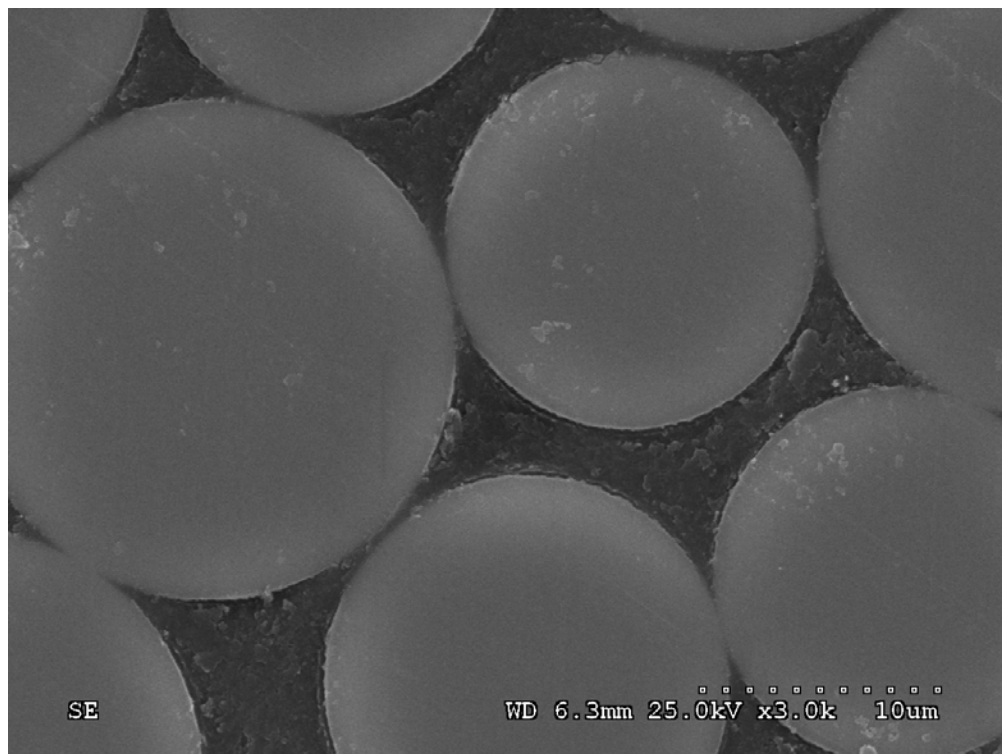


Figure 5.64: Cross section of basalt bar No. 3 at high magnification (Type G)

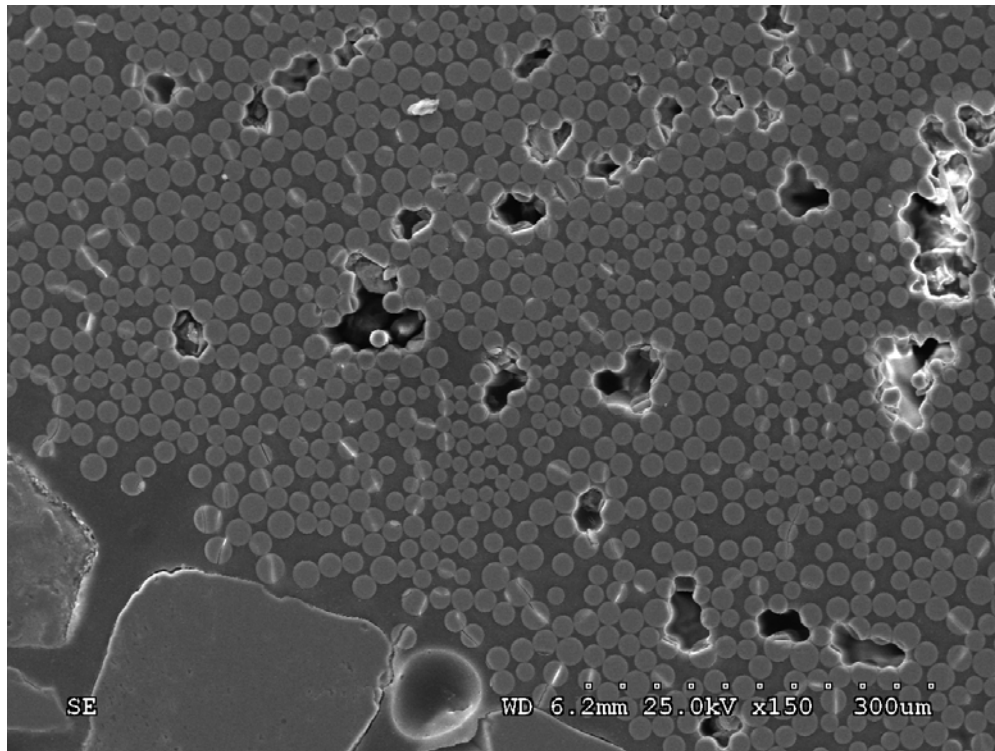


Figure 5.65: Cross section of basalt bar No. 4 (Type G)

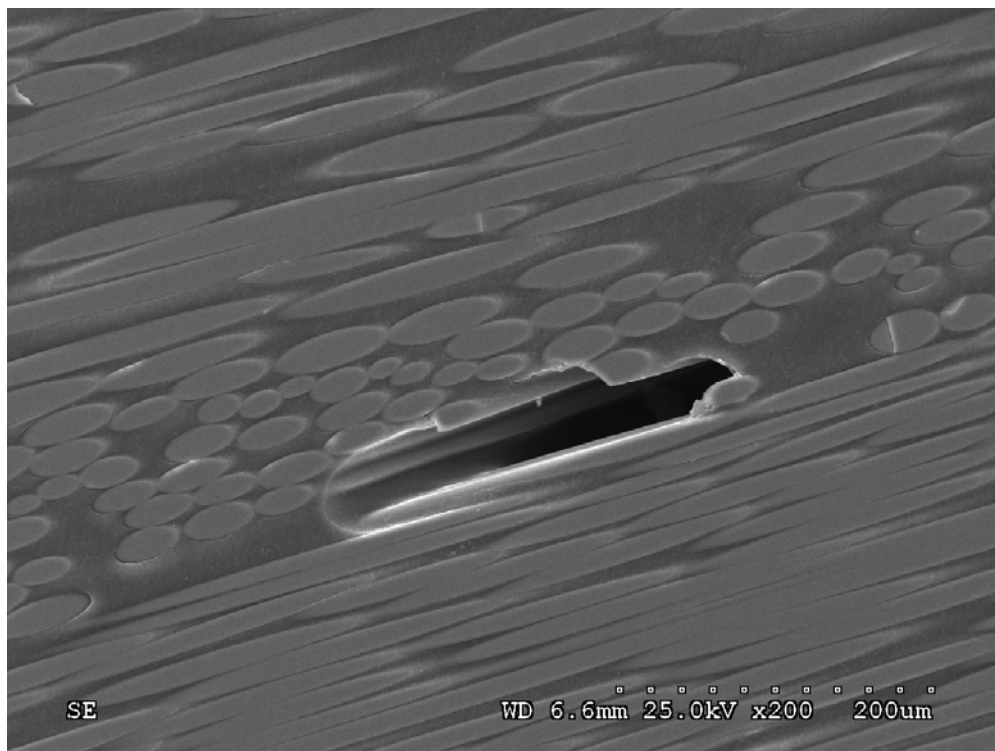


Figure 5.66: Longitudinal section of basalt bar No. 4 (Type G)

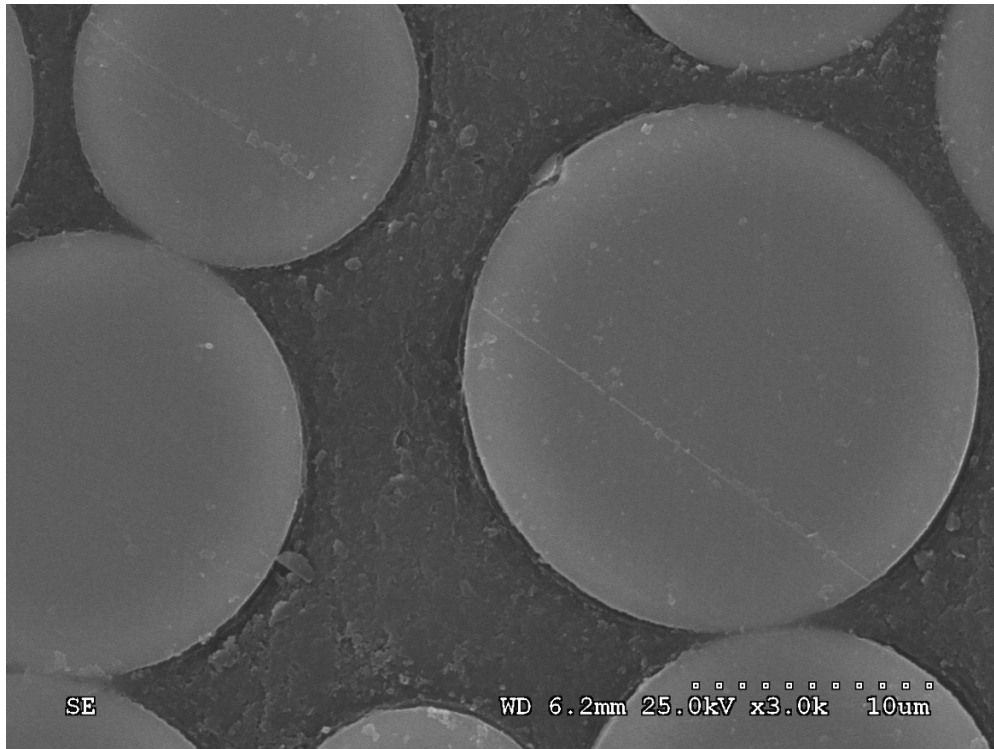


Figure 5.67: Cross section of basalt bar No. 4 at high magnification (Type G)

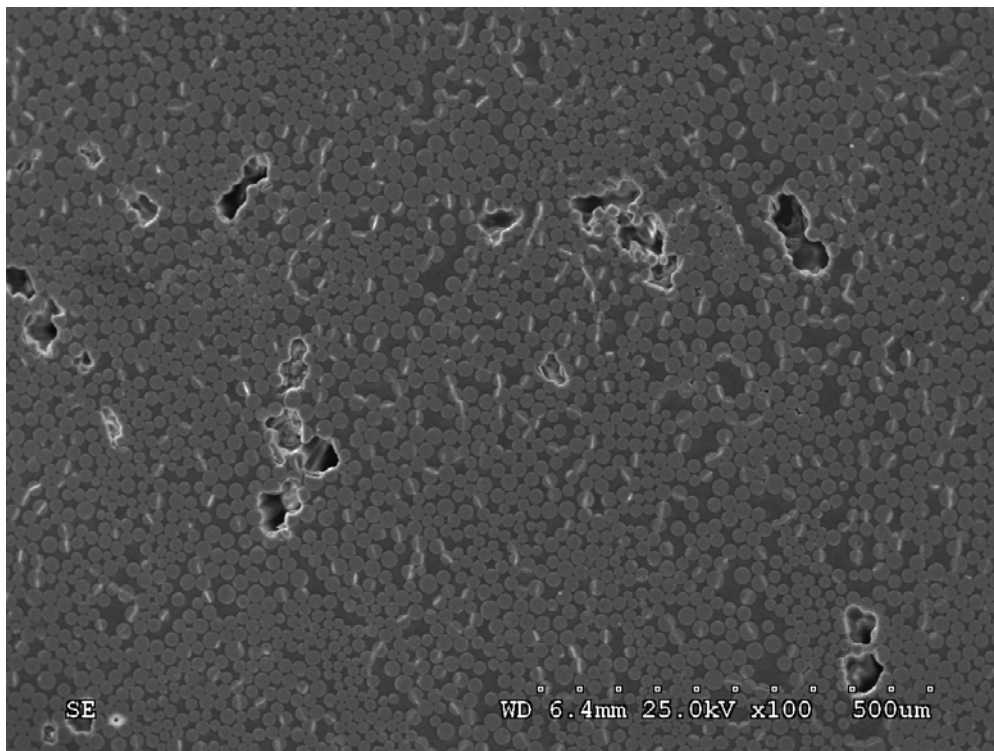


Figure 5.68: Cross section of basalt bar No. 5 (Type G)

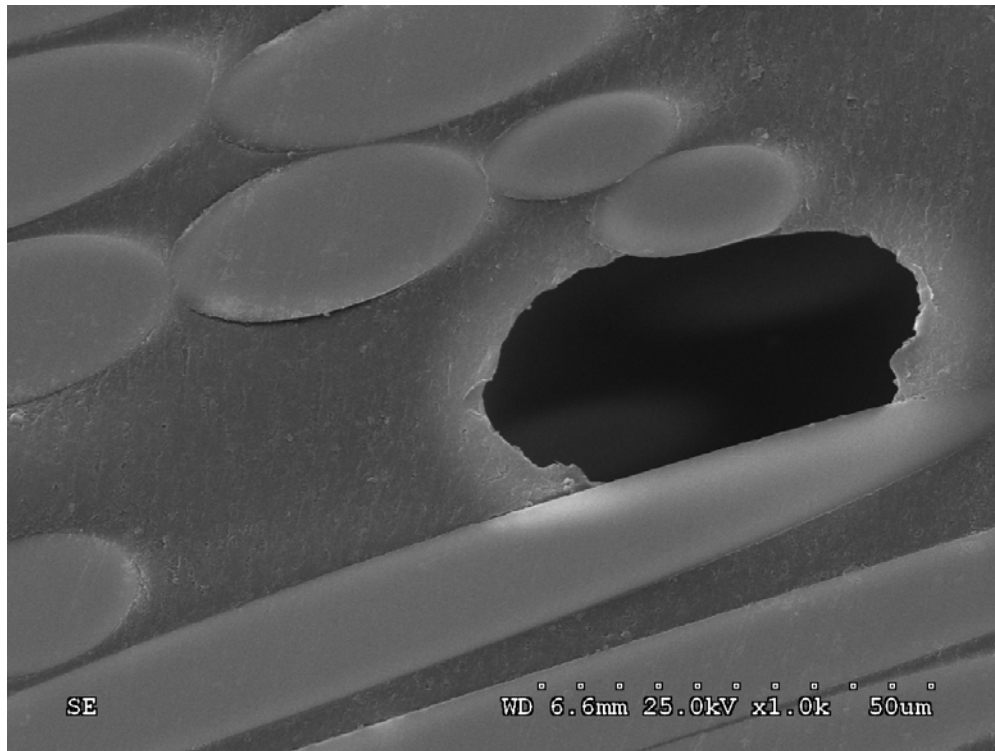


Figure 5.69: Longitudinal section of basalt bar No. 5 (Type G)

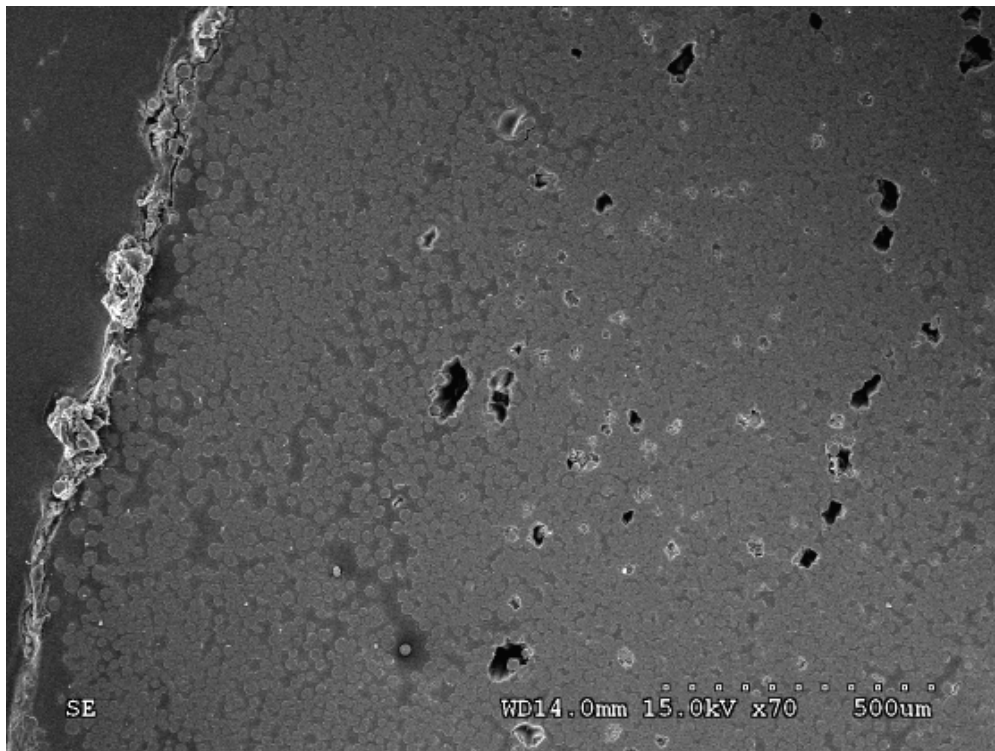


Figure 5.70: Typical general view of Basalt FRP bar at low magnification (Type H)

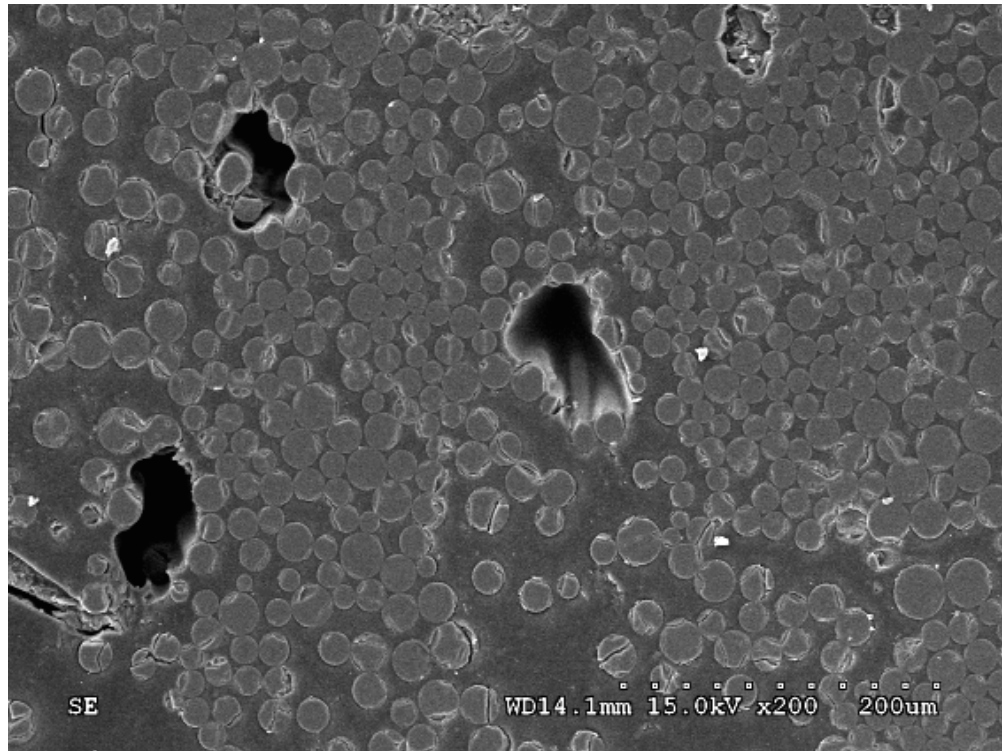


Figure 5.71: Tubular pores in basalt FRP bar (Type H)

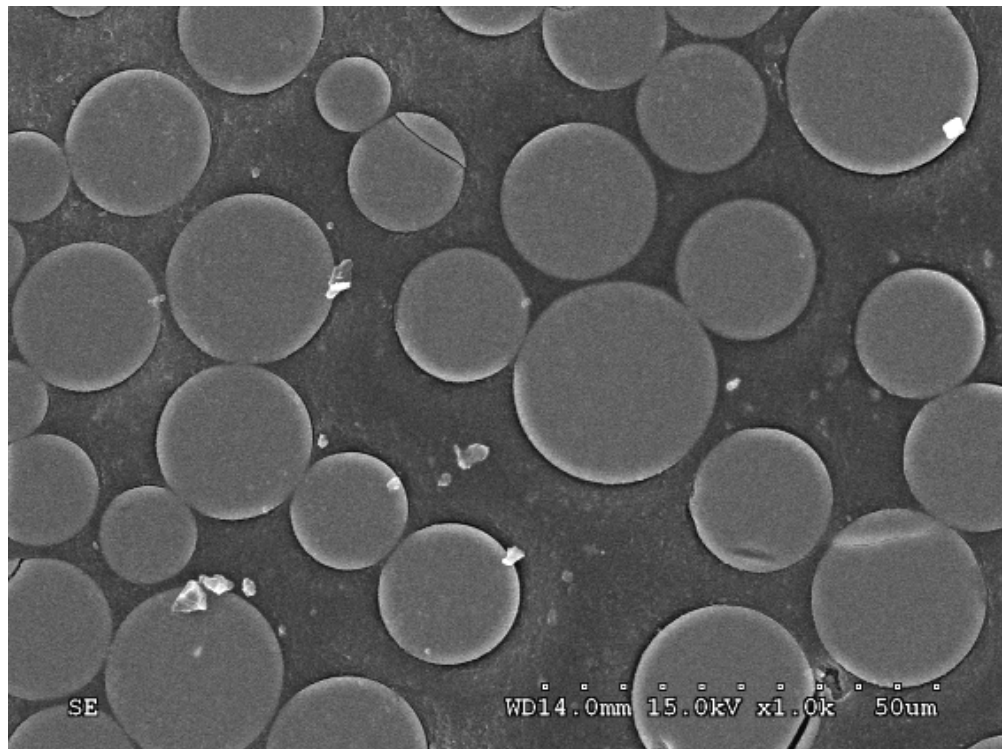


Figure 5.72: Basalt fibers embedded in resin matrix at high magnification (Type H)

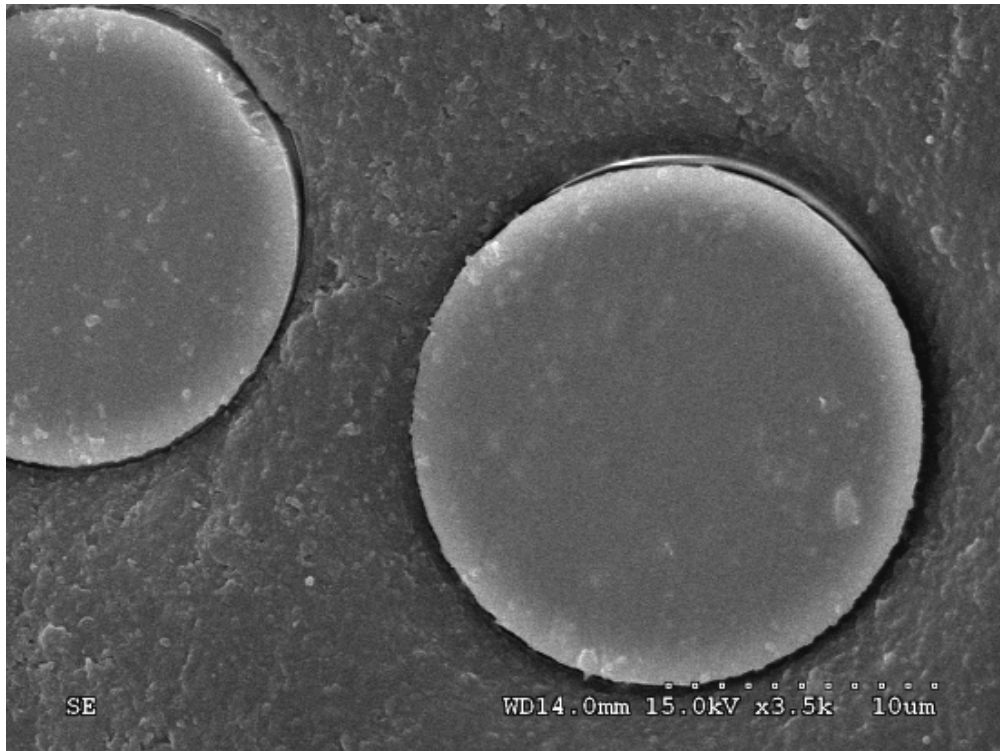


Figure 5.73: Debonding at the basalt fiber/resin matrix interface (Type H)

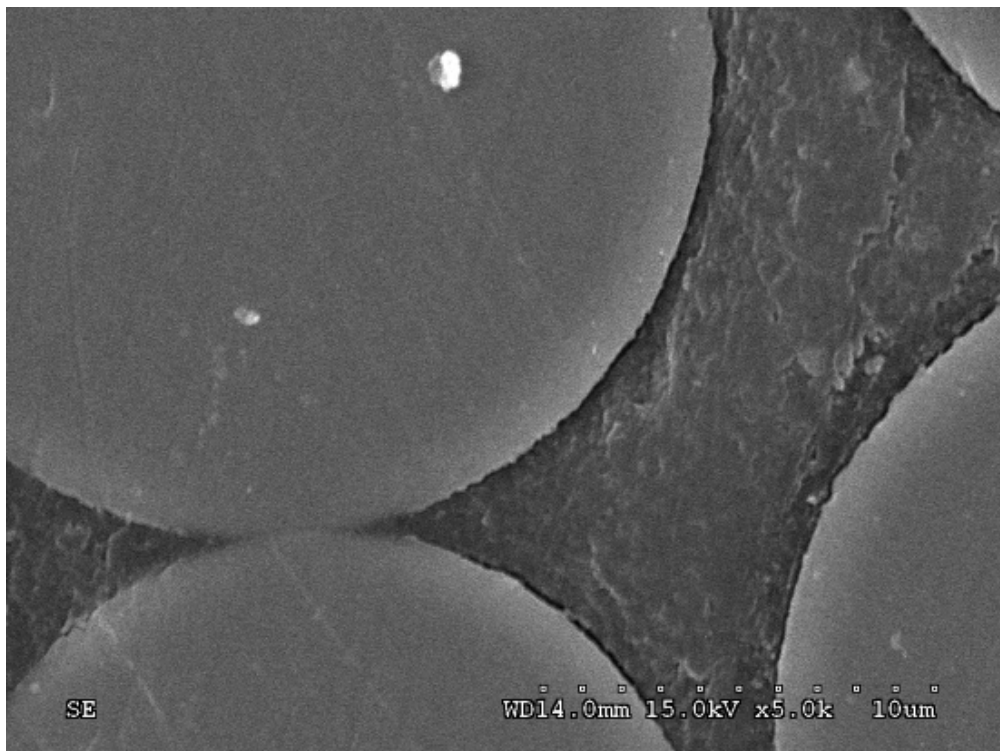


Figure 5.74: Partial debonding at the basalt fiber/resin matrix interface (Type H)

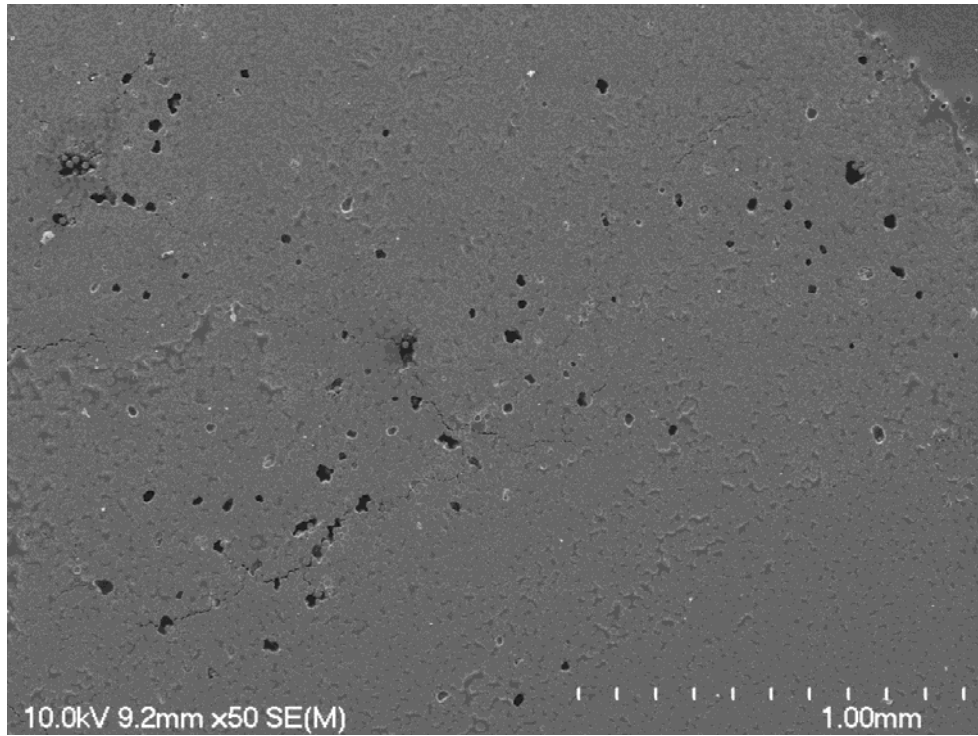


Figure 5.75: General view of the cross-section of TECH BFRP bar 12-mm

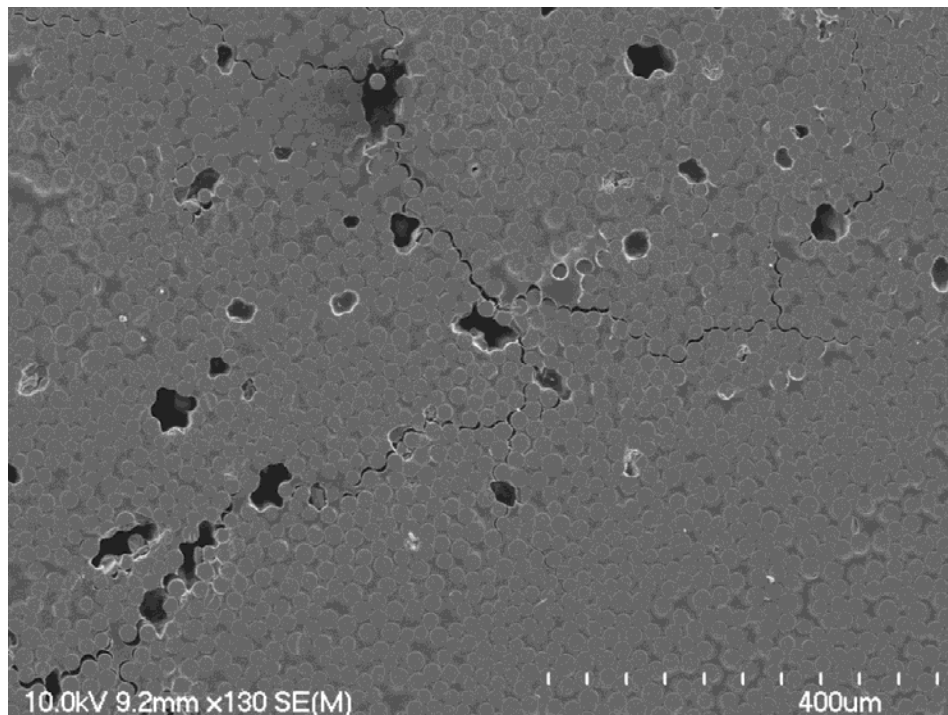


Figure 5.76: Cross-section of TECH BFRP bar 12-mm at higher magnification showing large pores and fine microcracks

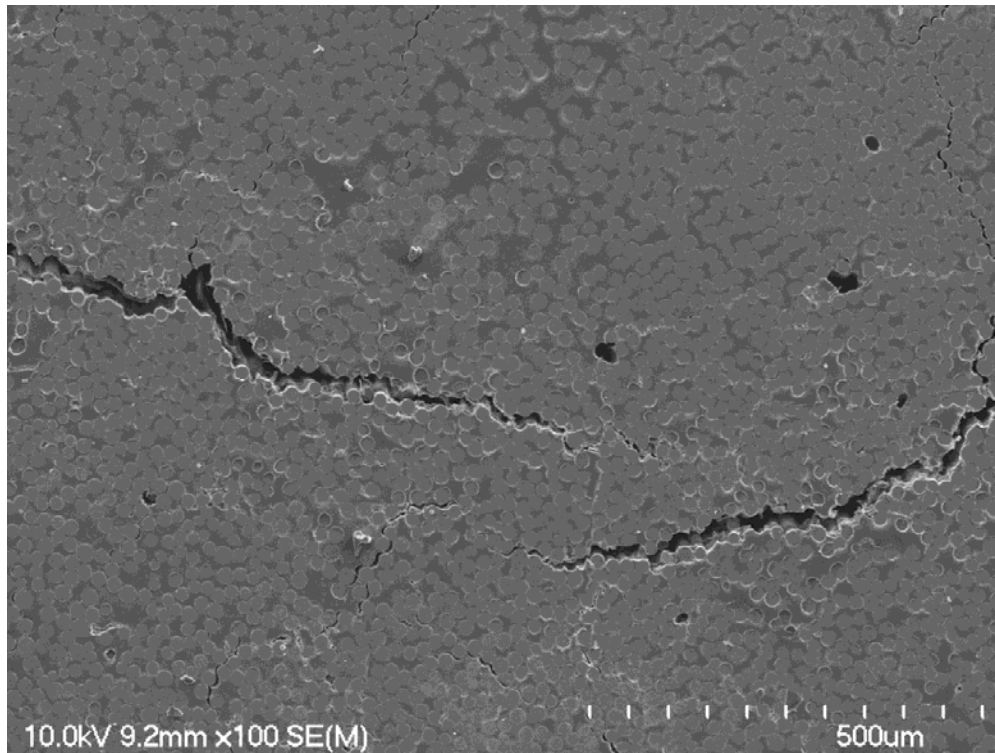


Figure 5.77: Cross-section of TECH BFRP bar 12-mm at higher magnification showing large microcracks

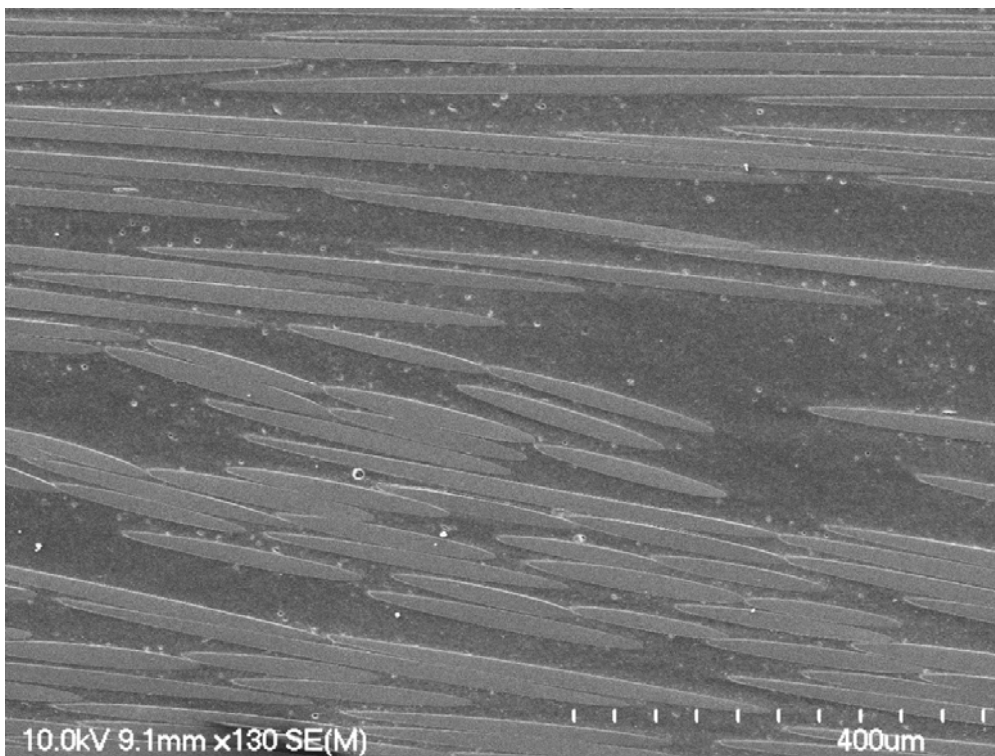


Figure 5.78: Longitudinal section of TECH BFRP bar 12-mm

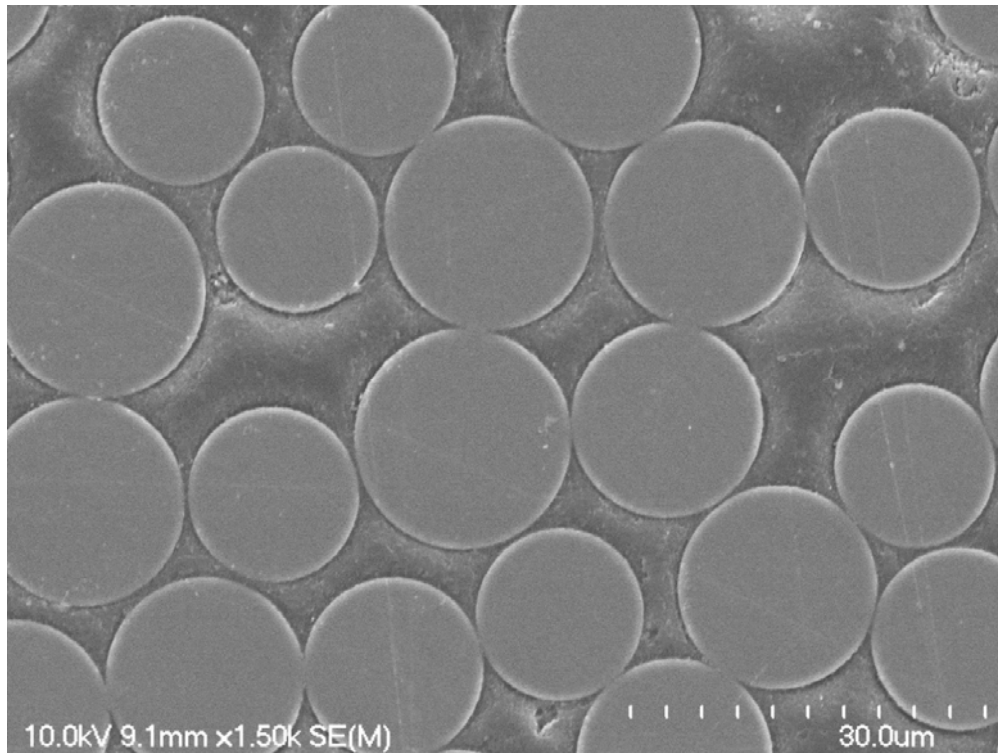


Figure 5.79: Cross-section of TECH BFRP bar 12-mm at high magnification

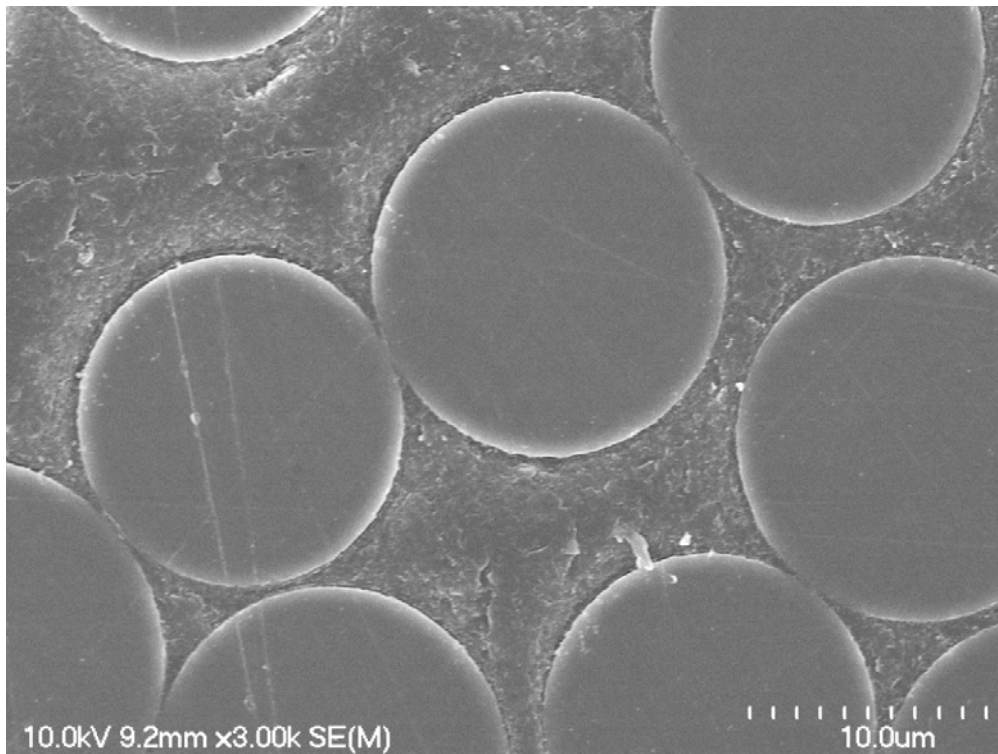


Figure 5.80: Cross-section of TECH BFRP bar 12-mm at very high magnification

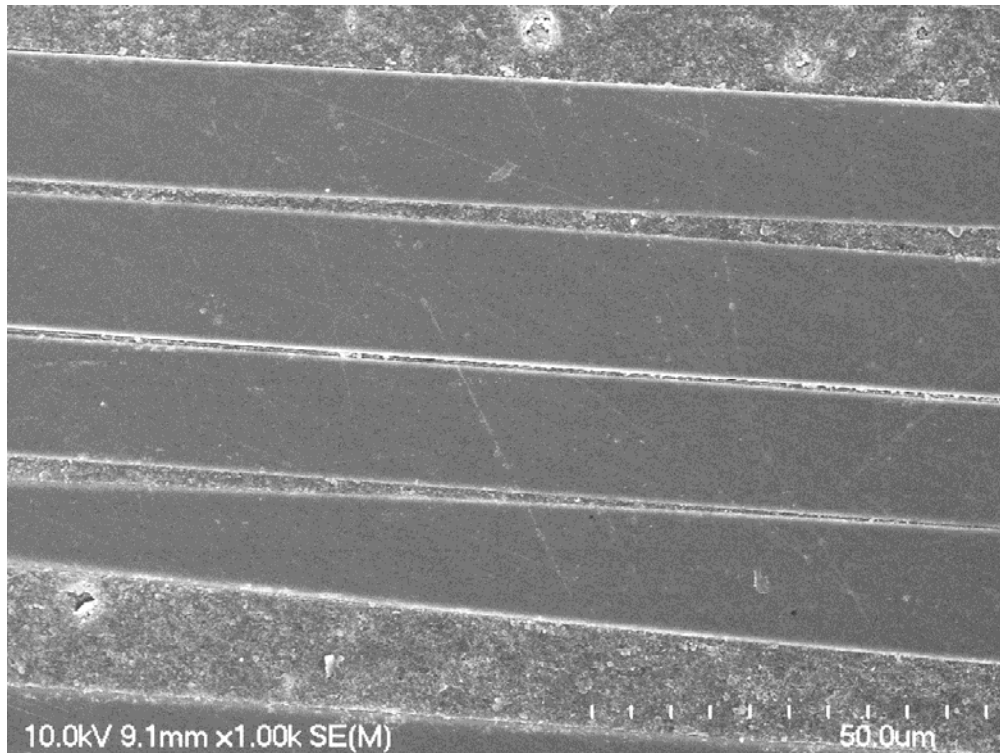


Figure 5.81: Longitudinal section of TECH BFRP bar 12-mm at high magnification

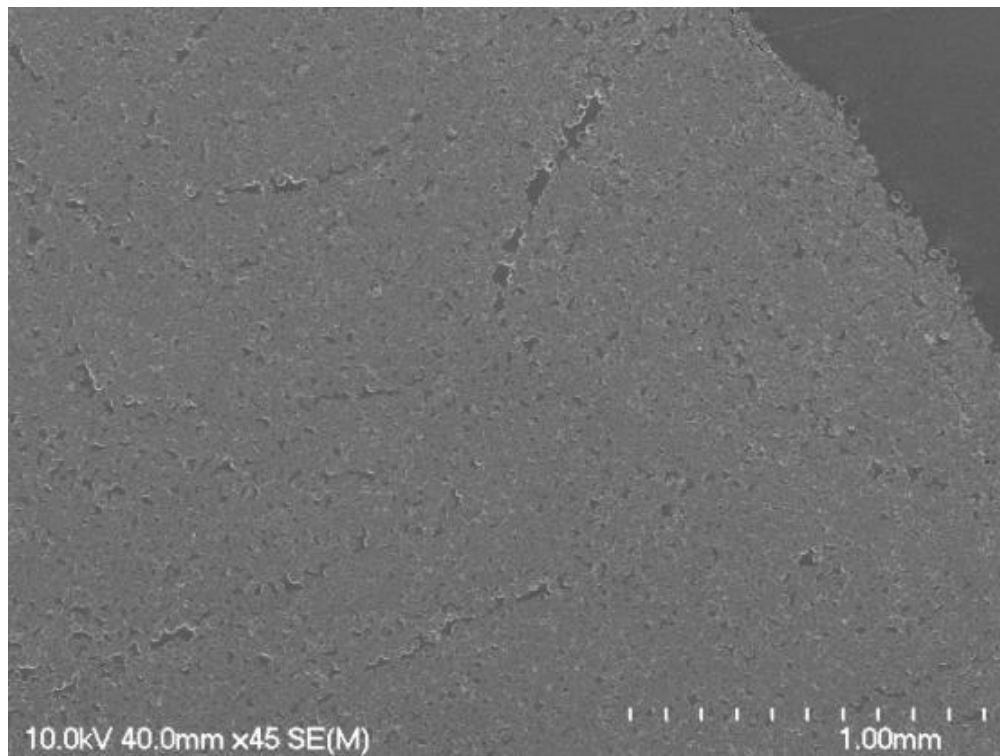


Figure 5.82: General view of the cross-section of prestressing BFRP bar 7-mm

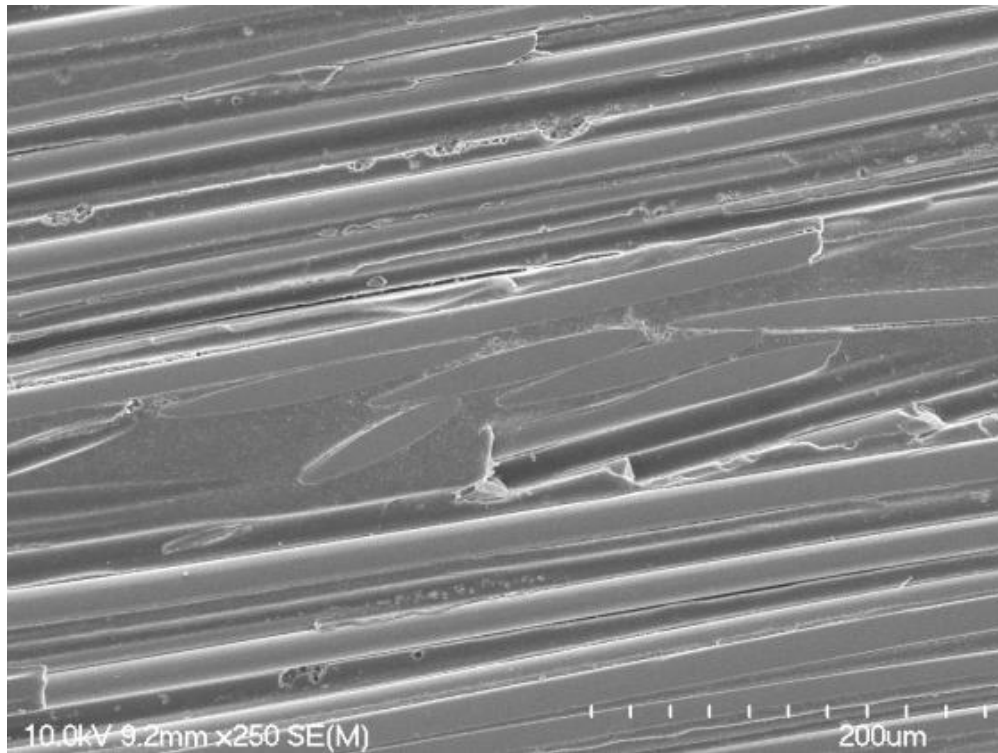


Figure 5.83: Longitudinal section of prestressing BFRP bar 7-mm at medium magnification

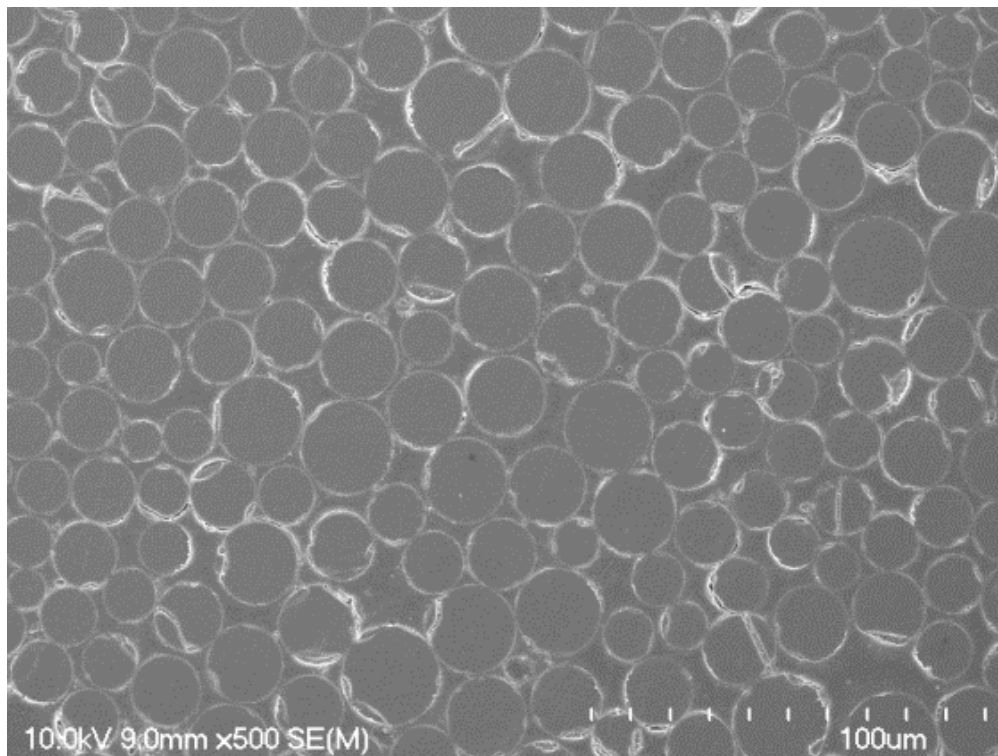


Figure 5.84: Cross-section of prestressing BFRP bar 7-mm at higher magnification

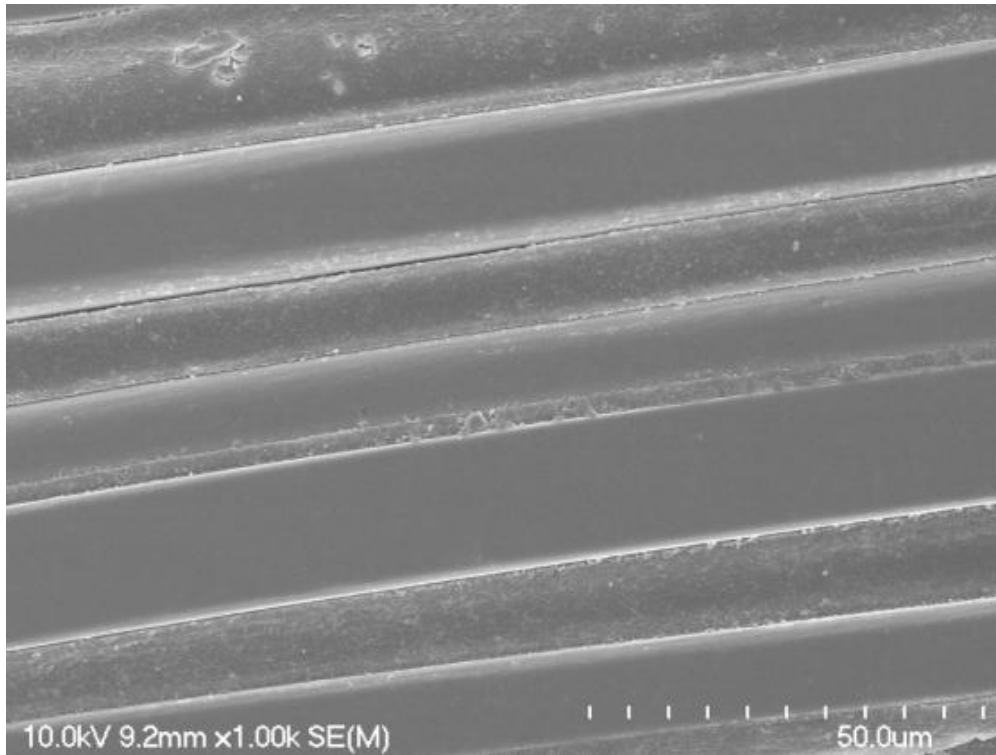


Figure 5.85: Longitudinal section of prestressing BFRP bar 7-mm at higher magnification

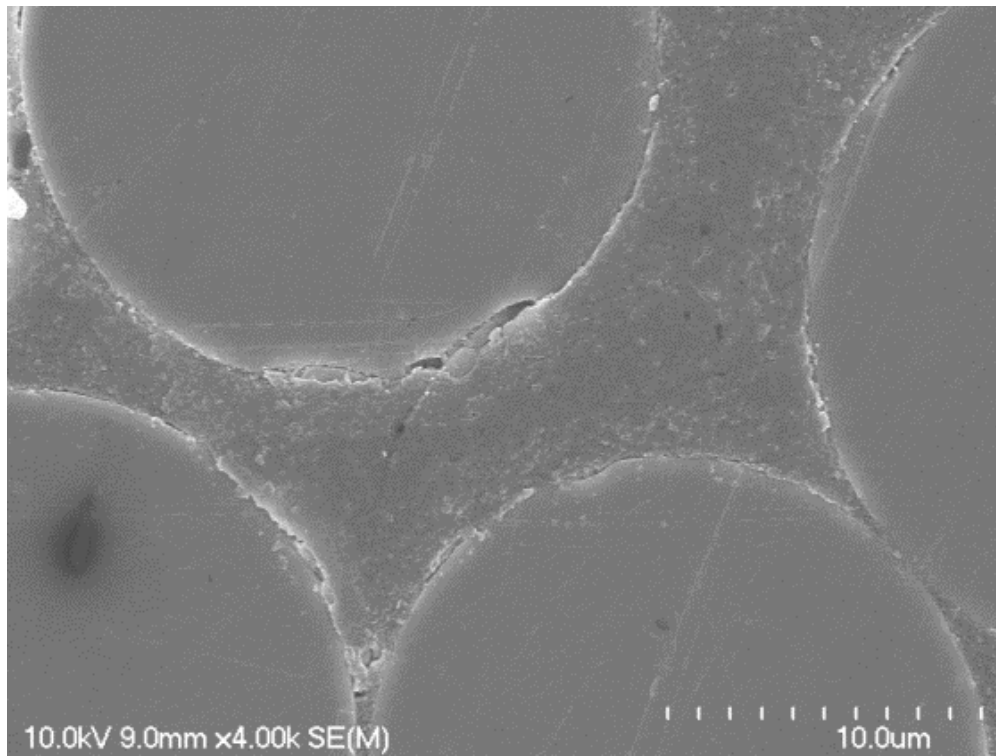


Figure 5.86: Cross-section of prestressing BFRP bar 7-mm at high magnification

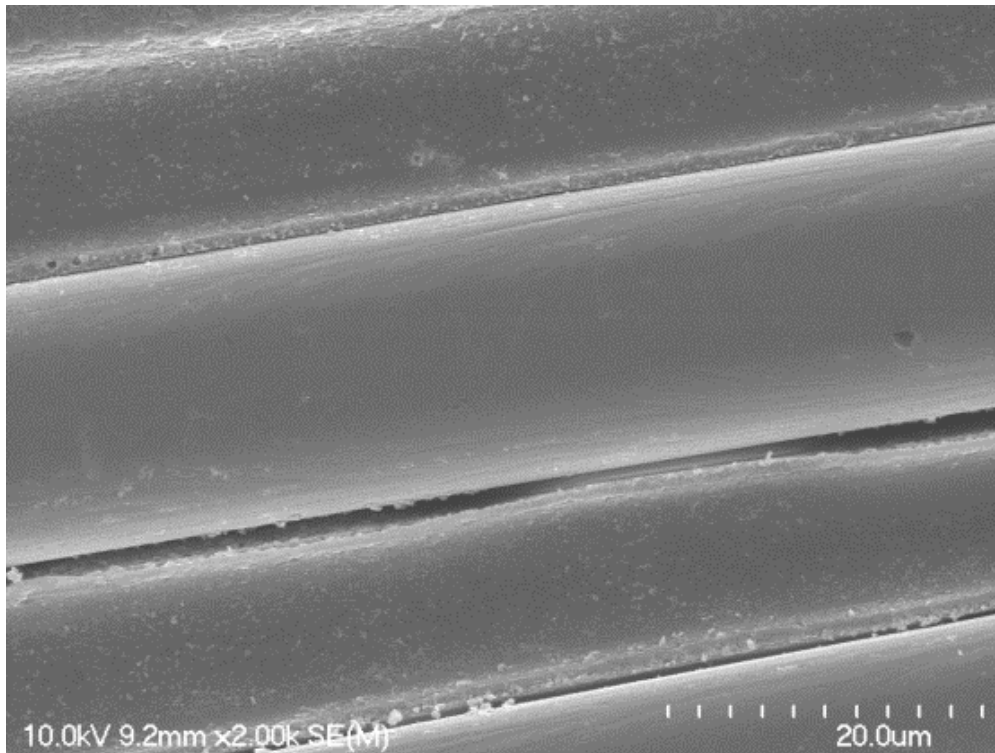


Figure 5.87: Longitudinal section of prestressing BFRP bar 7-mm at high magnification

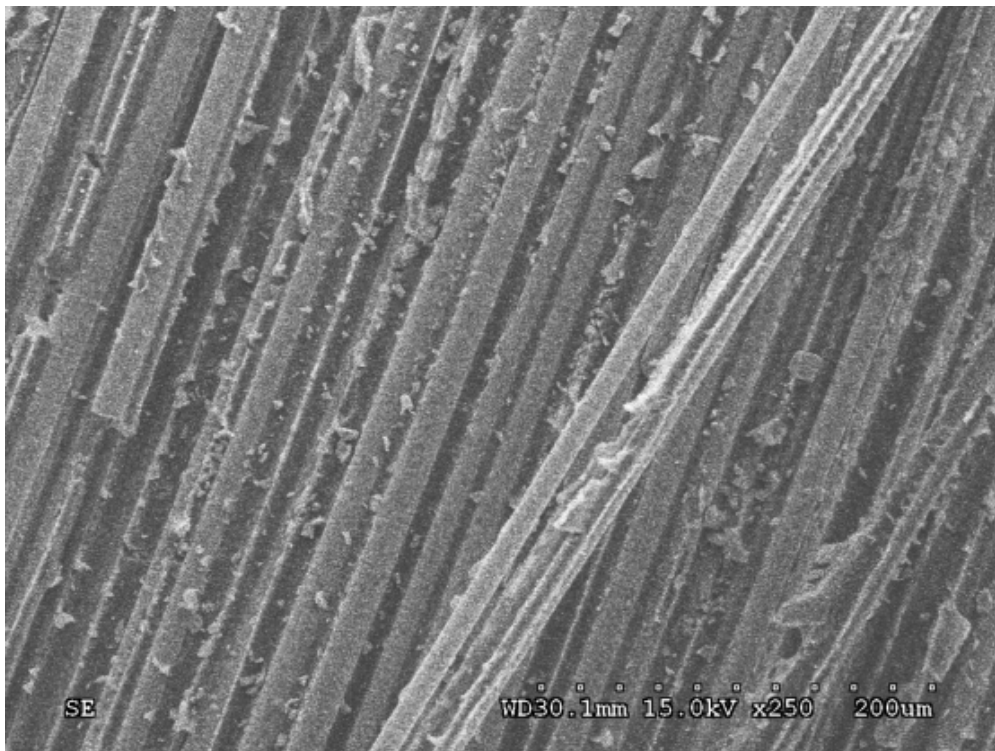


Figure 5.88: Fracture surface of prestressing BFRP bar 7-mm at low magnification

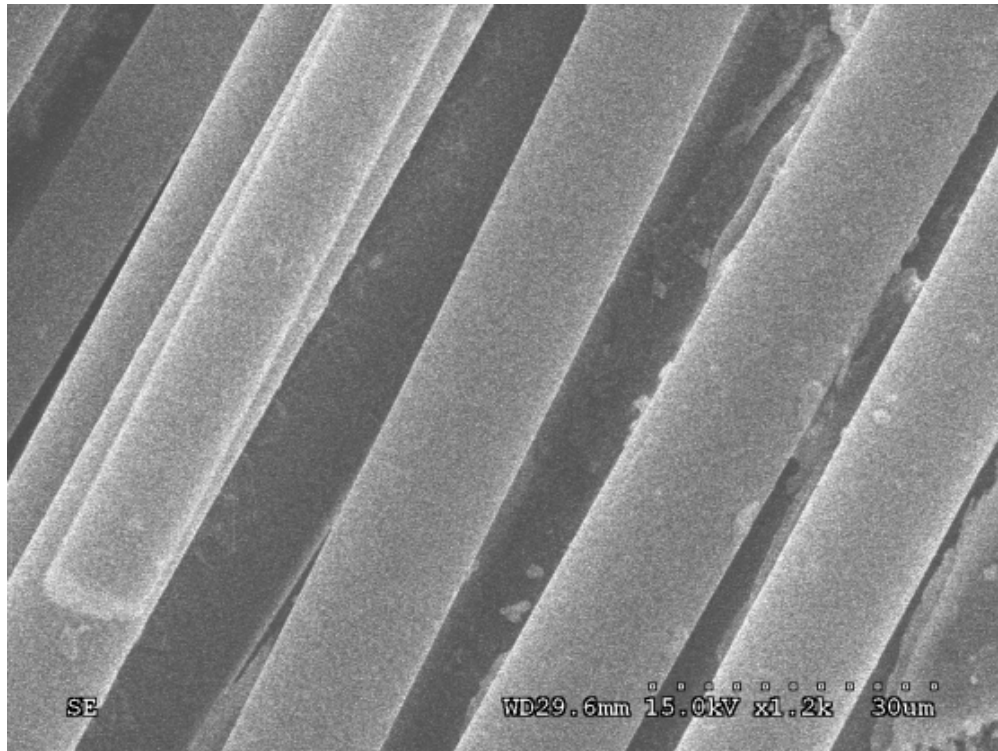


Figure 5.89: Fracture surface of prestressing BFRP bar 7-mm at higher magnification

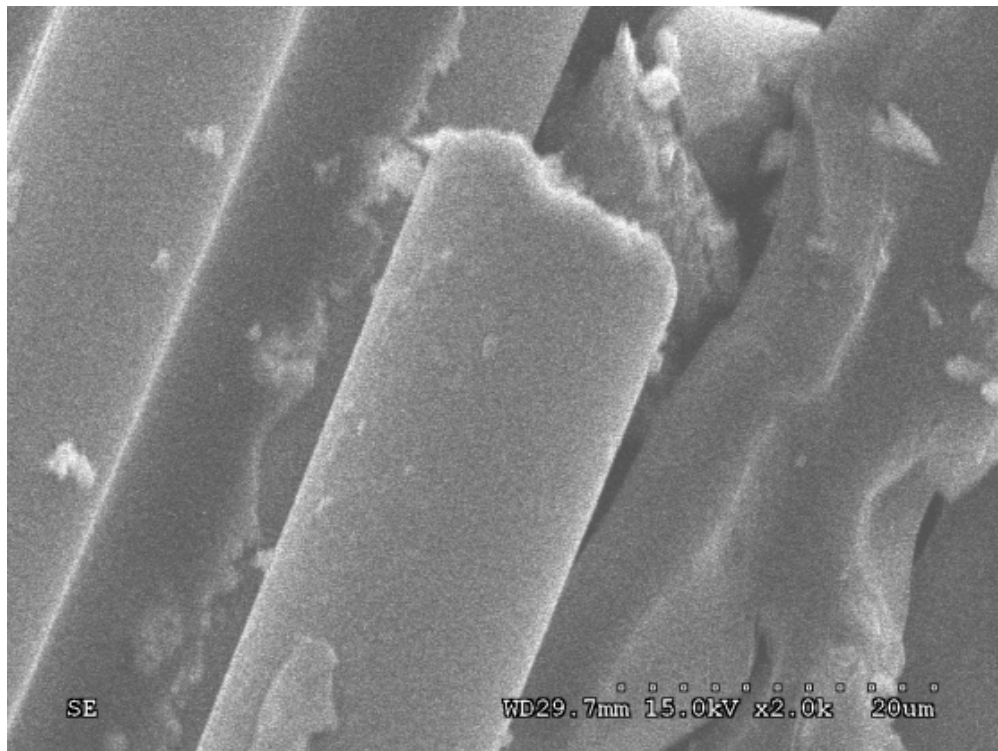


Figure 5.90: Fracture surface of prestressing BFRP bar 7-mm at high magnification

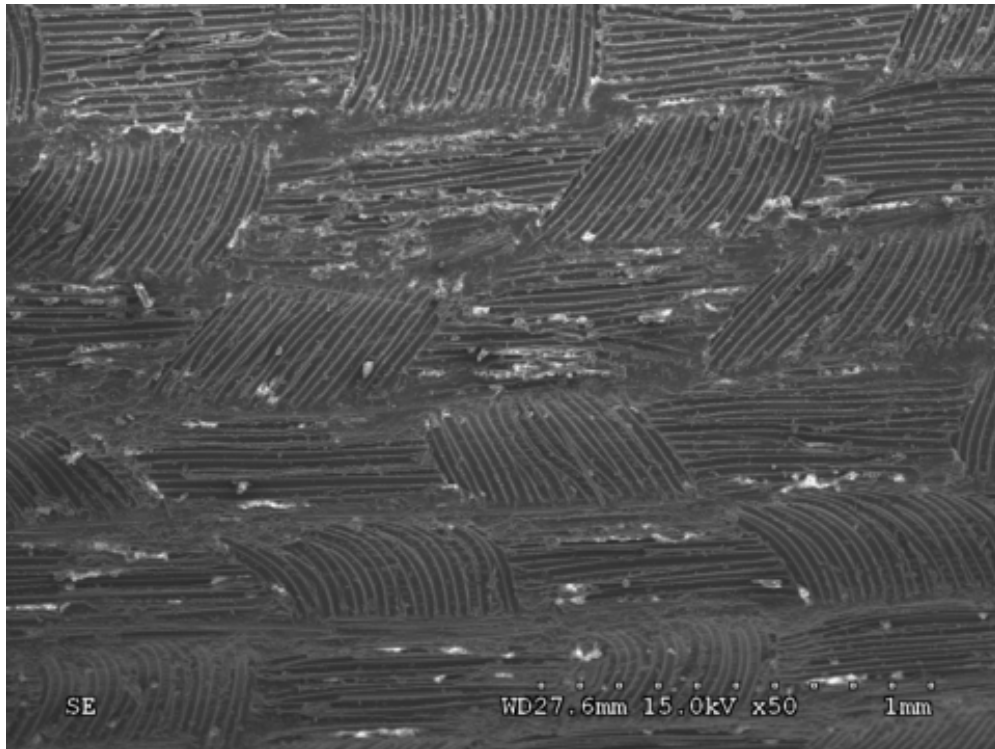


Figure 5.91: Surface of reference prestressing BFRP bar 7-mm at very low magnification

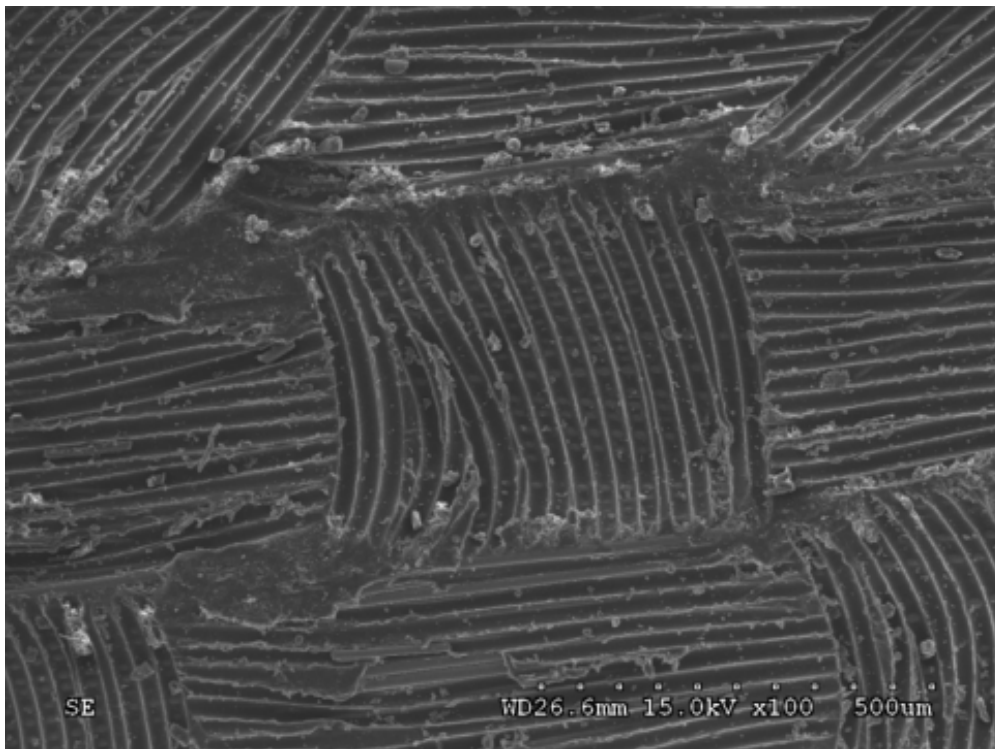


Figure 5.92: Surface of reference prestressing BFRP bar 7-mm at low magnification

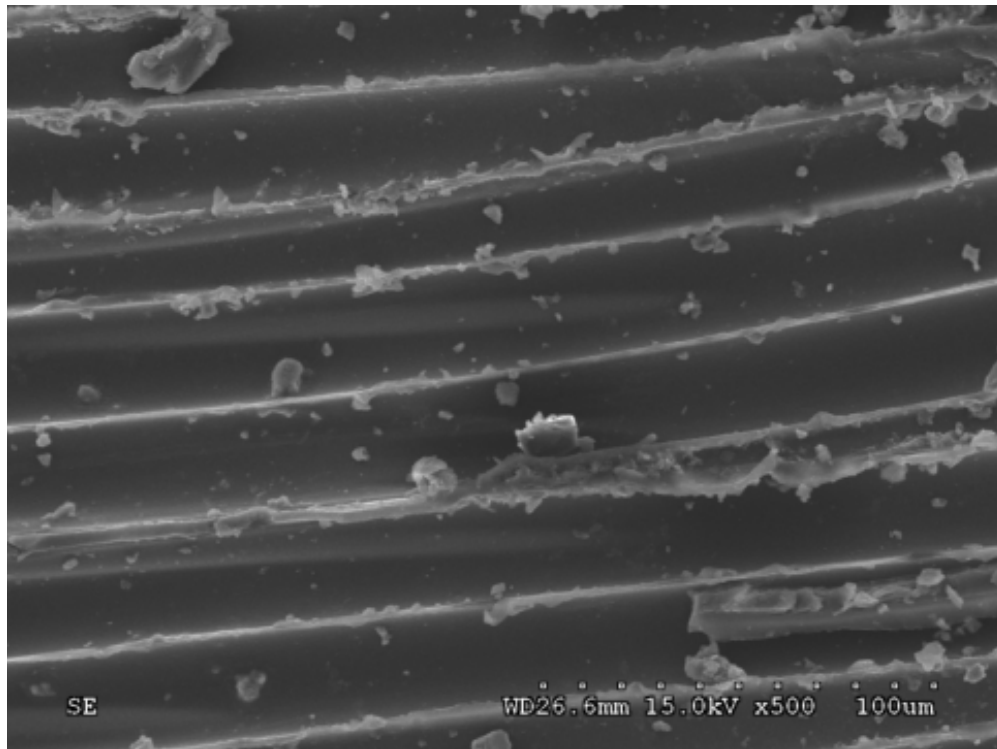


Figure 5.93: Surface of reference prestressing BFRP bar 7-mm at high magnification showing fibers

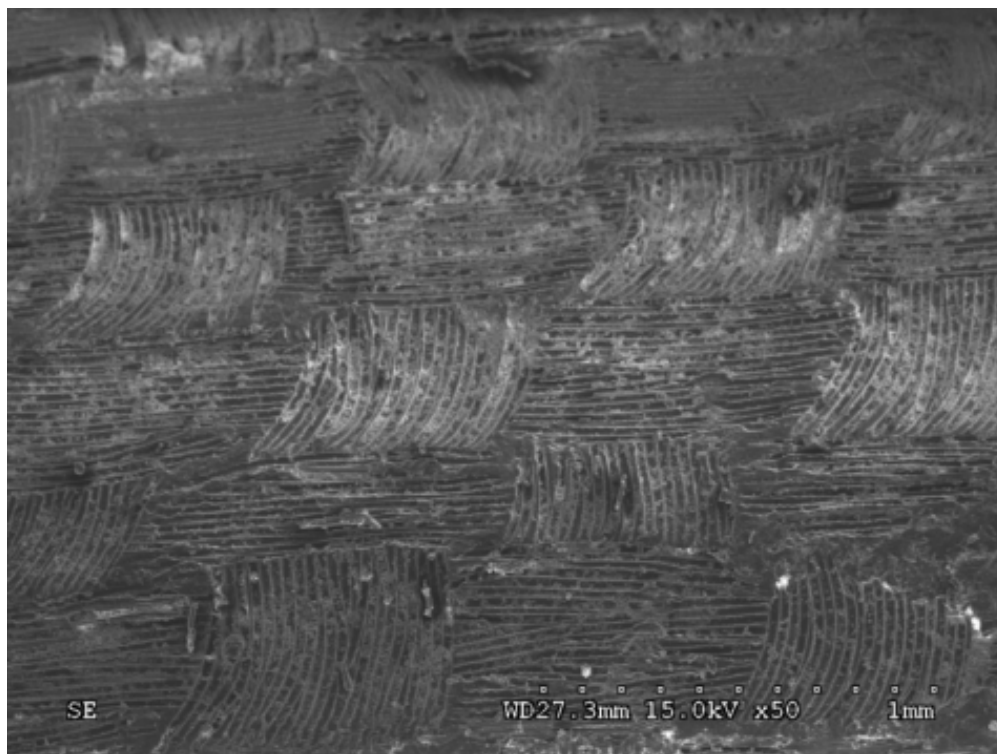


Figure 5.94: Surface of conditioned prestressing BFRP bar 7-mm at very low magnification

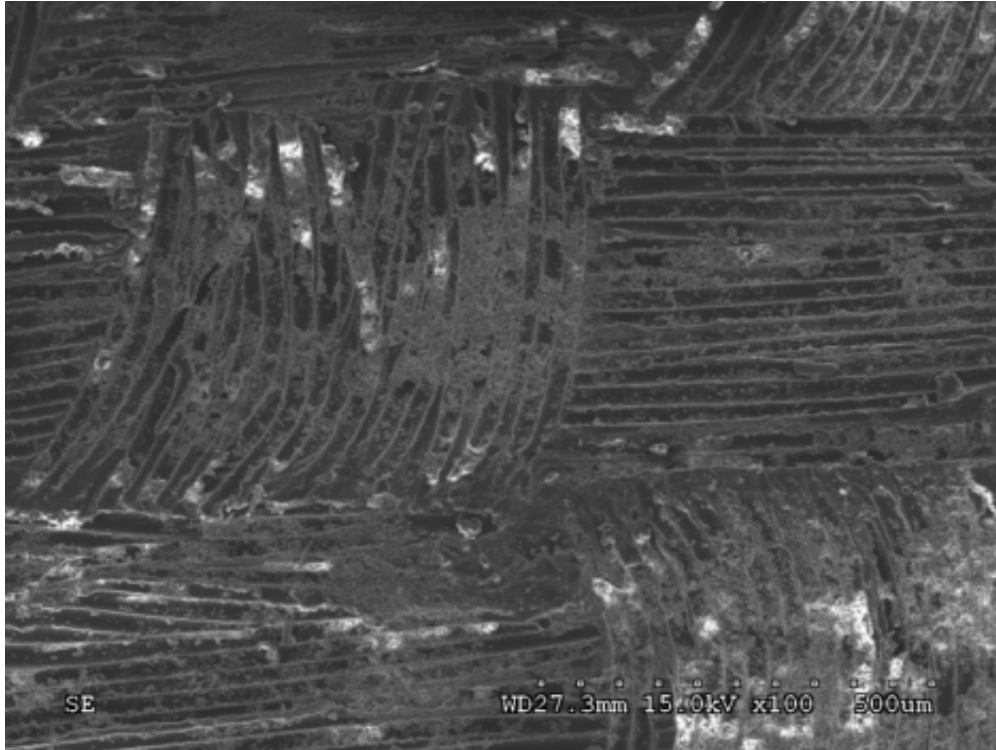


Figure 5.95: Surface of conditioned prestressing BFRP bar 7-mm at low magnification

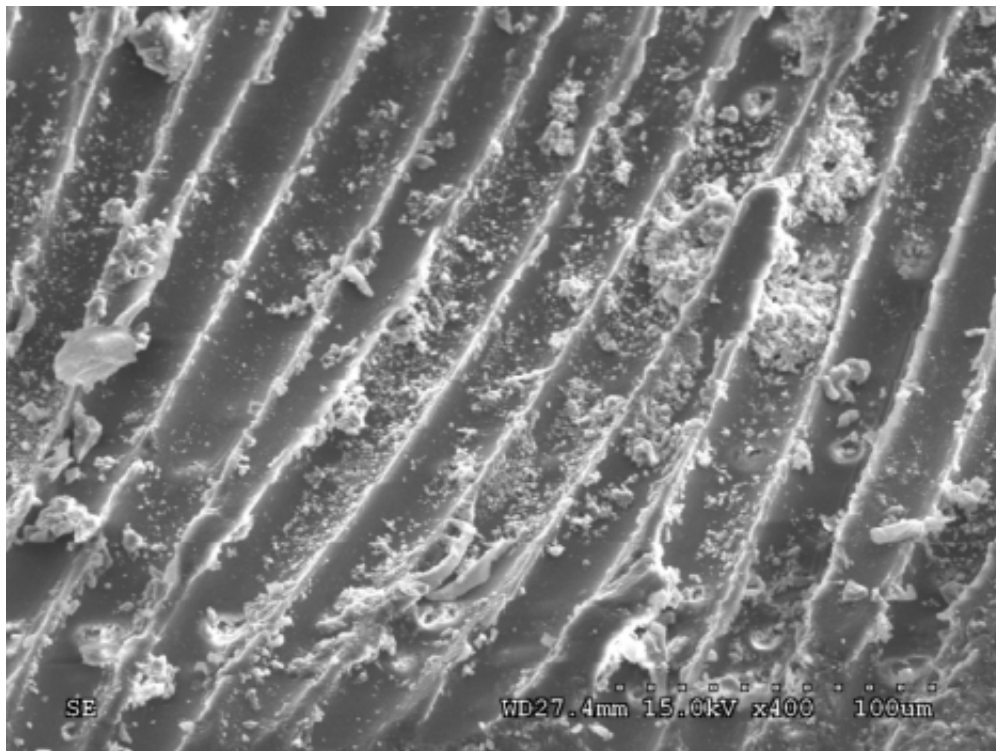


Figure 5.96: Surface of conditioned prestressing BFRP bar 7-mm at higher magnification showing fingerprint of fibers

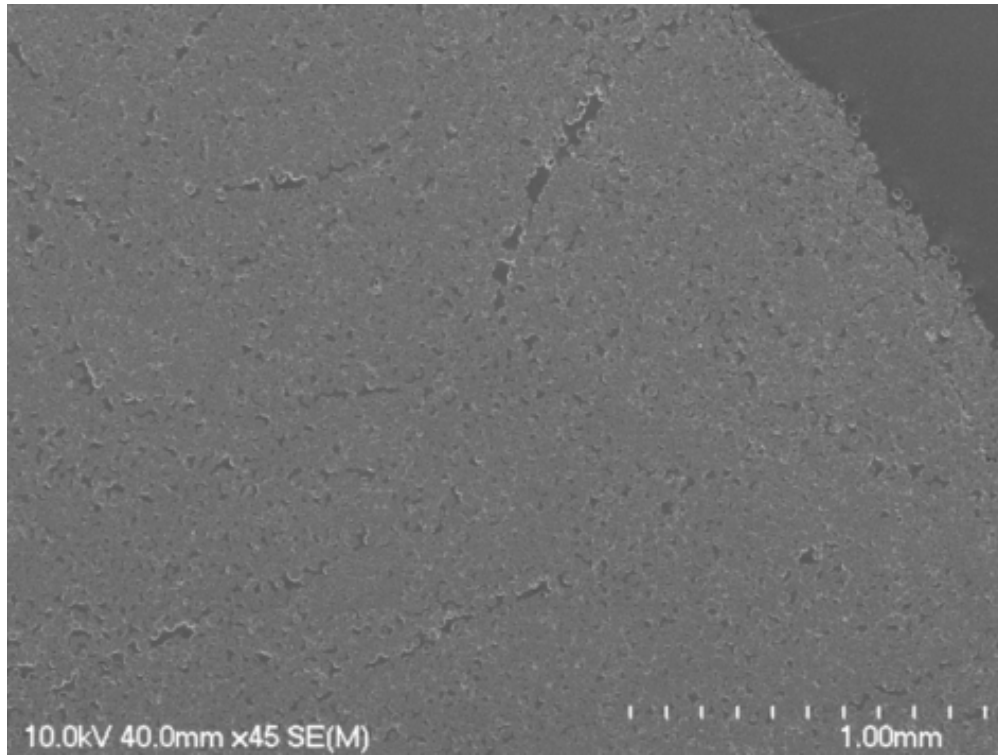


Figure 5.97: Cross-section of reference prestressing BFRP bar 7-mm at very low magnification

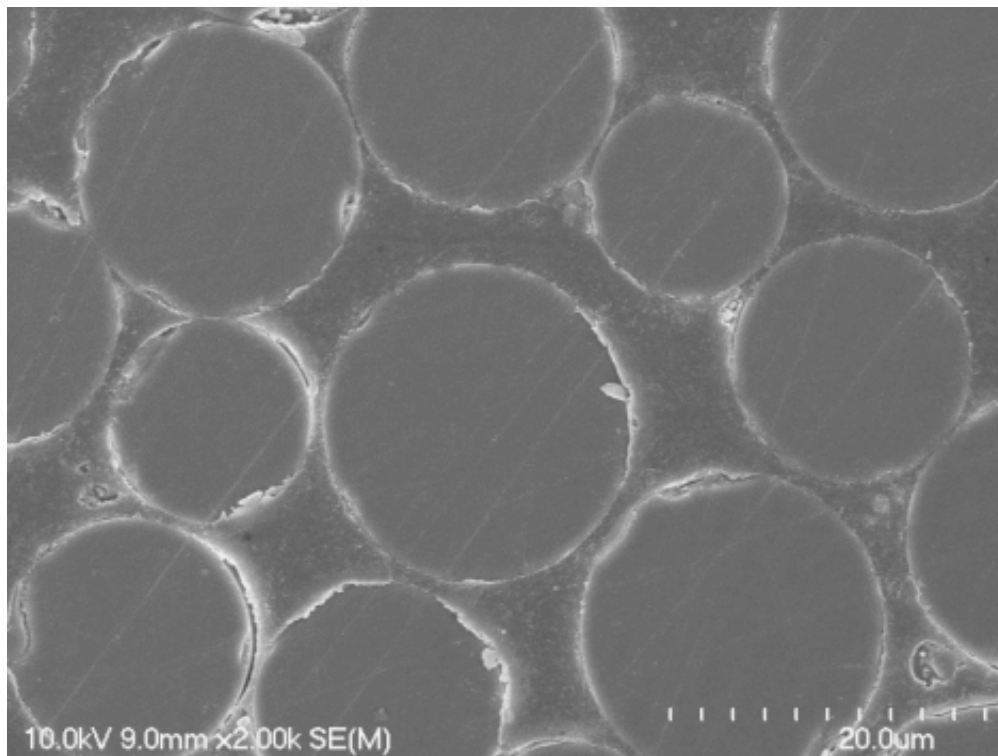


Figure 5.98: Cross-section of reference prestressing BFRP bar 7-mm at high magnification

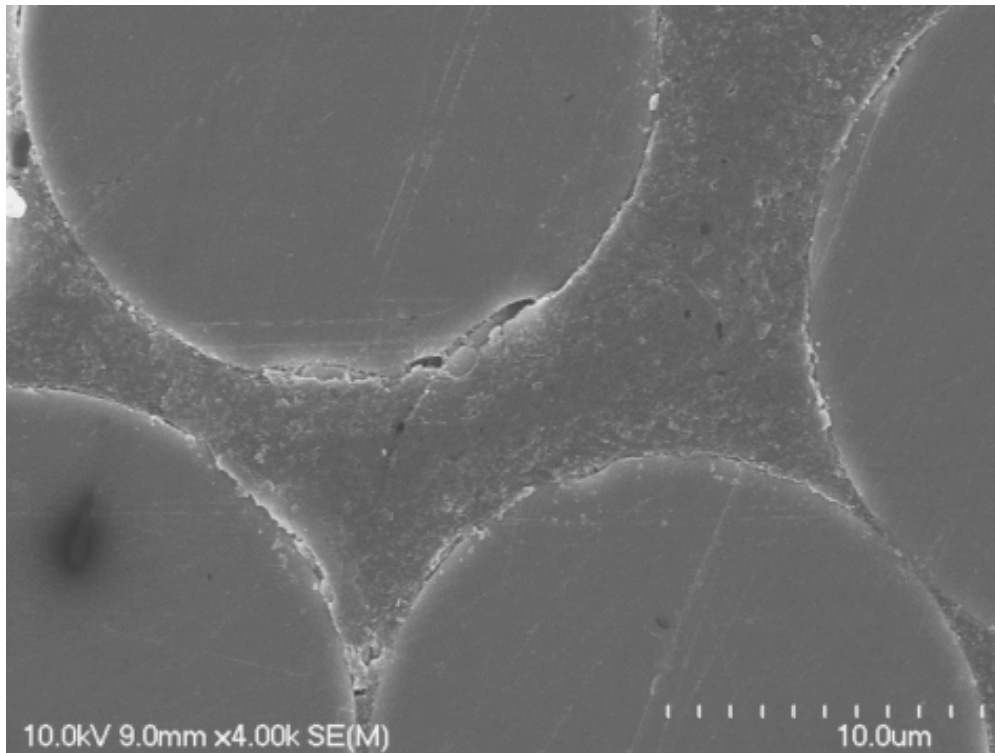


Figure 5.99: Cross-section of reference prestressing BFRP bar 7-mm at very high magnification

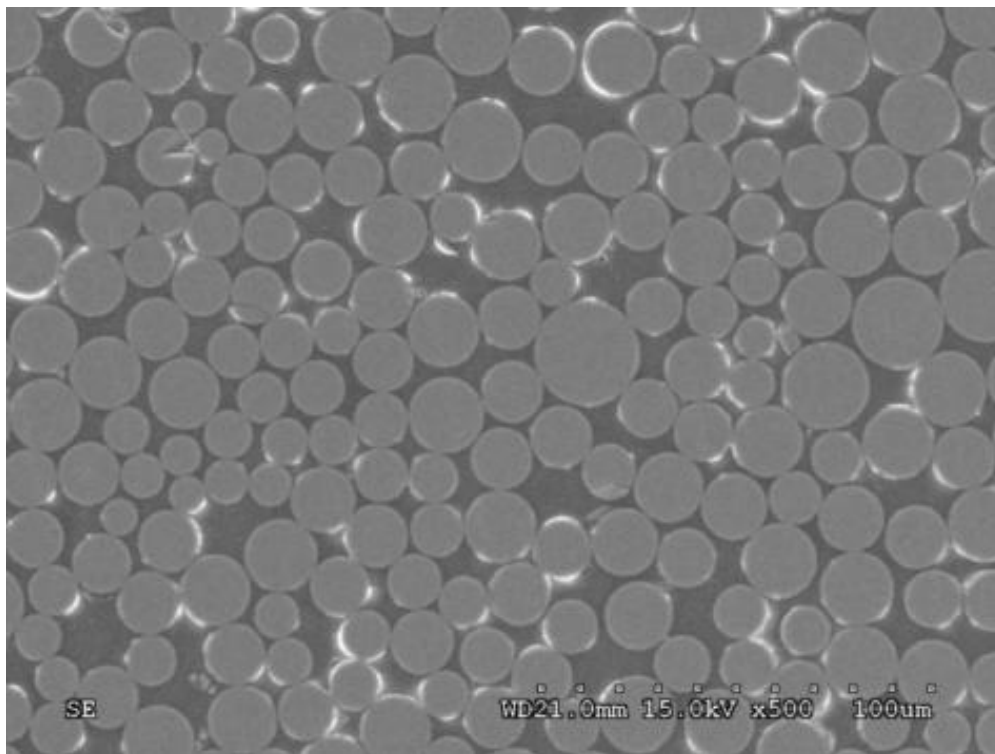


Figure 5.100: Cross-section of conditioned prestressing BFRP bar 7-mm at low magnification

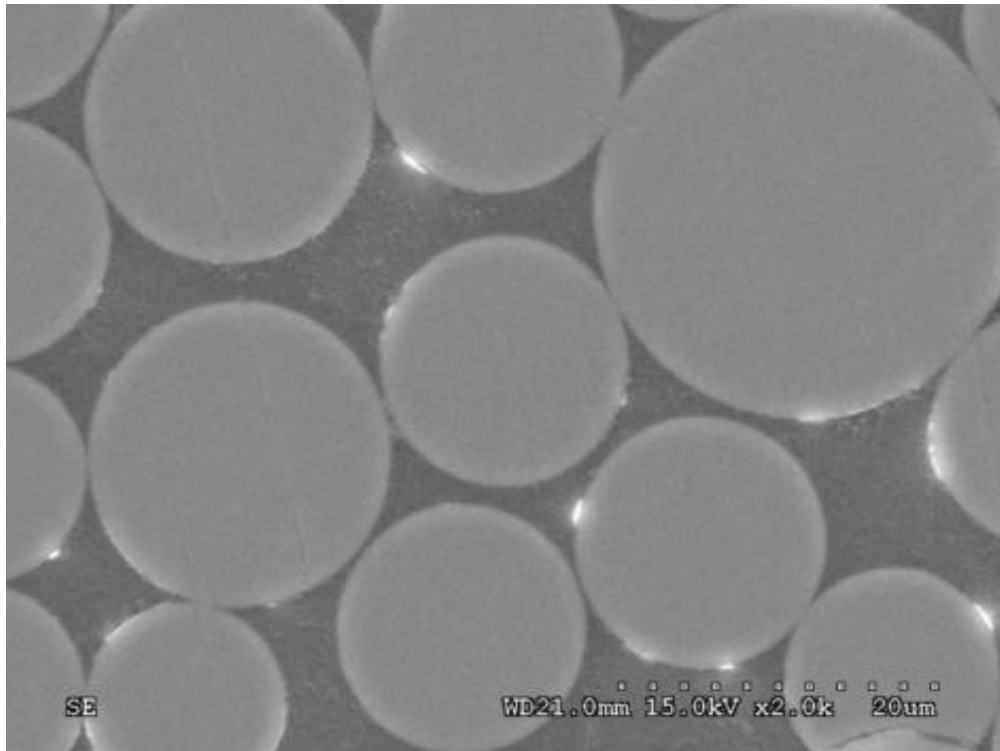


Figure 5.101: Cross-section of conditioned prestressing BFRP bar 7-mm at high magnification

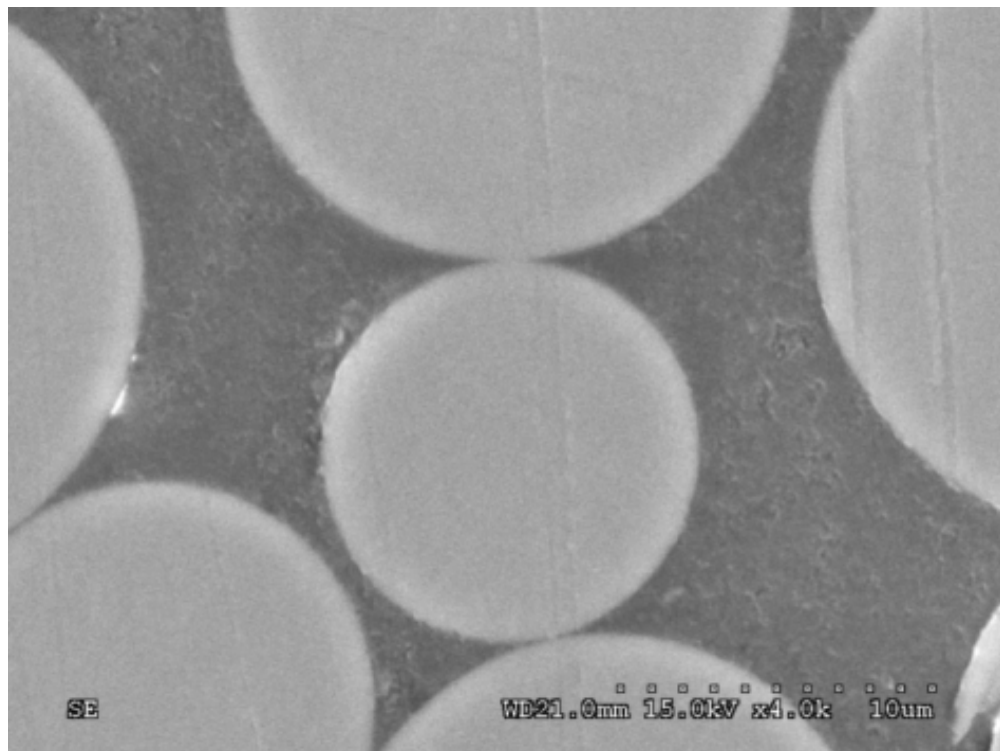


Figure 5.102: Cross-section of conditioned prestressing BFRP bar 7-mm at very high magnification

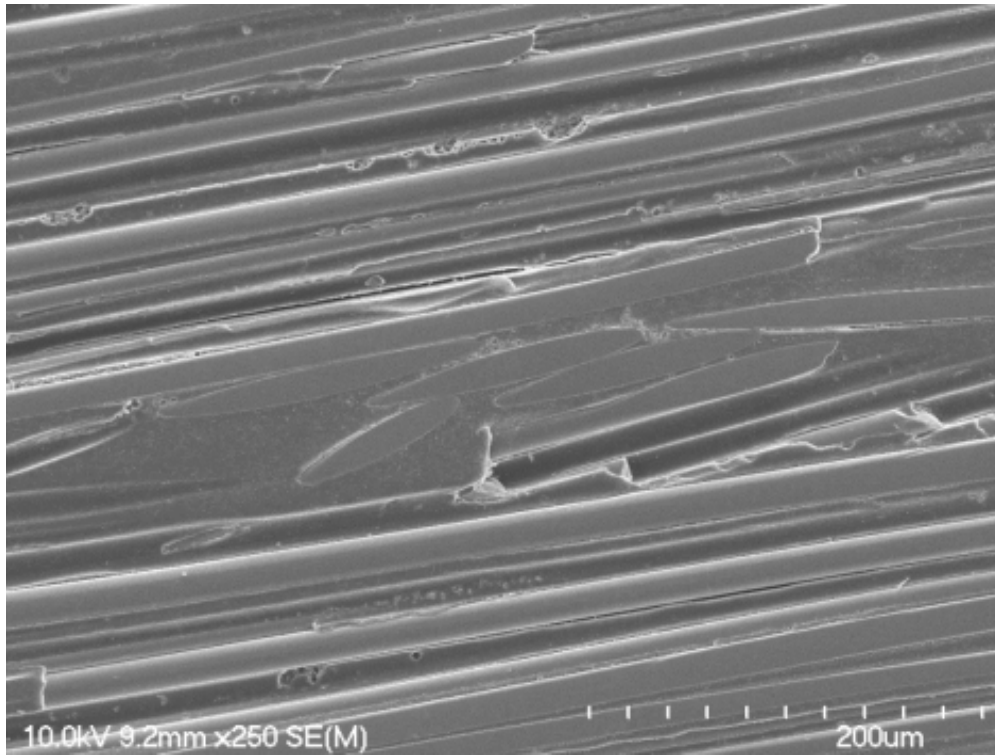


Figure 5.103: Longitudinal section of reference prestressing BFRP bar 7-mm at low magnification

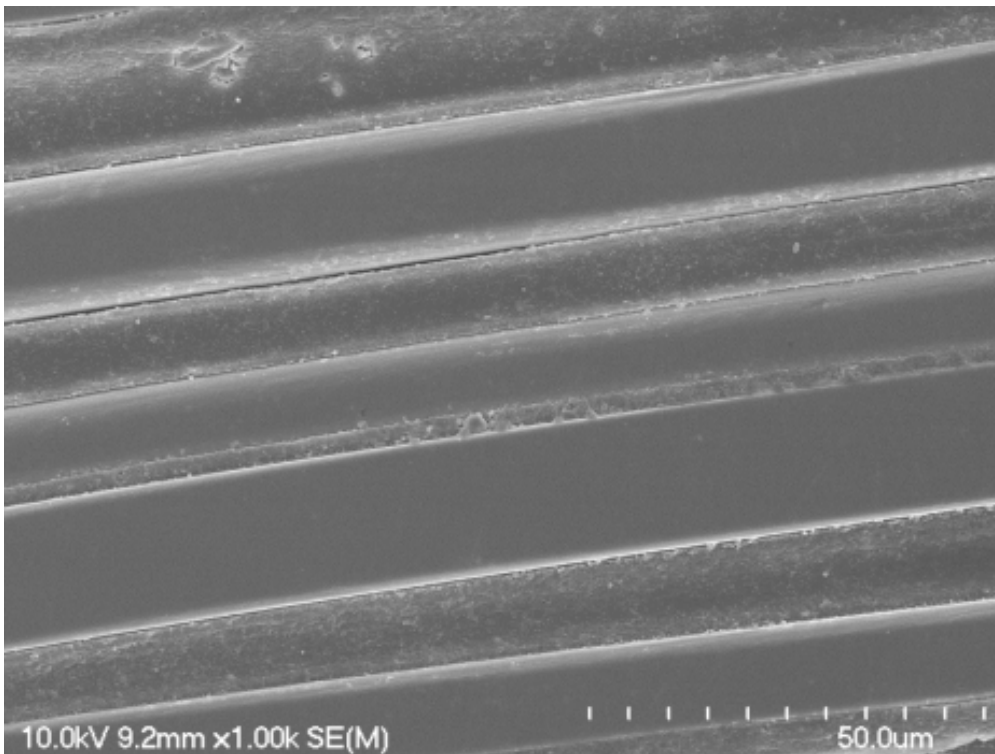


Figure 5.104: Longitudinal section of reference prestressing BFRP bar 7-mm at high magnification

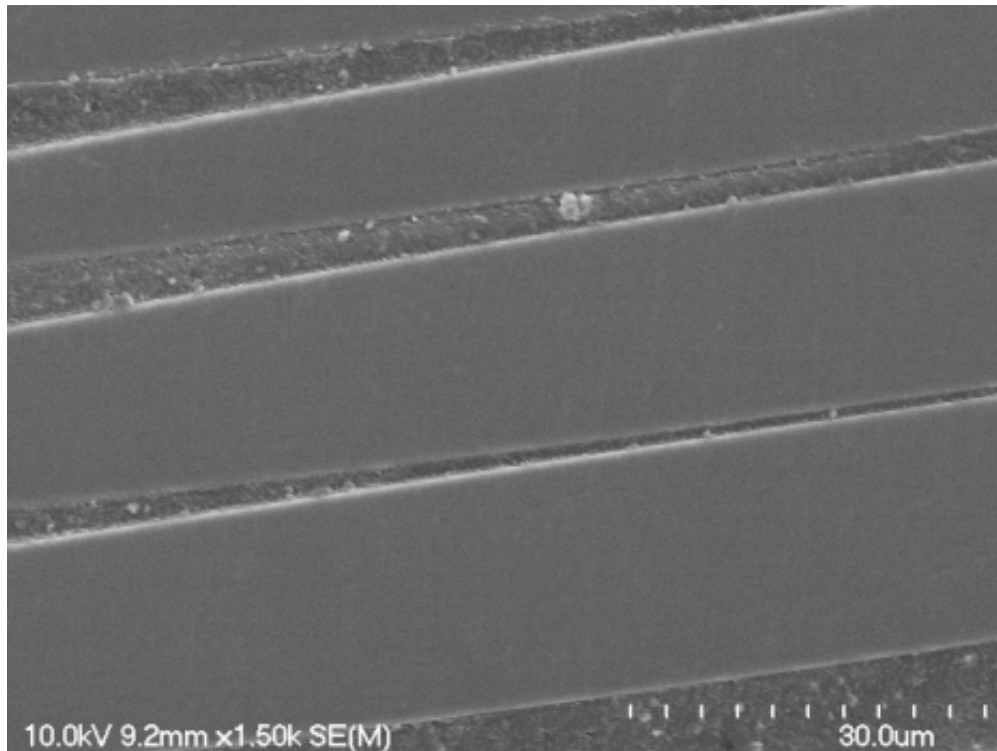


Figure 5.105: Longitudinal section of reference prestressing BFRP bar 7-mm at very high magnification

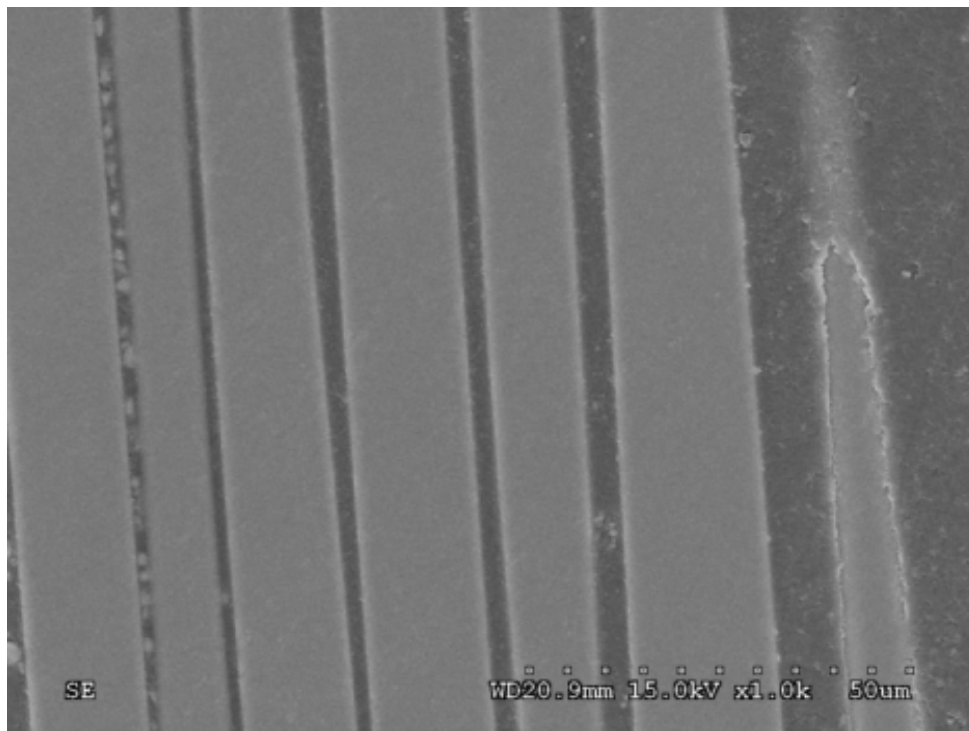


Figure 5.106: Longitudinal section of conditioned prestressing BFRP bar 7-mm at high magnification

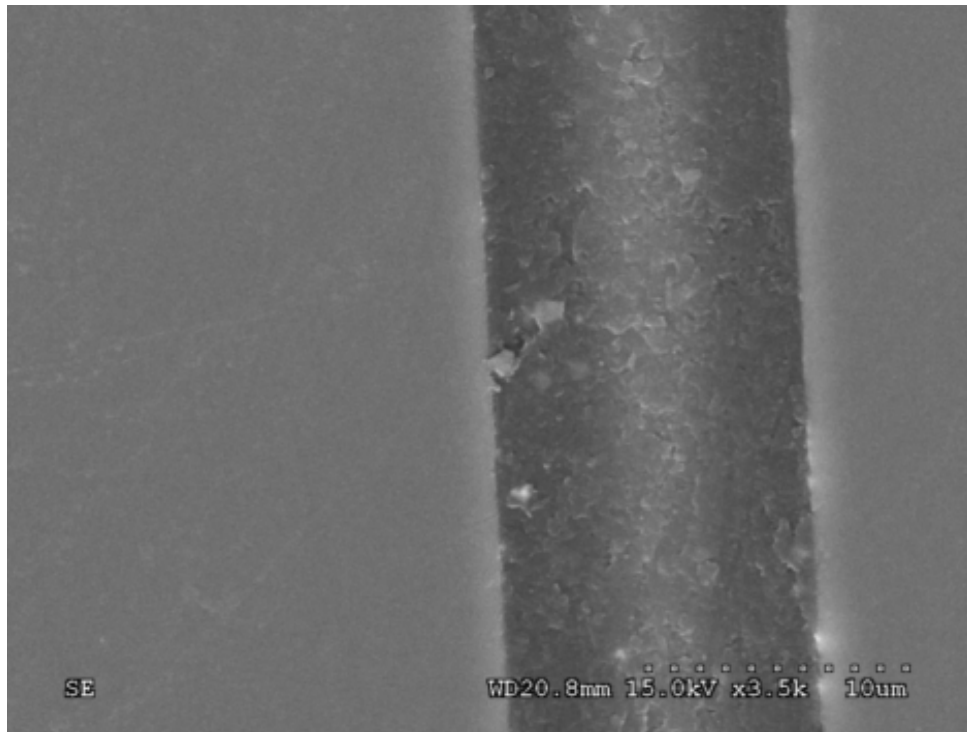


Figure 5.107: Longitudinal section of conditioned prestressing BFRP bar 7-mm at very high magnification

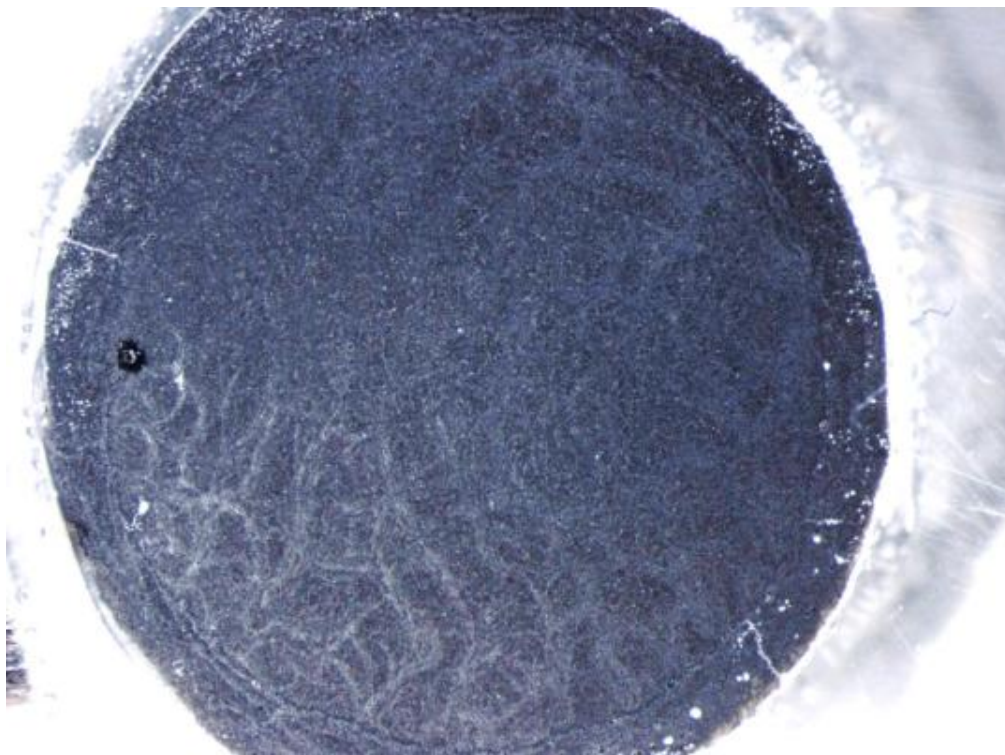


Figure 5.108: BASA BFRP bars 8-mm (code 43) at low magnification

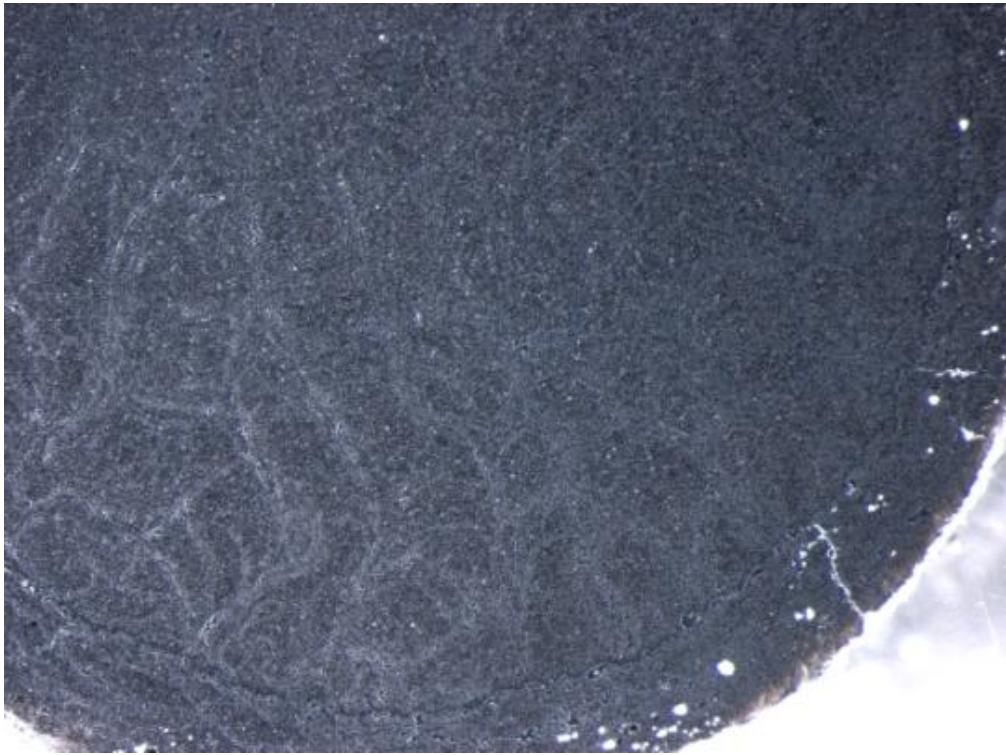


Figure 5.109: BASA BFRP bars 8-mm (code 43) at low magnification

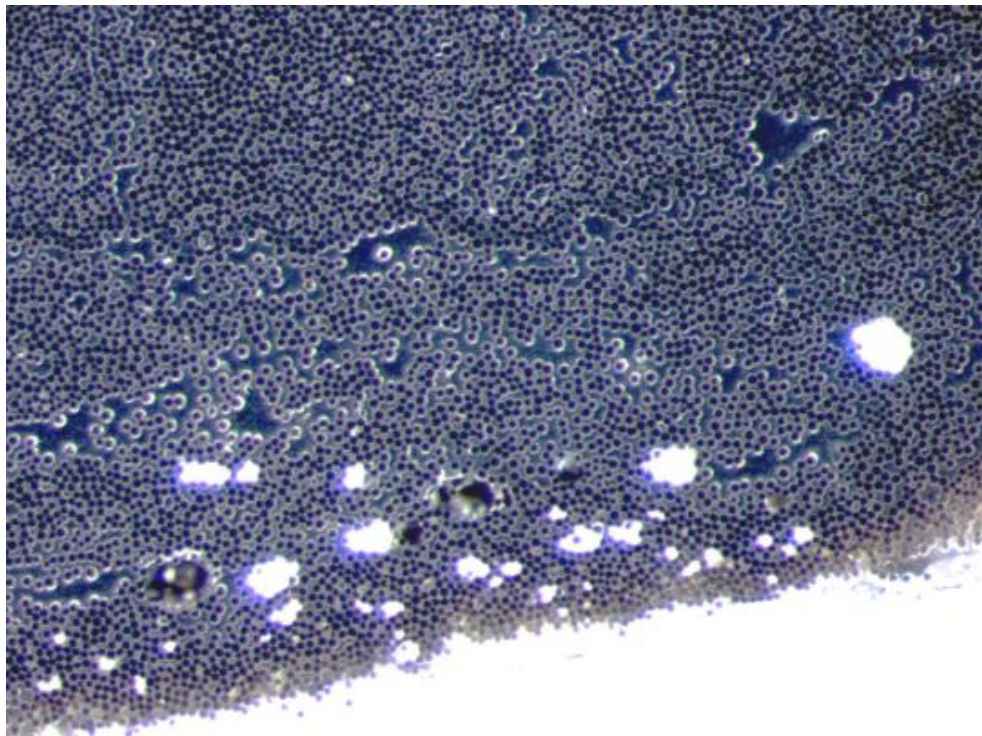


Figure 5.110: BASA BFRP bars 8-mm (code 43) at high magnification showing the coating

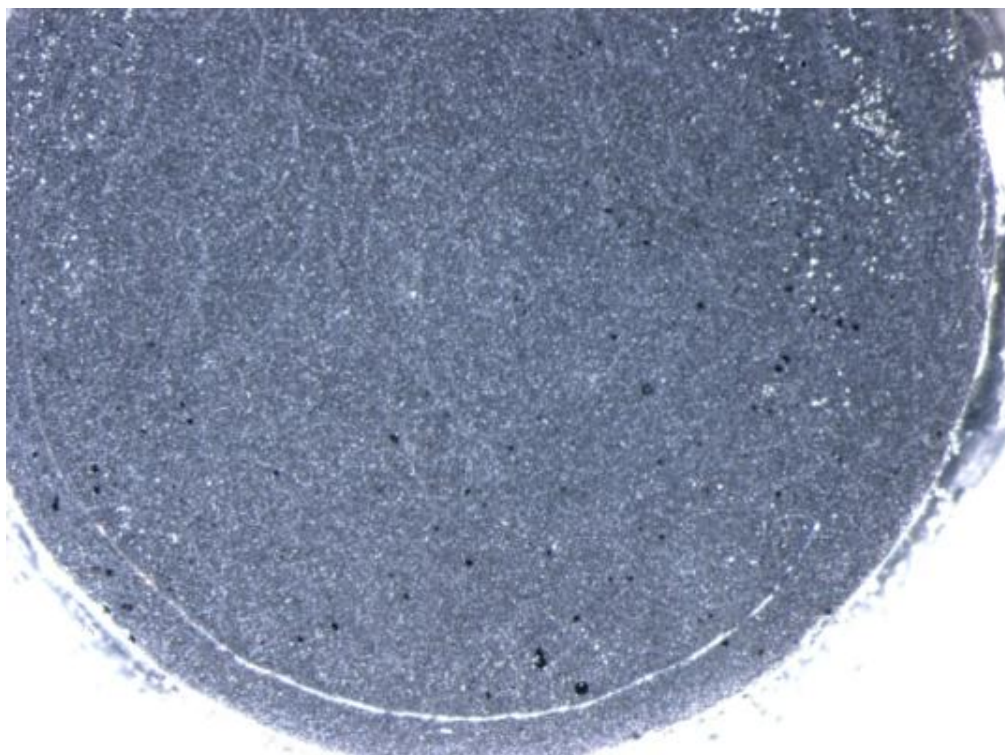


Figure 5.111: BASA BFRP bars 8-mm (code 51) at low magnification

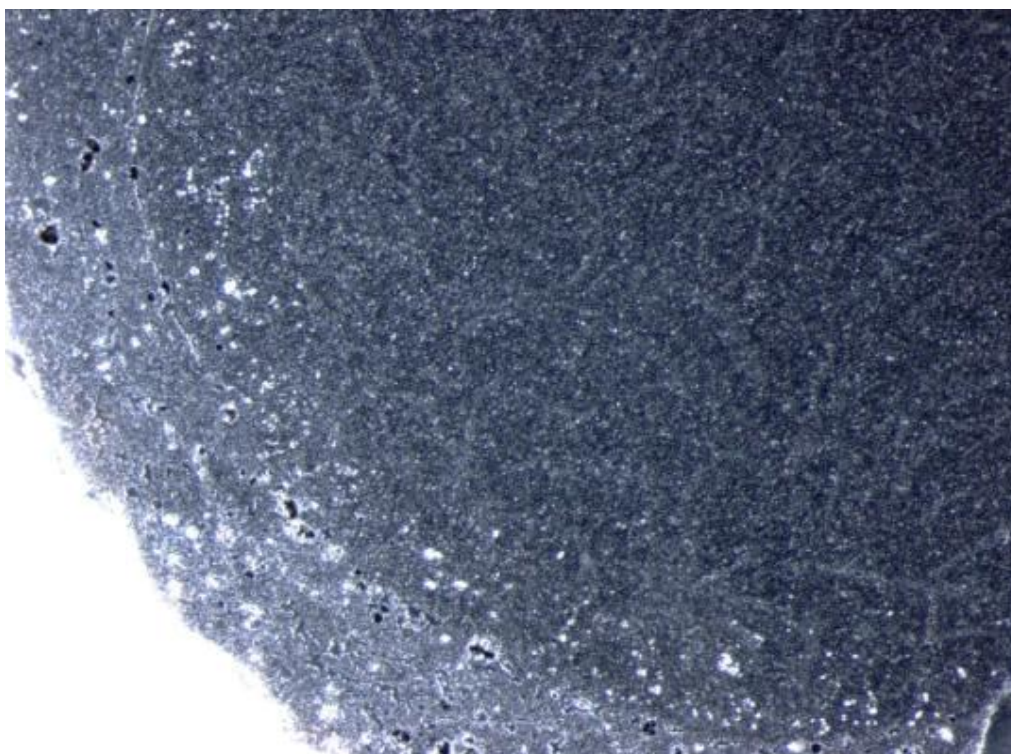


Figure 5.112: BASA BFRP bars 8-mm (code 51) at higher magnification

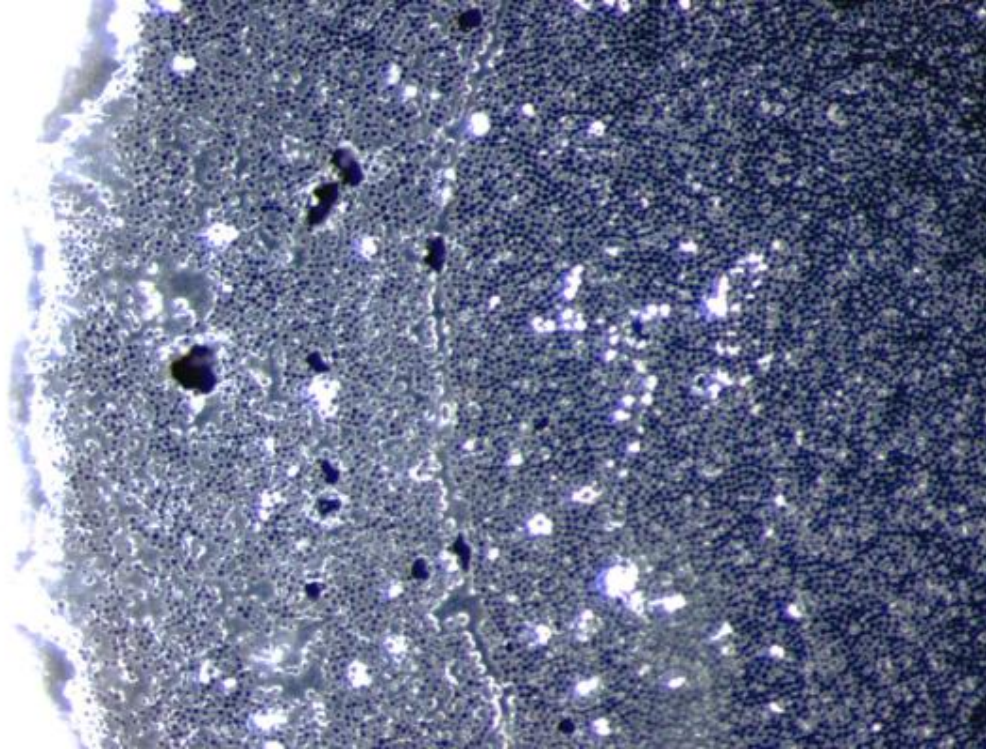


Figure 5.113 : BASA BFRP bars 8-mm (code 51) at high magnification showing pores in the coating

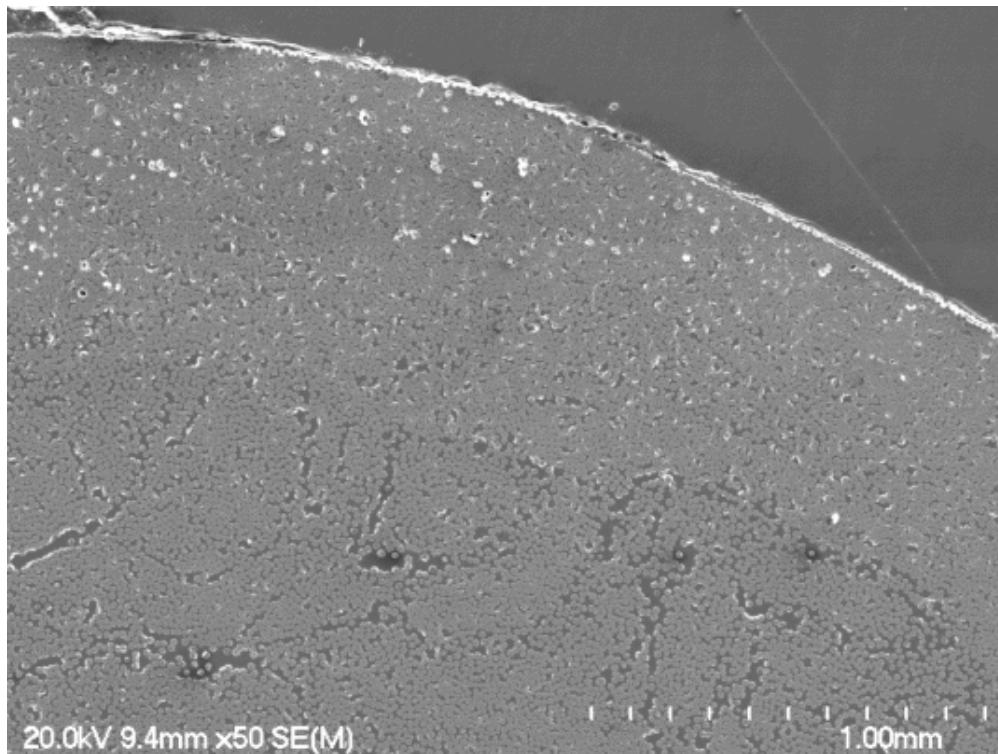


Figure 5.114: SEM micrograph of the cross-section of BASA BFRP bars 8-mm (code 43) at low magnification

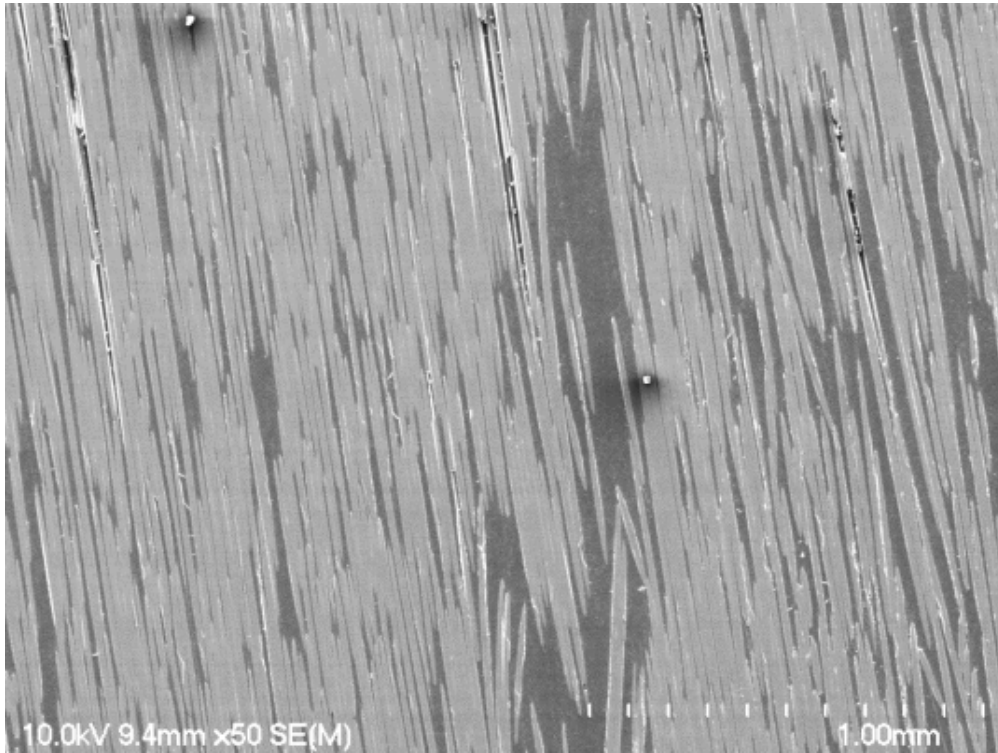


Figure 5.115: SEM micrograph of the longitudinal section of BASA BFRP bars 8-mm (code 43) at low magnification

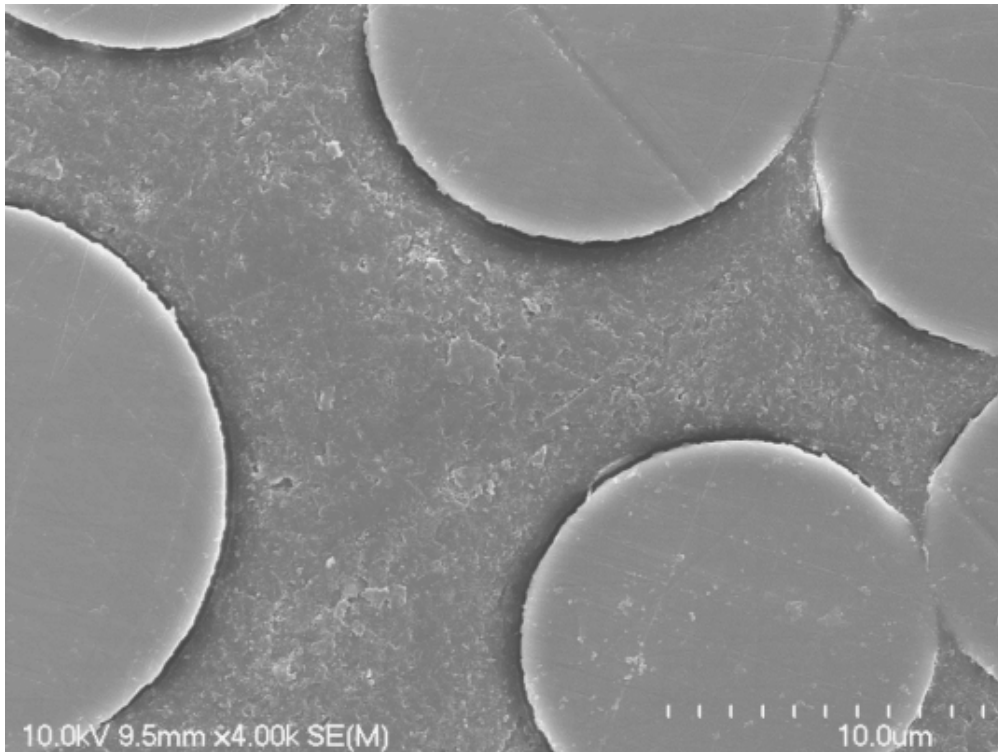


Figure 5.116: SEM micrograph of BASA BFRP bars 8-mm (code 43) at high magnification

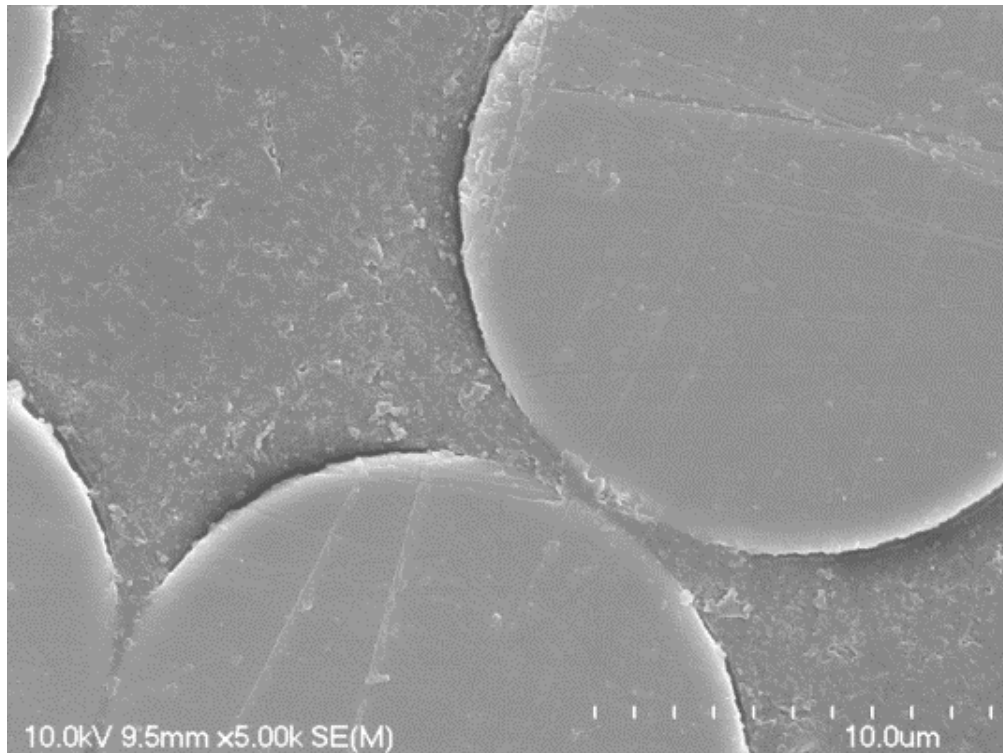


Figure 5.117: SEM micrograph of BASA BFRP bars 8-mm (code 43) at high magnification

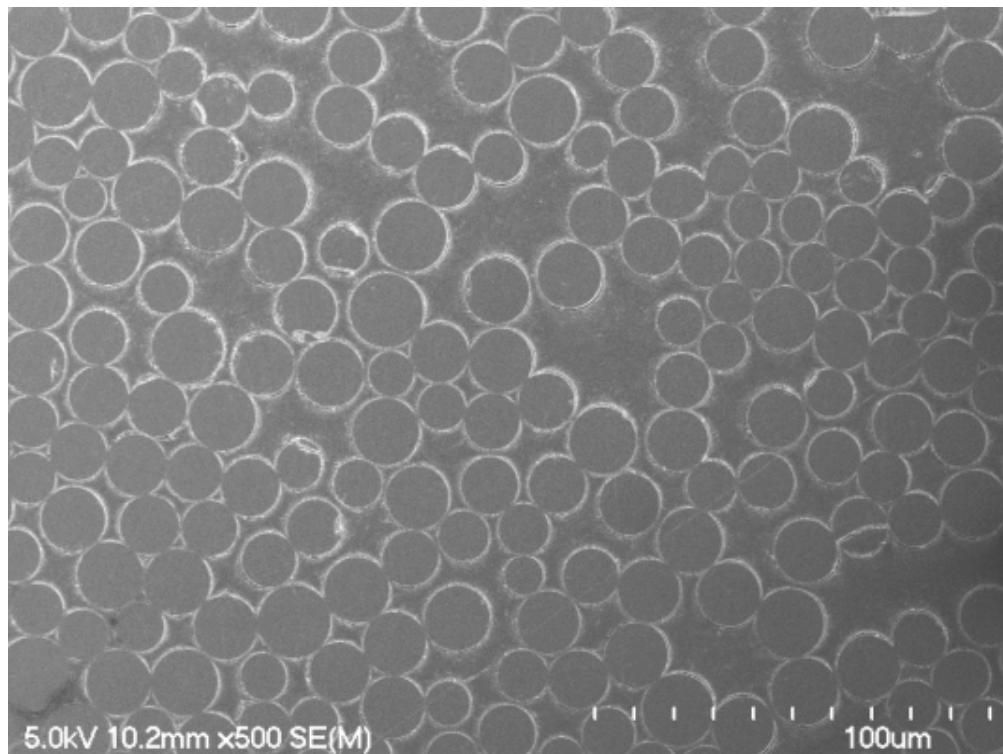


Figure 5.118: Cross-section of BASA BFRP bars 8-mm (code 43) after conditioning of 1 month (720 hrs.) 22°C

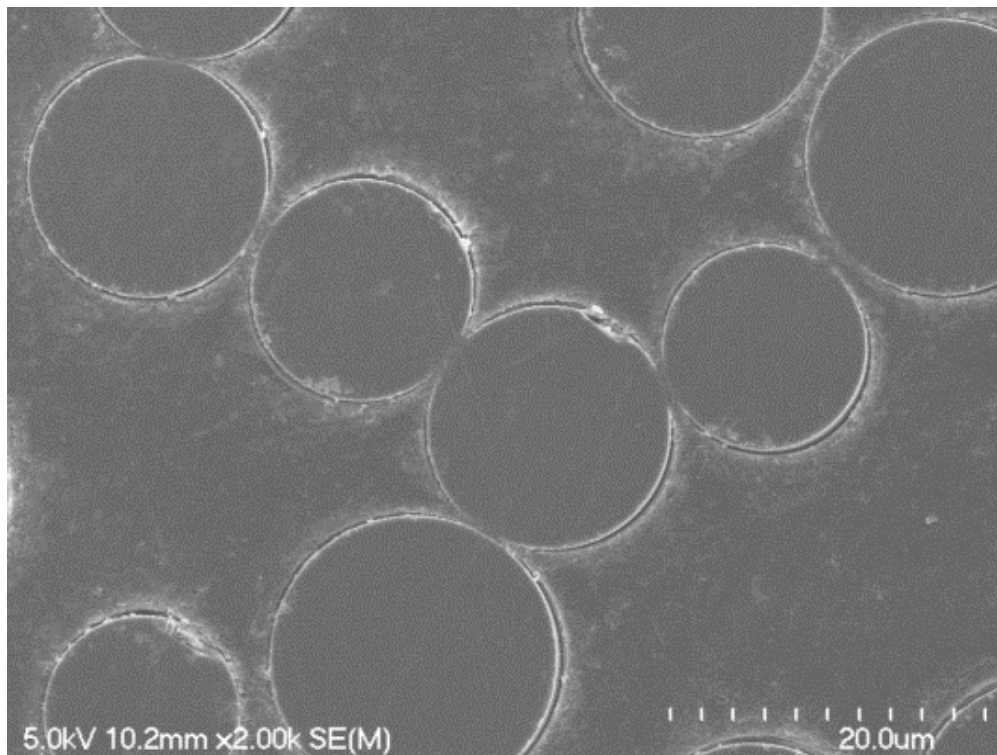


Figure 5.119: BASA BFRP bars 8-mm (code 43) after conditioning of 1 month (720 hrs.) at 22°C at high magnification

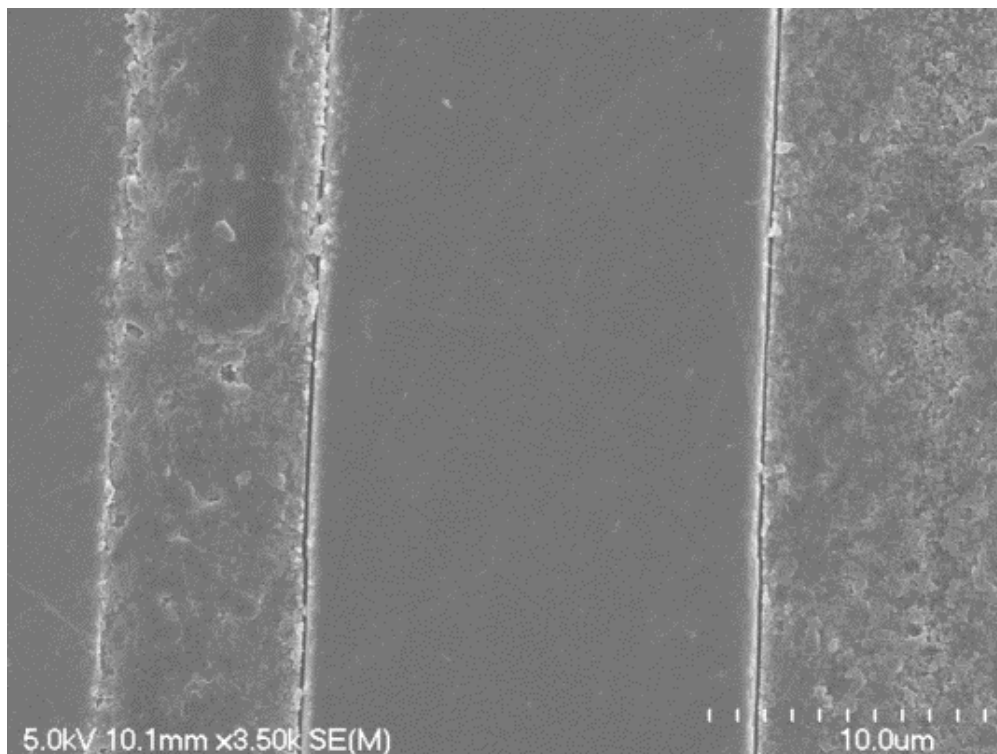


Figure 5.120: Longitudinal section of BASA BFRP bars 8-mm (code 43) after conditioning of 1 month (720 hrs.) at 22°C

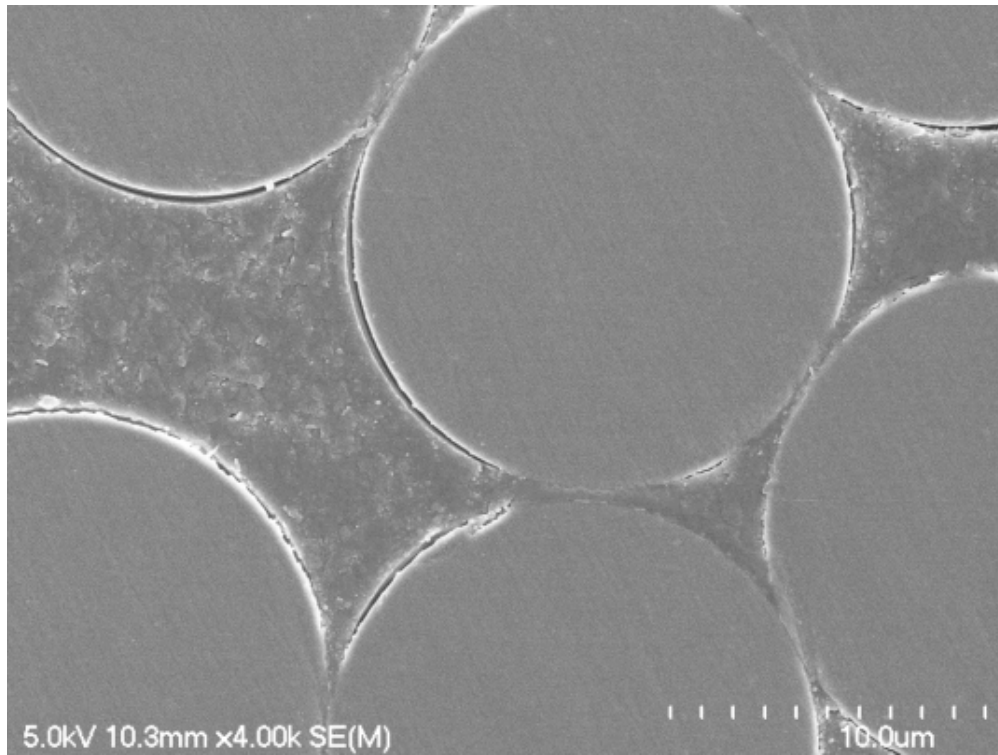


Figure 5.121: BASA BFRP bars 8-mm (code 43) after conditioning of 1 month (720 hrs.) at 60°C at high magnification

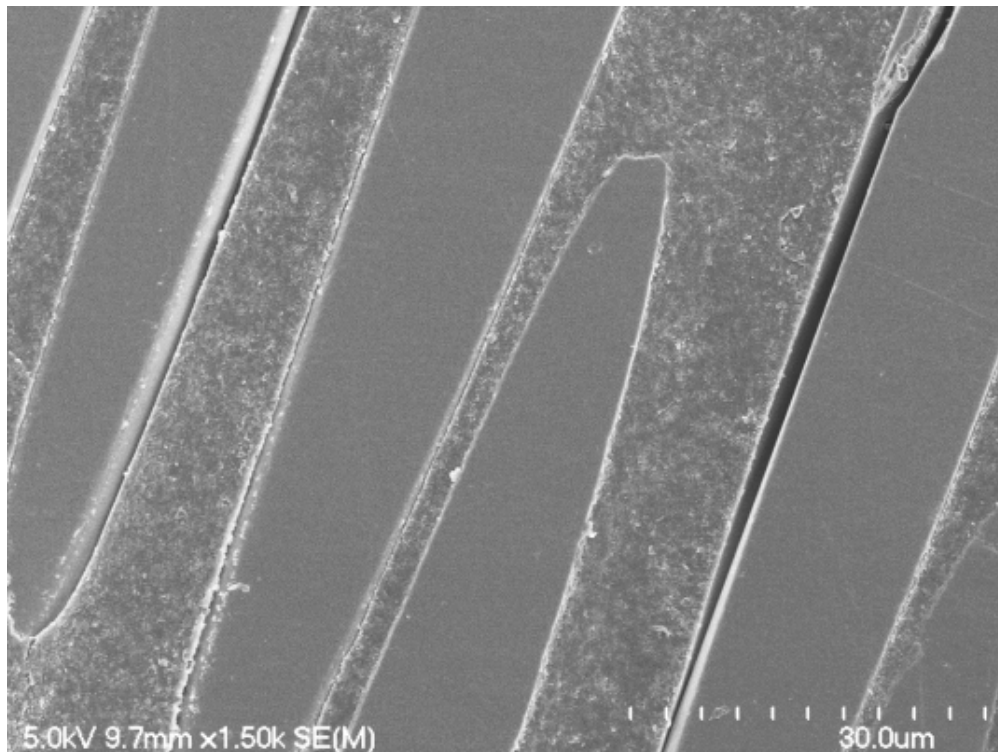


Figure 5.122: Longitudinal section of BASA BFRP bars 8-mm (code 43) after conditioning of 1 month (720 hrs.) at 60°C

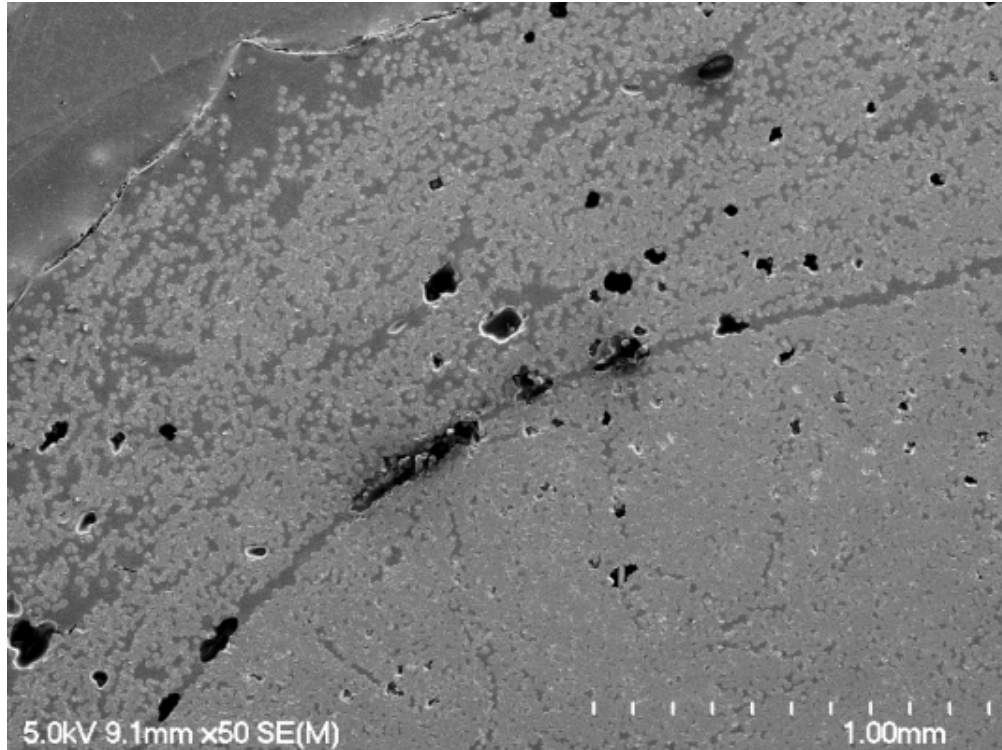


Figure 5.123: SEM micrograph of the cross-section of BASA BFRP bars 8-mm (code 51) at low magnification

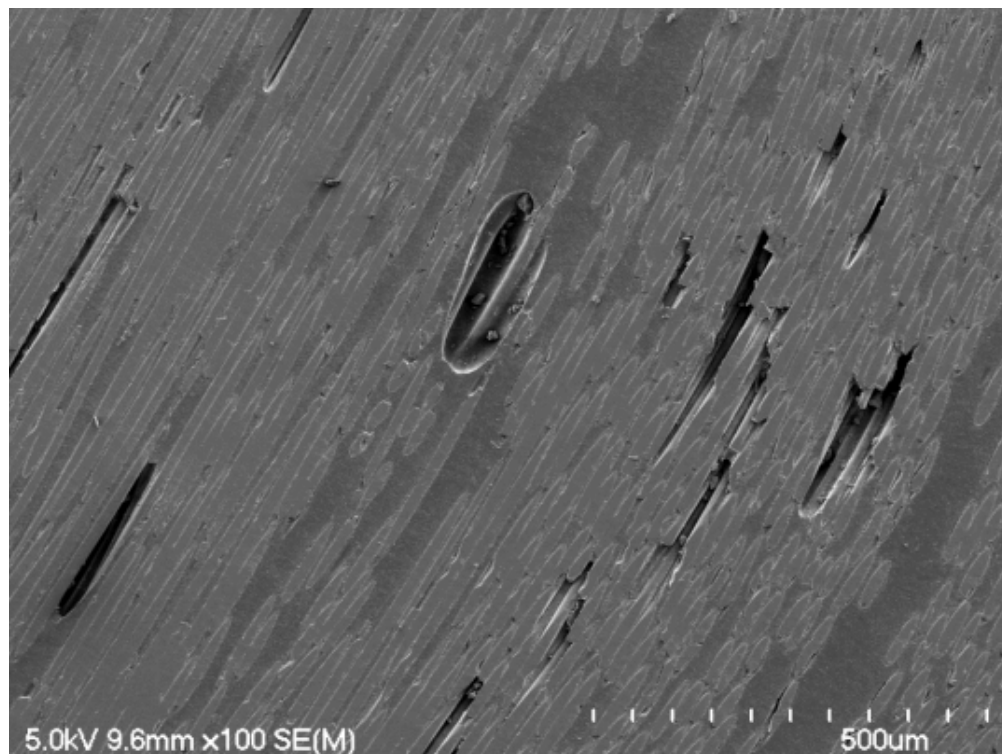


Figure 5.124: SEM micrograph of the longitudinal section of BASA BFRP bars 8-mm (code 51) at low magnification

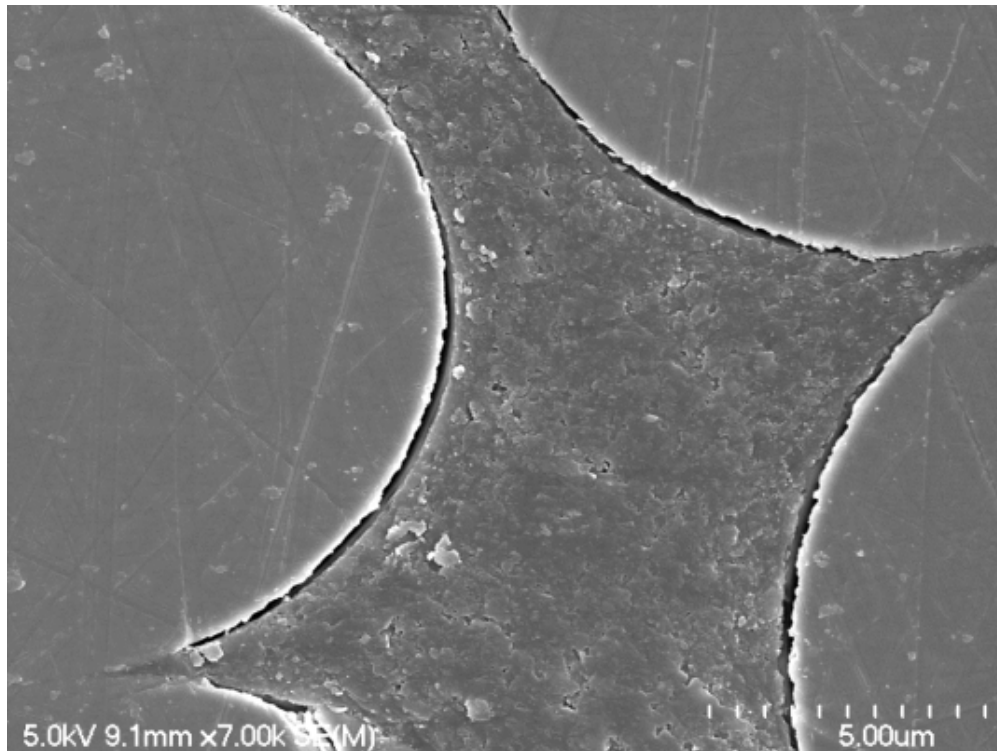


Figure 5.125: SEM micrograph of BASA BFRP bars 8-mm (code 51) at high magnification

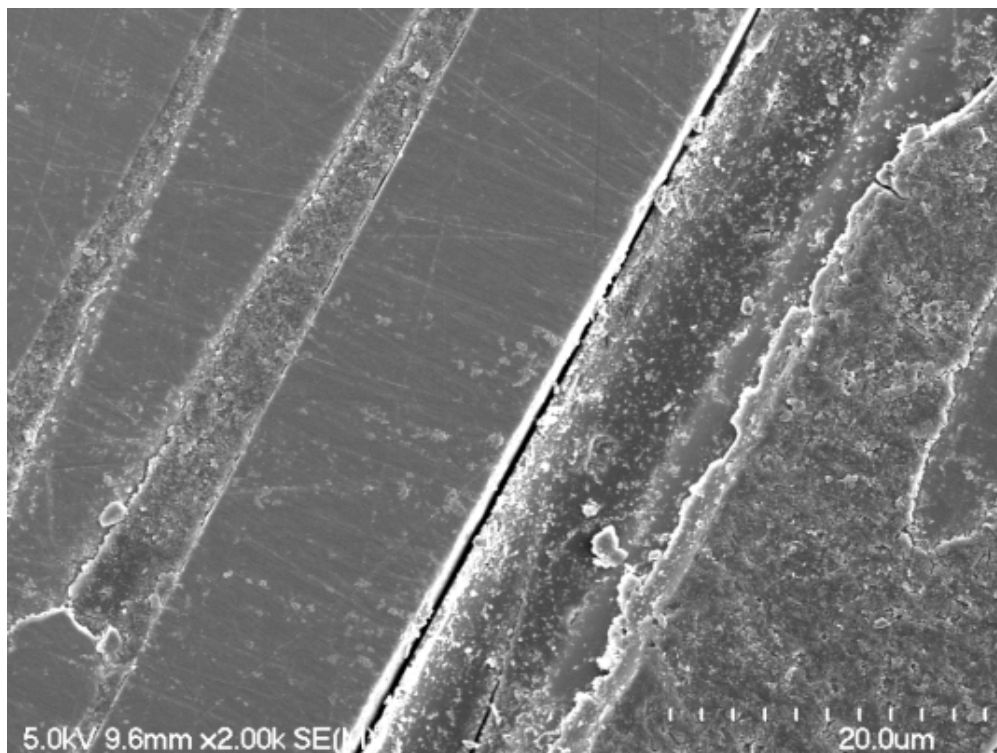


Figure 5.126: Longitudinal section of BASA BFRP bars 8-mm (code 51) at high magnification

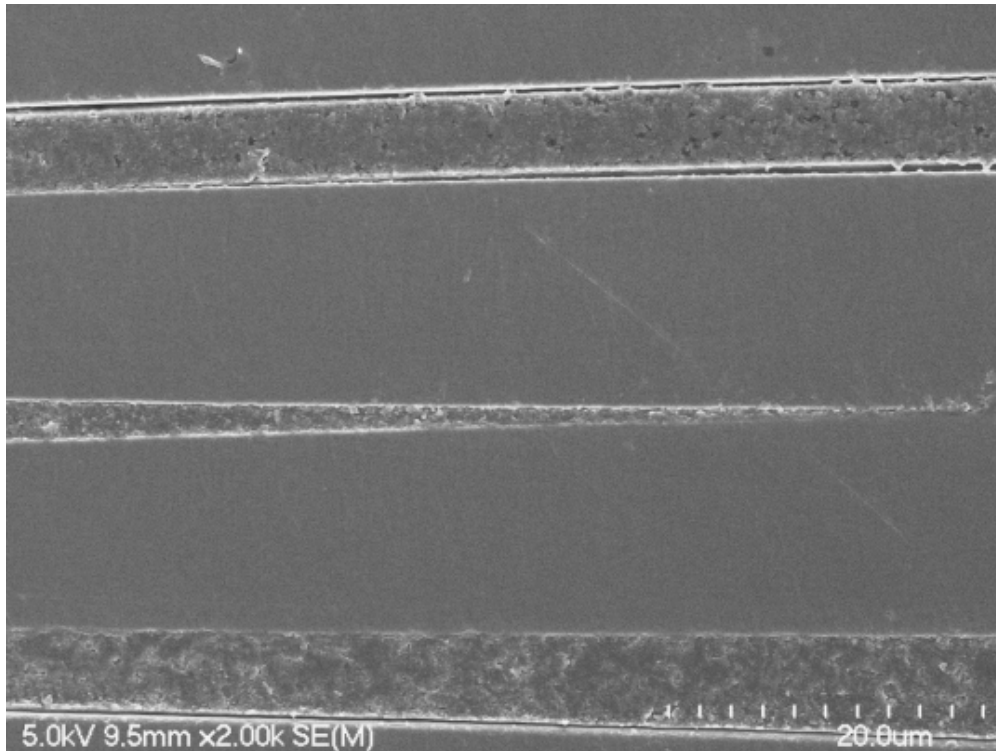


Figure 5.127: Longitudinal section of BASA BFRP bars 8-mm (code 51) after conditioning of 1 month (720 hrs.) at 22°C

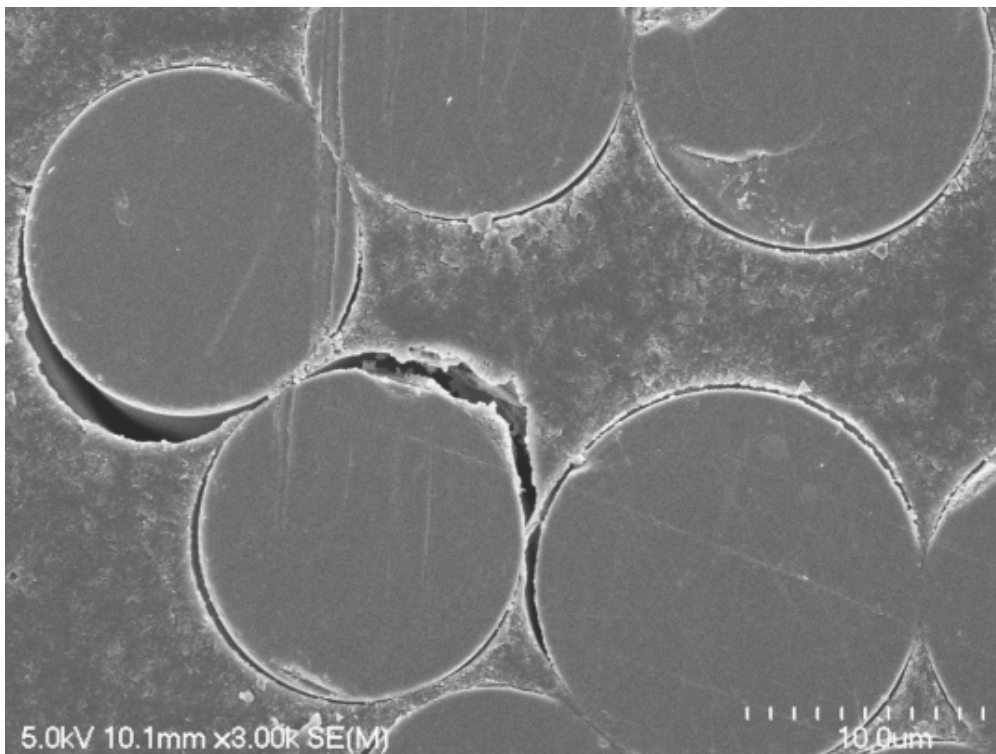


Figure 5.128: BASA BFRP bars 8-mm (code 51) after conditioning of 1 month (720 hrs.) at 22°C at high magnification

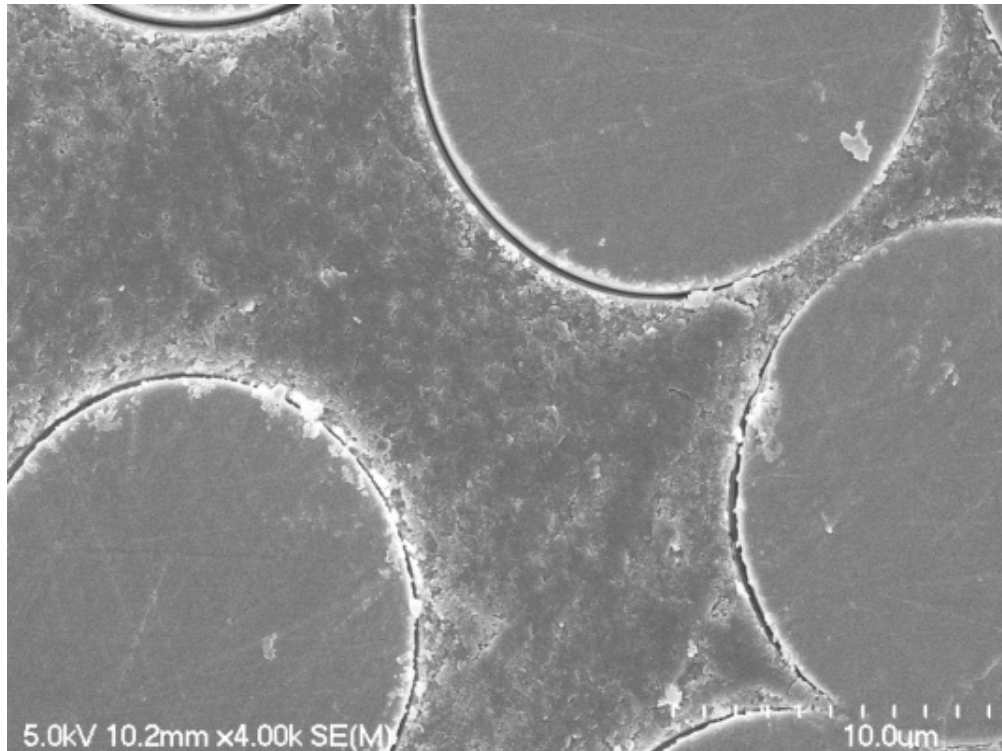


Figure 5.129: BASA BFRP bars 8-mm (code 51) after conditioning of 1 month (720 hrs.) at 22°C at high magnification

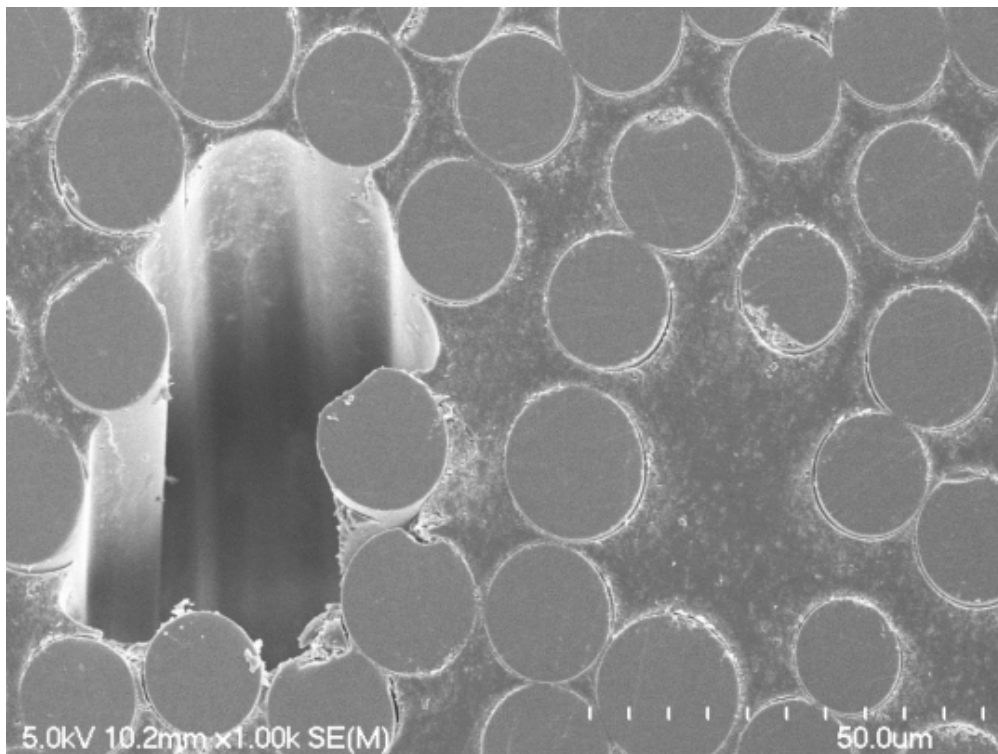


Figure 5.130: BASA BFRP bars 8-mm (code 51) after conditioning of 1 month (720 hrs.) at 60°C showing a pore

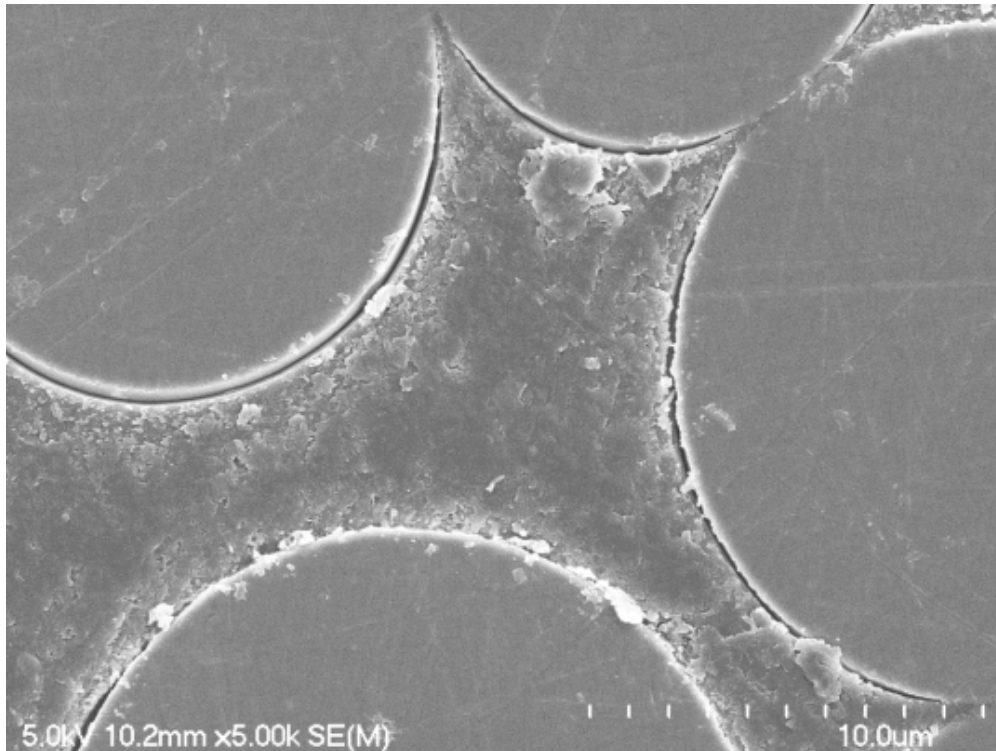


Figure 5.131: BASA BFRP bars 8-mm (code 51) after conditioning of 1 month (720 hrs.) at 60°C at high magnification

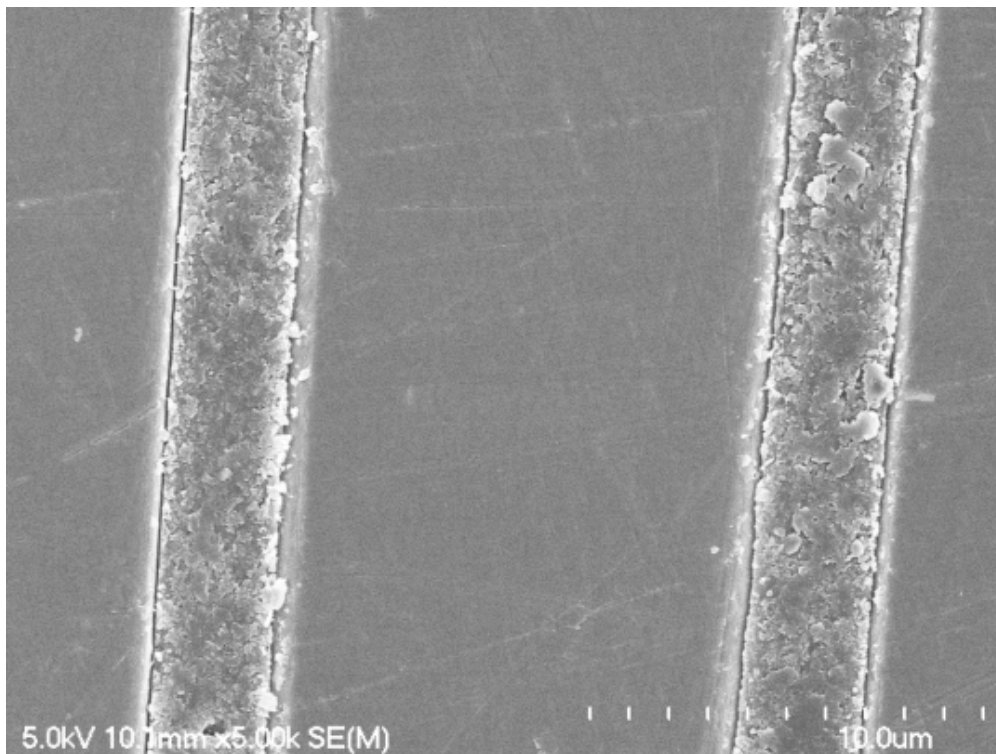


Figure 5.132: BASA BFRP bars 8-mm (code 51) after conditioning of 1 month (720 hrs.) at 60°C at high magnification

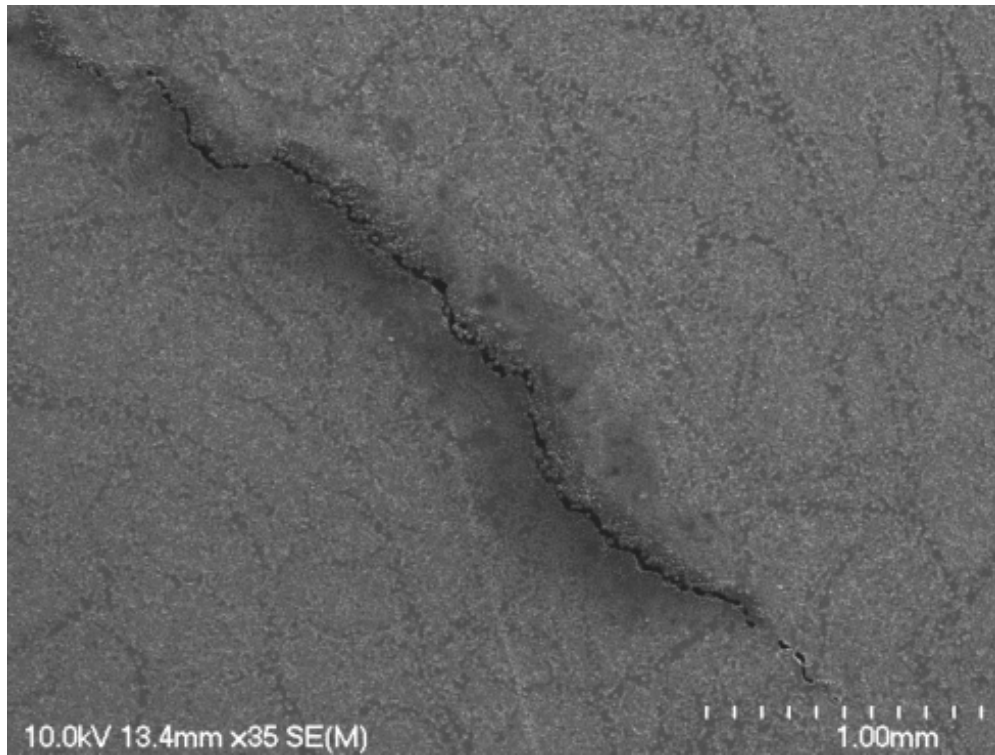


Figure 5.133: Sample 1 (BASA BFRP bars 8-mm (code 43)) at low magnification (1,000 hr-Alkaline - 60°C)

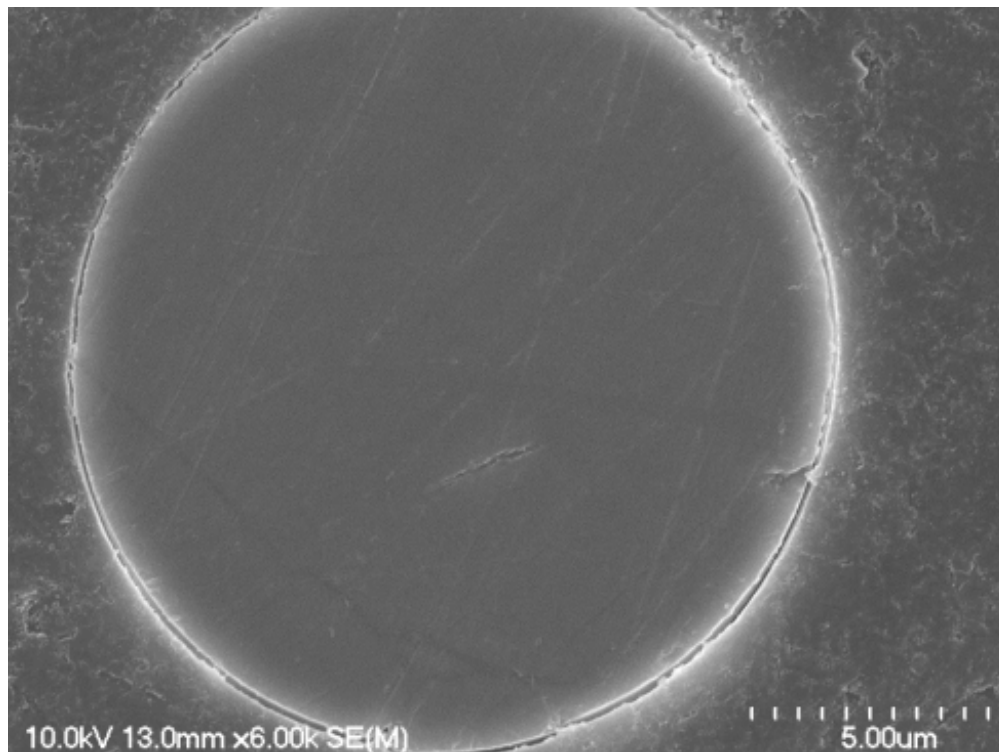


Figure 5.134: Sample 1 (BASA BFRP bars 8-mm (code 43)) at high magnification (1,000 hr-Alkaline - 60°C)

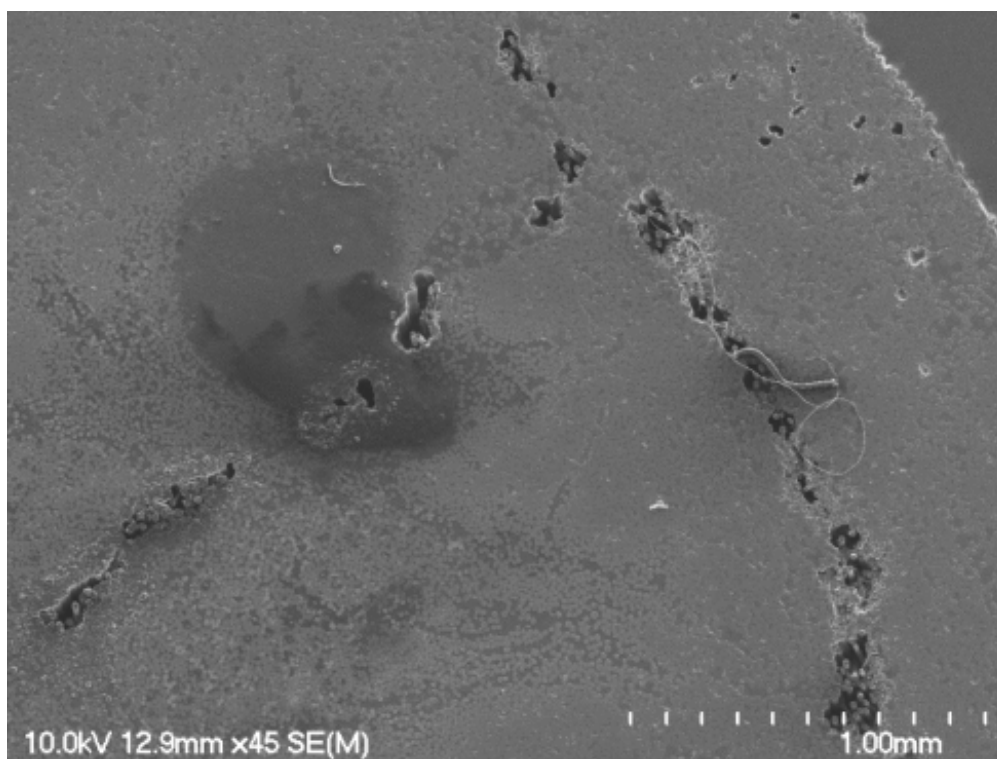


Figure 5.135: Sample 2 (BASA BFRP bars 8-mm (code 43)) at low magnification (3,000 hr-Alkaline - 60°C)

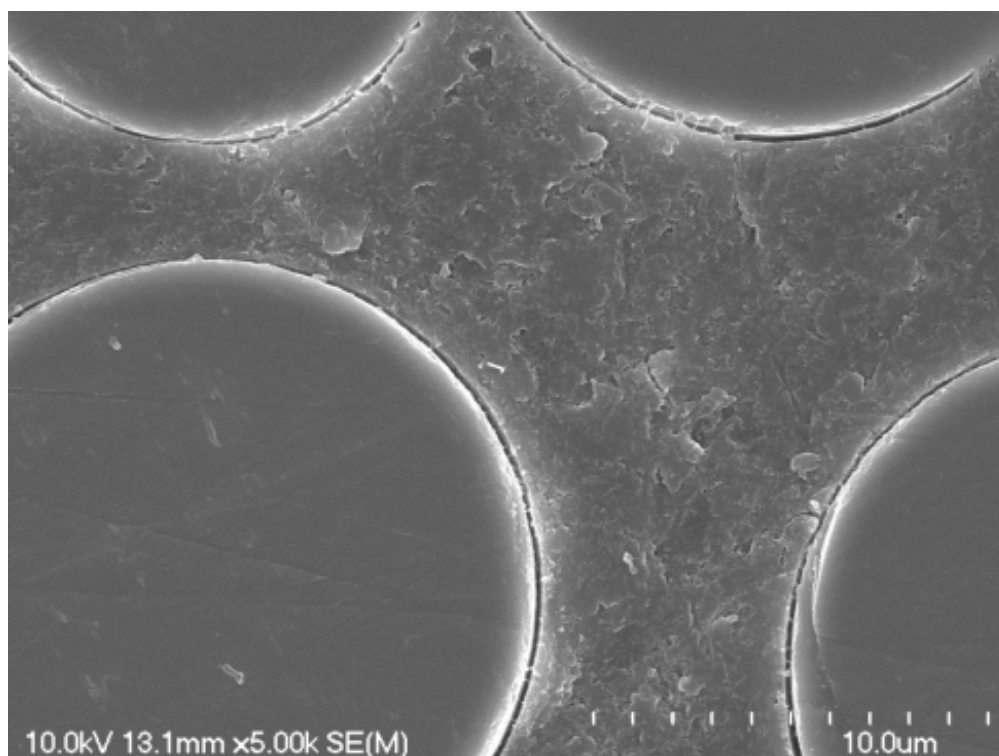


Figure 5.136: Sample 2 (BASA BFRP bars 8-mm (code 43)) at high magnification (3,000 hrs. - Alkaline - 60°C)

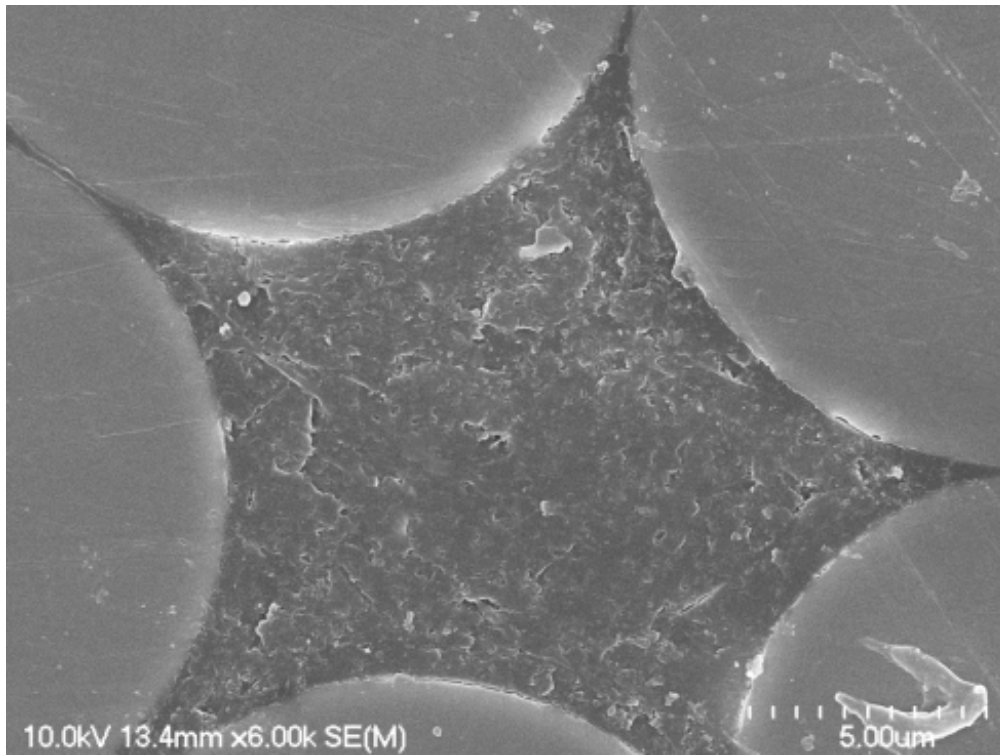


Figure 5.137: Sample 3 (BASA BFRP bars 8-mm (code 43)) at high magnification (3,000 h-Water - 60°C)

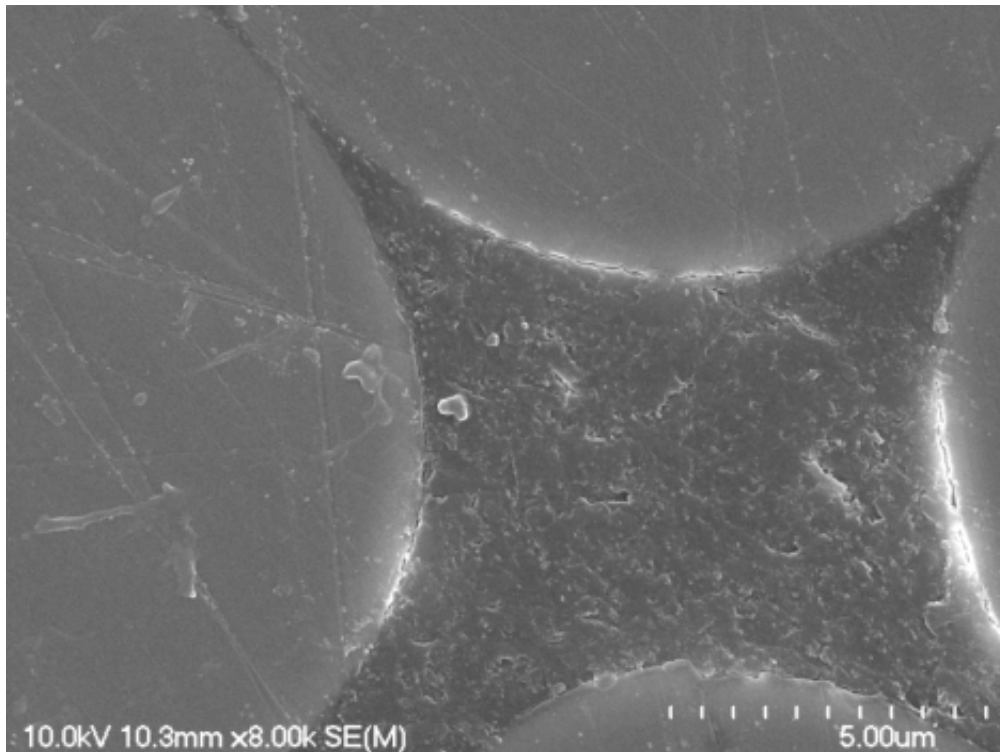


Figure 5.138: Sample 4 (BASA BFRP bars 8-mm (code 43)) at high magnification (3 months-Alkaline - 50°C)

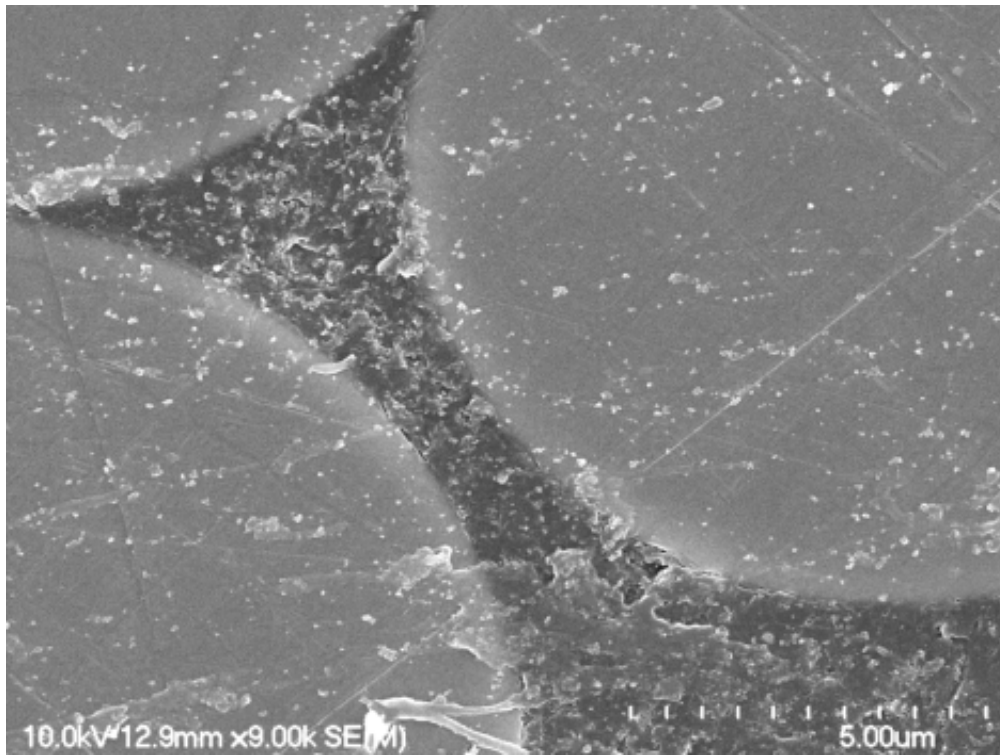


Figure 5.139: Sample 5 (BASA BFRP bars 8-mm (code 51)) at high magnification (1,000 h-Alkaline - 60°C)

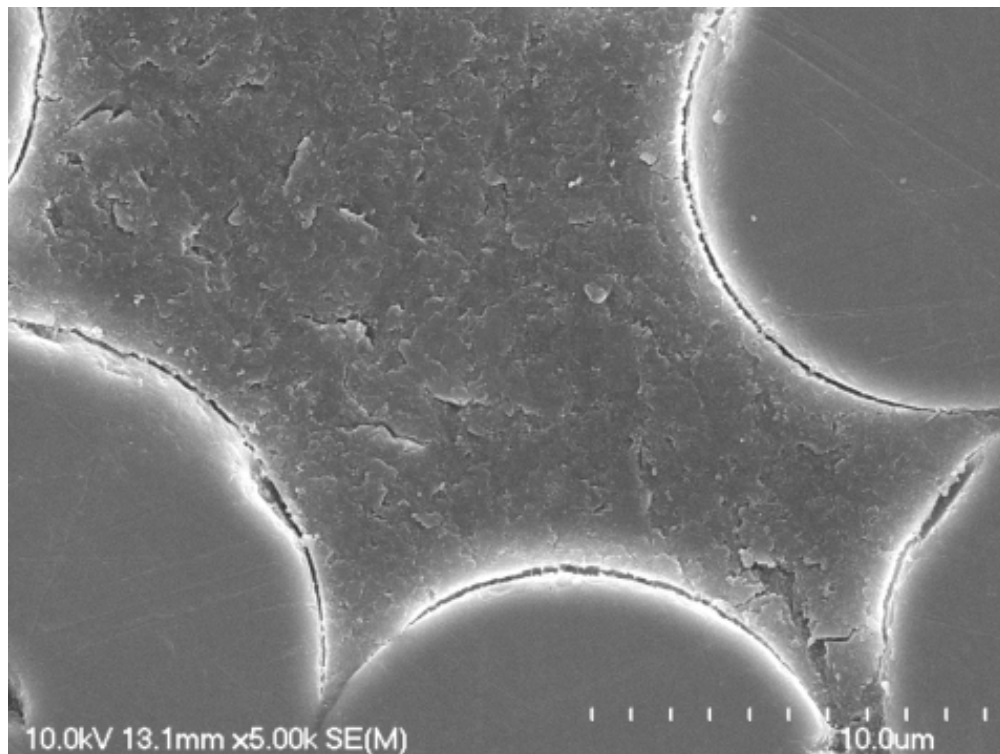


Figure 5.140: Sample 6 (BASA BFRP bars 8-mm (code 51)) at high magnification (3,000 h-Alkaline - 60°C)

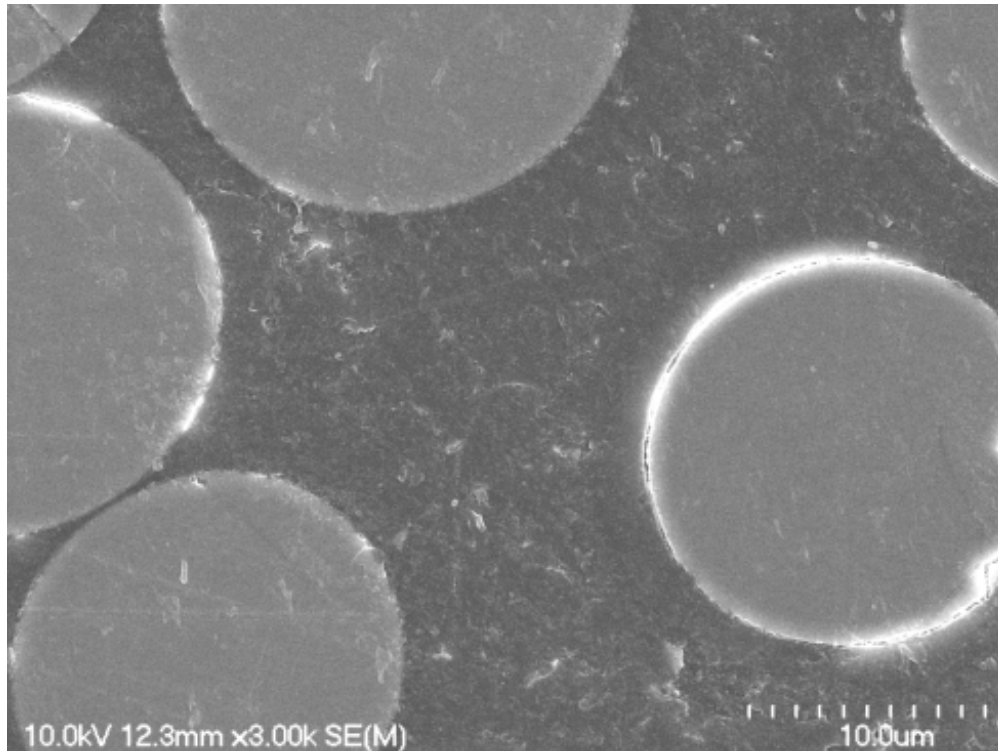


Figure 5.141: Sample 7 (BASA BFRP bars 8-mm (code 51)) at high magnification (3,000 h-Water - 60°C)

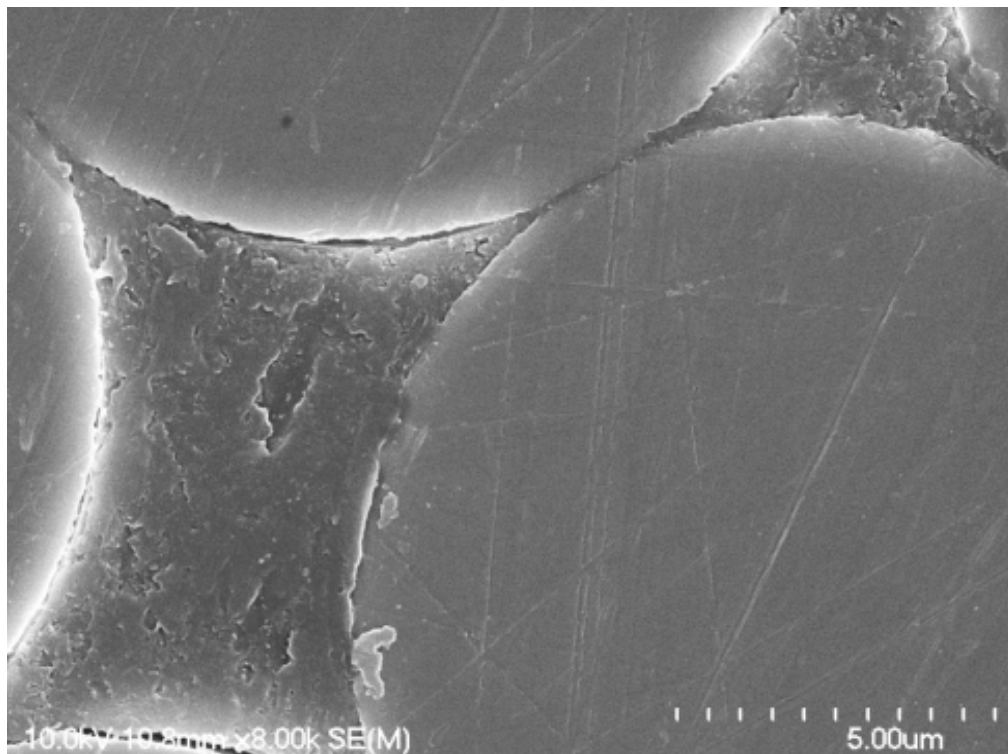


Figure 5.142: Sample 8 (BASA BFRP bars 8-mm (code 51)) at high magnification (3 month-Alkaline - 50°C)

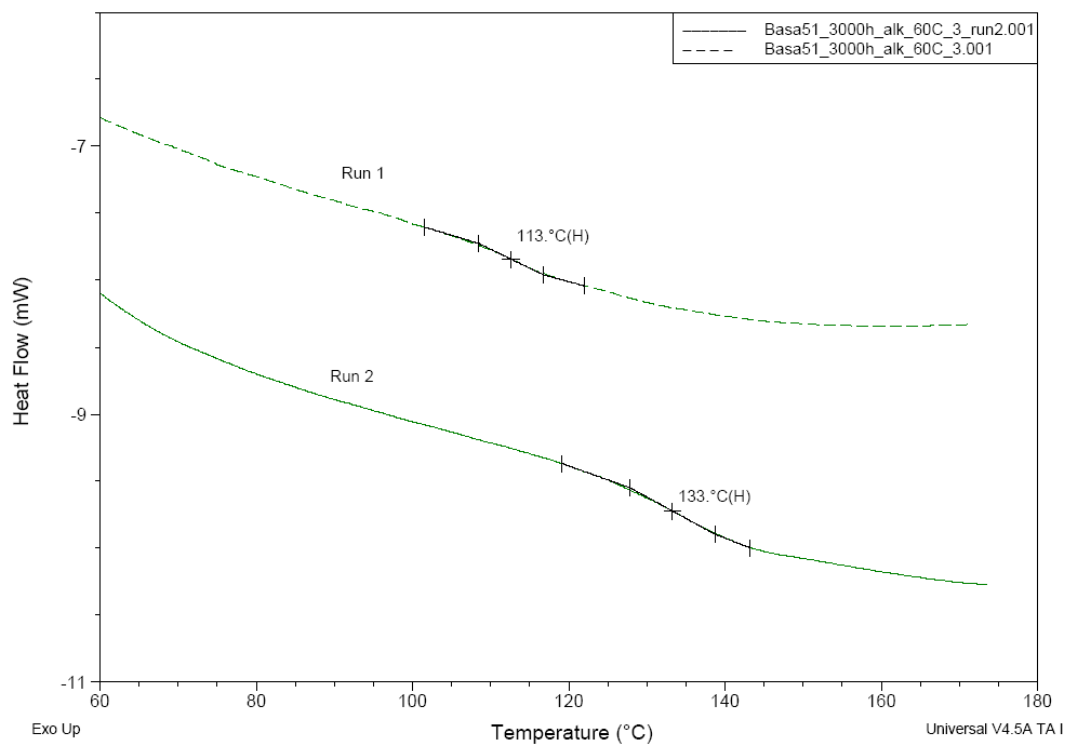


Figure 5.143: Thermograms (runs 1 and 2) of BASA BFRP bars (code 51) conditioned in alkaline solution at 60C during 3,000 hrs.

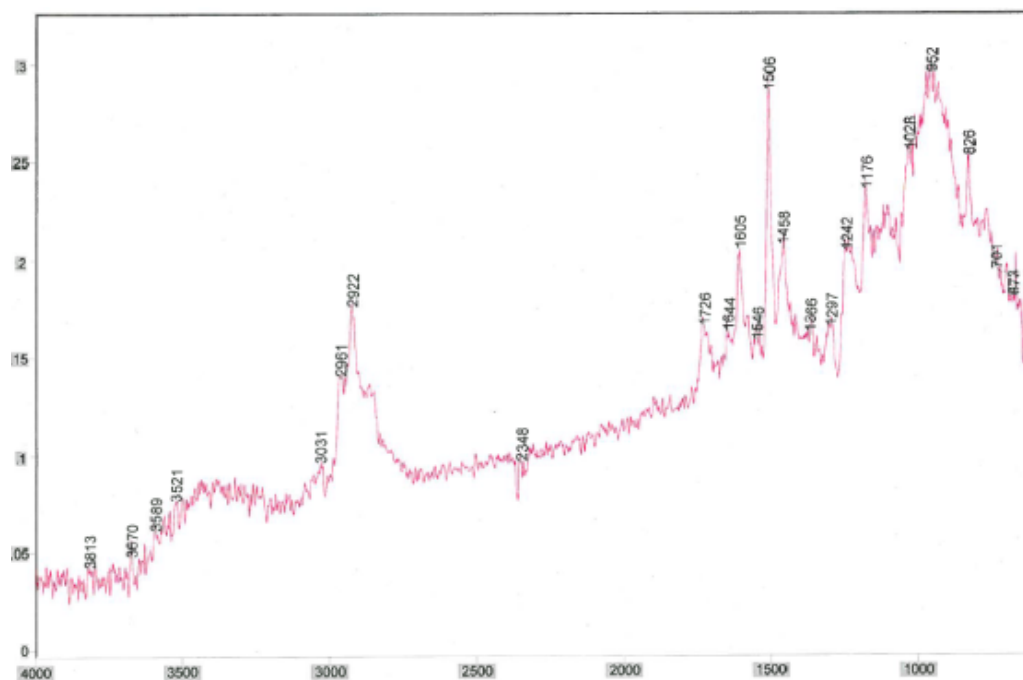


Figure 5.144: FTIR spectrum of BASA BFRP bars (code 43) reference

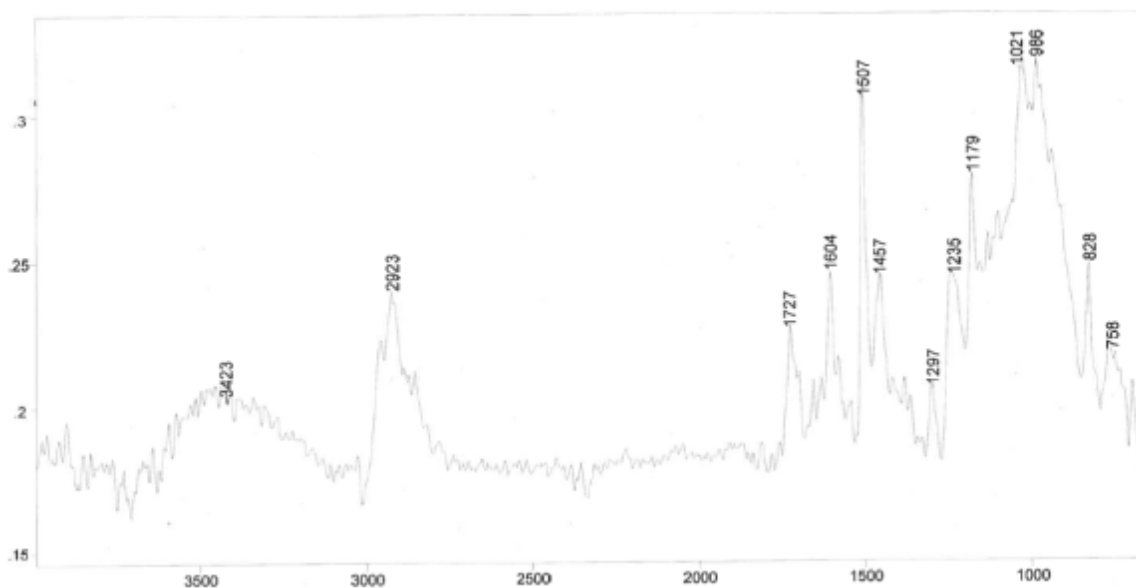


Figure 5.145: FTIR spectrum of BASA BFRP bars (code 43) conditioned in alkaline solution at 60C during 3,000 hours

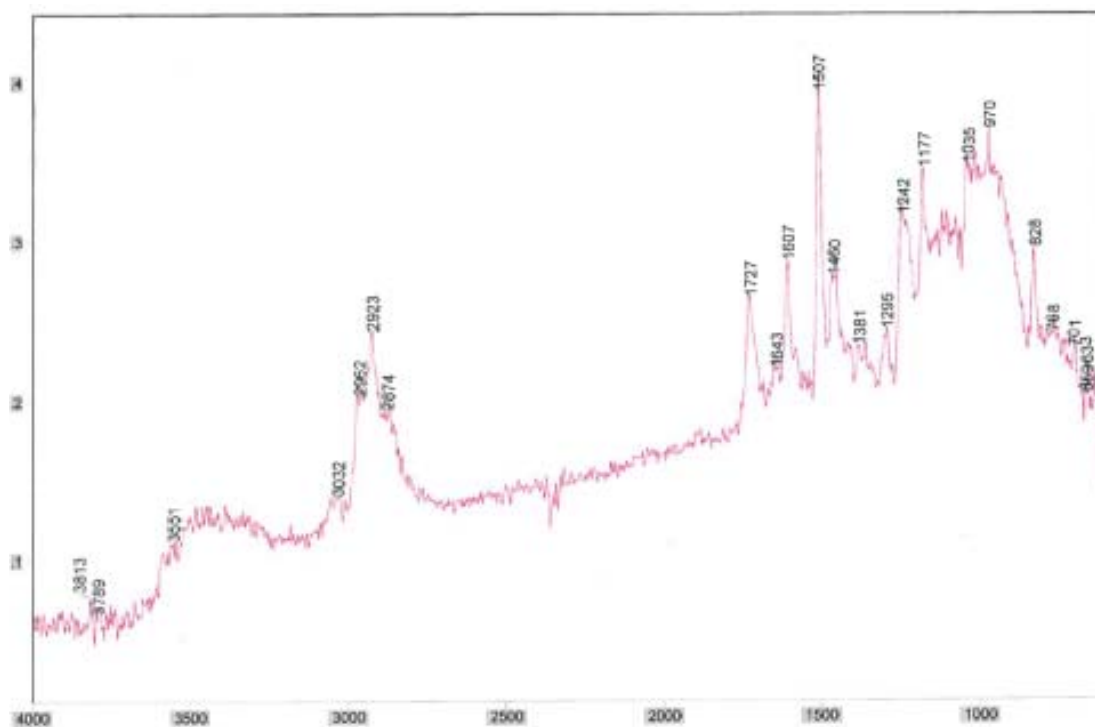


Figure 5.146: FTIR spectrum of BASA BFRP bars (code 51) reference

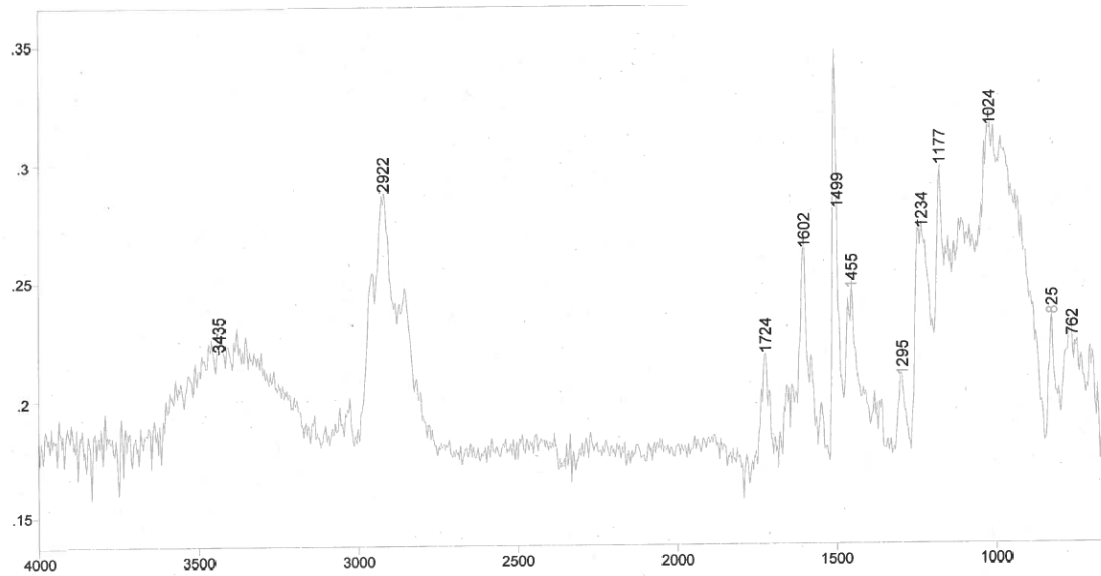


Figure 5.147: FTIR spectrum of BASA BFRP bars (code 51) conditioned in alkaline solution at 60C during 3,000 hours

5.3.2 Mechanical Properties

5.3.2.1 Tensile properties

This section presents the test results of tension testing of different types of BFRP bars; such as I and J types (Figure 5.7), prestressing BFRP bars 7-mm (Figure 5.8), BASA BFRP bars 8-mm code 43 and 51 (Figure 5.9), BASA BFRP bars 12-mm code 43 (Figure 5.10) and BASA BFRP 16-mm (Figure 5.11). The objectives of this test were to determine the ultimate tensile strength, rupture strain, and modulus of elasticity of the BFRP bars. In addition, to determine the residual tensile strengths of the BFRP bars under different exposure times in conditioning and environmental temperatures.

BFRP bars type “I” and “J”

The cross-sectional areas of all bars were determined. It should be noted for BFRP bars type J that the spiral wrapping was manually removed from samples in order to measure the cross-sectional area actually engaged during tension testing. The cross-sectional area of each bar type was taken as the average of the eight respective samples. The results are listed below in Table 5.9.

Table 5.9: Tensile properties of BFRP type I and J

Bar Size	d_b (mm)	Area (in ²)	Elastic tensile modulus, E_f (ksi)	Ultimate Ten- sile Strength, f_{fu} (ksi)	Rupture Strain, ϵ_{fu} (%)
Type I	12.7	0.169	6076± 136.1	150±3.2	2.39±0.08
Type J	15.9	0.167	7033±454.6	151±6.3	2.23±0.16

Ten specimens of each BFRP bar type (I and J) were cut at random from stock bar. Each specimen was 48 inches in total length, leaving a free length of 24 inches between the grips. The average ultimate tensile strength of bar type I is 150 ksi with an average rupture strain of 2.39%. The average modulus of elasticity of bar type I is 6,076 ksi. The average ultimate tensile strength of bar type J is 151 ksi with an average rupture strain of 2.23%. The average modulus of elasticity of bar type J is 7,033 ksi. The modulus of elasticity was computed between 20% and 50% of the ultimate load for each test specimen. The result from tension testing of type I specimens and type J specimens is reported in Table 5.9. Figure 5.148 shows a typical stress-strain curve for the test specimens and Figure 5.149 to Figure 5.156 show the mode of failure of different bars after tension failure.

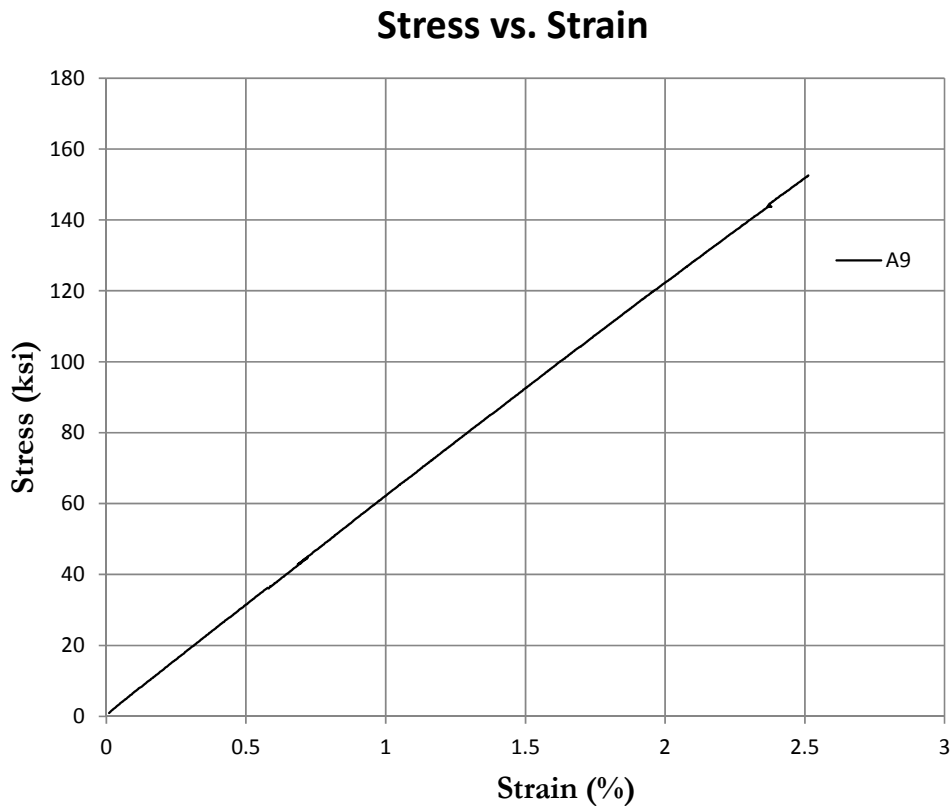


Figure 5.148: Typical stress-strain diagram for BFRP bar type I or J (Specimen I9)



Figure 5.149: Specimen I3



Figure 5.150: Specimen I8,
typical failure of type I bars



Figure 5.151: Specimen I2



Figure 5.152: Specimen J1,
failure near grip



Figure 5.153: Specimen J8,
failure at middle



Figure 5.154: Specimen J9,
failure near grip



Figure 5.155: Specimen J9,
failure at middle



Figure 5.156: Specimen J2,
failure near grip

Prestressing BFRP bars 7-mm in diameter

A total of 8 specimens of prestressing BFRP bars 7-mm were tested in tension until failure. Each specimen was cut into 1285-mm-long section and anchored with steel tubes at each end. The anchorage steel tubes were cut to a length of 450-mm and filled cement based grout. The free length (the length between the two steel anchorages) to bar-diameter ratio was about $40d$. Table 5.10 presents the tensile properties of the tested specimens. The nominal cross-sectional area was calculated considering the nominal diameter of the bars (7 mm). Cross-sectional area of 38.48 mm^2 was employed in the calculations of the tensile strength and the elastic modulus for the tested bars.

Table 5.10: Tensile properties of prestressing BFRP bars of size 7 mm (Area = 38.48 mm^2)

Specimen No.	Ultimate Load (kN)	Ultimate Stress (MPa)	Tensile Modulus (GPa)	Ultimate Strain (%)
1	55.66	1446	68.02	2.13
2	65.22	1695	68.43	2.48
3	60.22	1565	69.02	2.27
4	68.31	1775	68.86	2.58
5	67.35	1750	69.61	2.51
6	62.57	1626	68.84	2.36
7	72.06	1873	69.46	2.70
8	65.82	1710	70.10	2.44
Average	64.65	1680	69.04	2.43
SD	5.10	133	0.67	0.18
COV (%)	7.89	7.89	0.97	7.37

Figure 5.157 shows mode of failure of the tested specimens. The typical stress-strain curves of the 7-mm-diameter BFRP bars were linear elastic up to failure. For all specimens the mode of failure was brittle failure. The results indicate that the tensile strength of unconditioned specimens is 1680 ± 133.0 MPa while maximum strain at failure was 2.3 ± 0.18 , which is higher than the minimum value of 1.2% (CSA-S807, 2010).



Figure 5.157: Mode of failure of prestressing BFRP bars of 7-mm in diameter

BASA BFRP bars 8-mm in diameter (code 43 and 51)

A total of 50 specimens of each BFRP bar type (BASA code 43 and 51) were tested in tension until failure, and tensile properties were determined. Five specimens of each type were tested as reference specimens without conditioning, while the others were tested after being conditioned in alkaline solution at 22°C, 40°C, and 60 °C for one and three months. Each specimen was cut into 1285-mm-long section and anchorage with steel tubes at both ends with a length of 450-mm and the free length was 320 mm ($=40d$). Figure 5.158 shows the BASA BFRP bars before testing. The nominal cross-sectional area was used in the calculation ($\text{Area} = 50.26 \text{ mm}^2$). A typical mode of failure for the test specimens is shown in Figure 5.159 and it was a brittle failure with debonding and/or matrix failure. The average stress-strain curves of the BFRP bars were practically linear and elastic up to failure (Figure 5.160 and Figure 5.161).

Table 5.11 and Table 5.12 show the tensile properties test results for unconditioned and conditioned. The tensile strength for the reference BASA BFRP bars code 43 and 51 showed an average of 1655 ± 94.9 (83.2 kN) and 1567 ± 114.5 (79 kN) and modulus of elasticity of 64.6 ± 1.4 and 59.5 ± 3.3 GPa, respectively. While maximum strain at failure was 2.6 ± 0.17 and 2.6 ± 0.28 , respectively, which were higher than the minimum value of 1.2% (CSA-S807, 2010). In addition, the conditioned specimens test results indicate that the tensile strength retention after 1 or 3 months at 22, 40, and 60°C was an average of 89.7, 76.3, and 86.3% (for BASA BFRP bars 8-mm code 43) and 103.5, 77.9, and 83.2% (for BASA BFRP bars 8-mm code 51), respectively. While the

tested BFRP bars evidenced maximum strain at failure ranging from 2.0 to 2.3% and 2.2 to 2.8%, respectively, which were higher than the minimum value of 1.2% (CSA S807, 2010).



Figure 5.158: BASA BFRP bars of sizes 8 mm (code 43 and 5) before testing



Figure 5.159: Mode of failure of BASA BFRP bars of size 8 mm (code 43 and 51)

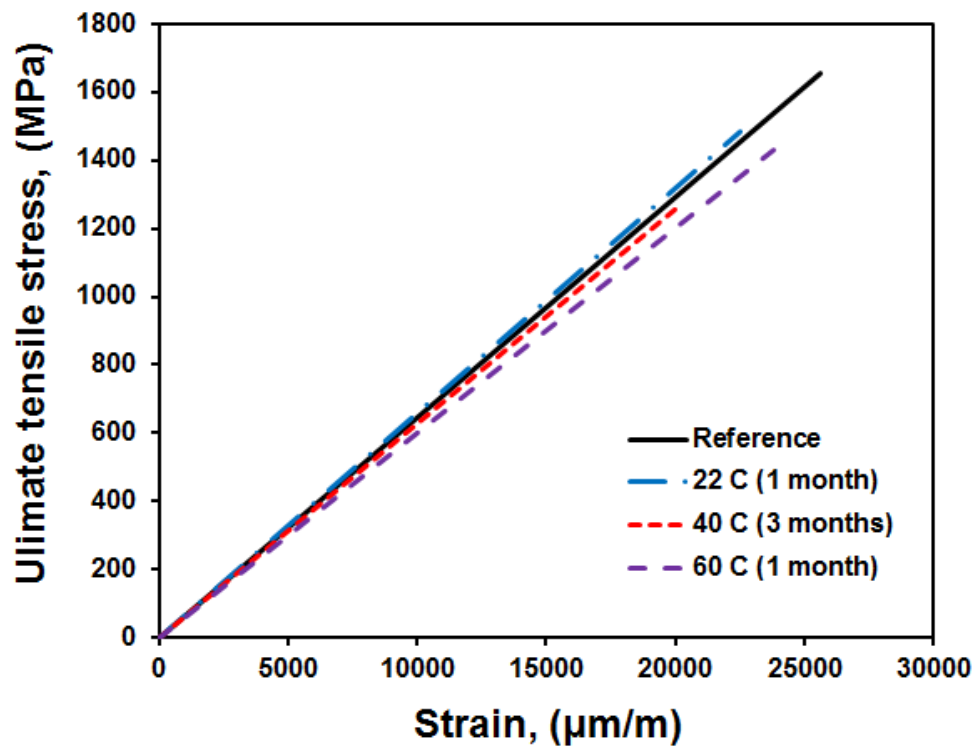


Figure 5.160: Ultimate tensile stress-strain relationships of BASA BFRP bars of size 8 mm (code 43): reference and conditioned under different environmental temperatures

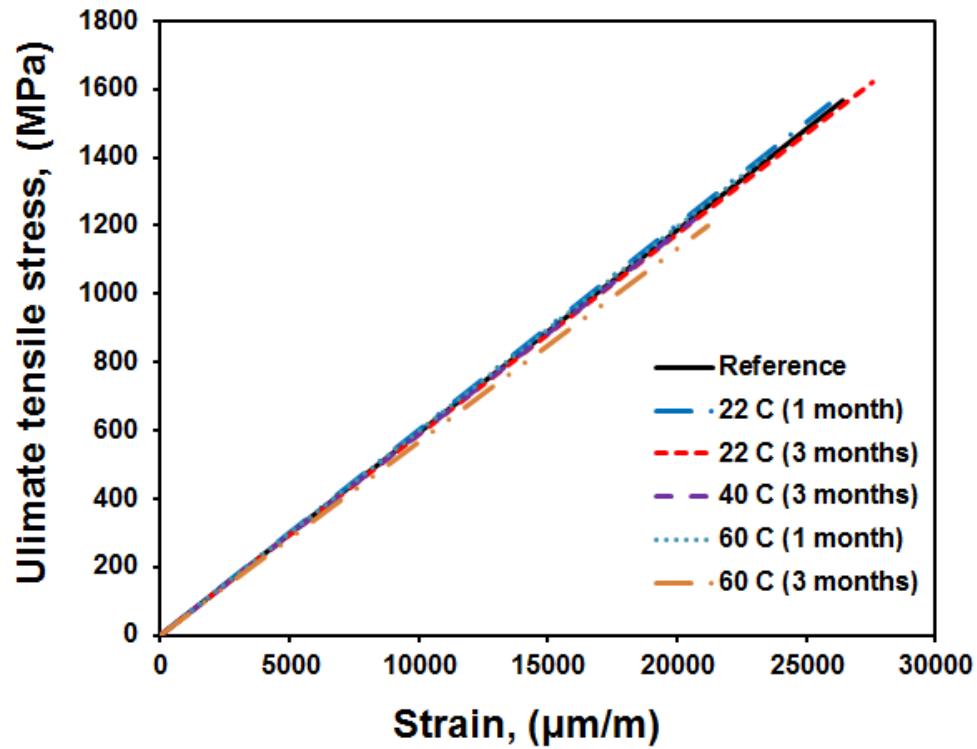


Figure 5.161: Ultimate tensile stress-strain relationships of BASA BFRP bars of size 8 mm (code 51): reference and conditioned under different environmental temperatures

Table 5.11: Tensile properties of BFRP bars of size 8 mm (BASA BFRP bars code 43)

Bar Status	Ultimate Load (kN)	Ultimate Tensile Strength, f_{tu} (MPa)	Elastic tensile modulus, E_f (GPa)	Ultimate Tensile Elongation, ϵ_u (%)	Tensile strength retention (%)
Reference	83.2±4.8	1655±94.9	64.6±1.4	2.6±0.17	-
Conditioned at 22 °C for 1 month	74.6±3.1	1484±61.2	66.1±2.0	2.3±0.16	89.7
Conditioned at 60 °C for 1 month	71.8±2.7	1429±52.6	60.1±1.2	2.4±0.06	86.3
Conditioned at 40 °C for 3 months	63.4±4.6	1262±92.4	62.9±1.2	2.0±0.19	76.3

Table 5.12: Tensile properties of BFRP bars of size 8 mm (BASA BFRP bars code 51)

Bar Status	Ultimate Load (kN)	Ultimate Tensile Strength, f_{fu} (MPa)	Elastic tensile modulus, E_f (GPa)	Ultimate Tensile Elongation, ϵ_u (%)	Tensile strength retention (%)
Reference	78.7±5.8	1567±114.5	59.5±3.3	2.6±0.28	-
Conditioned at 22 °C for 1 month	78.2±2.6	1555±52.2	60.0±0.9	2.6±0.12	99.3
Conditioned at 22 °C for 3 months	81.5±3.8	1622±75.2	58.8±0.2	2.8±0.13	103.5
Conditioned at 40 °C for 3 months	61.3±3.6	1219±70.9	59.0±0.5	2.1±0.12	77.9
Conditioned at 60 °C for 1 month	70.8±4.6	1409±91.7	59.7±1.6	2.4±0.13	89.9
Conditioned at 60 °C for 3 months	60.2±1.6	1198±31.7	56.6±0.9	2.1±0.02	76.5

BASA BFRP bars 12-mm in diameter (code 43)

A total of 16 specimens of BASA BFRP bars, ten unconditioned specimens and 6 conditioned specimens for exposure time of 2 months (1440 hrs.) at 60 °C, were tested up to failure. Each specimen was cut into 1630-mm-long section and each end were attached anchorage steel tubes with a length of 560-mm while the free length between tubes was 480 mm (=40 d). Table 5.13 presents the tensile properties of the BASA BFRP reference and conditioned specimens of sizes 12 mm (code 43). The cross-sectional area of 113.14 mm² was employed in the calculations of the tensile strength and the elastic modules for all the tested bars. A typical mode of failure of the tested specimens is shown in Figure 5.162 and Figure 5.163. It can be seen that the reference and aged bars showed similar mode of failure. The tensile strength for the reference and conditioned specimens was an average of 1758.9±47.0 (199.0 kN) and 1473.4±23.2 (166.7 kN) and modulus of elasticity of 65.1±0.5 and 64.2±0.7 GPa, respectively. While maximum strain at failure was 2.7±0.08 and 2.3±0.04, respectively. The tensile strength retention for the conditioned specimens indicates that the tensile strength after 2 months at 60°C was an average of 80.6%, means a loss of resistance by 19.4% while the loss of elastic modulus was negligible.

Table 5.13: Ultimate tensile properties of BASA BFRP bars of size 12 mm code 43

Bar Status	Ultimate Load (kN)	Ultimate Tensile Strength, f_{fu} (MPa)	Elastic tensile modulus, E_f (GPa)	Ultimate Tensile Elongation, ϵ_u (%)	Tensile strength retention (%)
Reference	199.0±5.3	1758.9±47.0	65.1±0.5	2.7±0.08	-
Conditioned at 60 °C for 2 months	166.7±2.6	1473.4±23.2	64.2±0.7	2.3±0.04	80.6



Figure 5.162: A typical mode of failure of BASA BFRP bars 12 mm (code 43) (Reference)



Figure 5.163: A typical mode of failure of BASA BFRP bars 12 mm (code 43) (Conditioned for 2 months (1440 hrs.))

BASA BFRP bars 16-mm in diameter

Eight specimens of BASA BFRP bars 16-mm in diameter were tested in tension until brittle failure. The specimens were cut to a length of 1935-mm and anchorage steel tubes attached at each end with a length of 650-mm and the free length was 640 mm ($=40d$). Table 5.14 presents the tensile properties of the test specimens BASA BFRP of 16 mm. The cross-sectional areas used for the calculation were obtained by a physical analysis of each diameter of bars. Cross-sectional area of 201.14 mm^2 was used in the calculations of the tensile strength and the elastic modulus for all the tested bars. The mode of failure of the test specimens is shown in Figure 5.164. The tensile strength for the reference BASA BFRP bars of 16 mm showed an average of 1724 ± 63.6 (346.76 kN) and modulus of elasticity of 64.8 ± 3.3 GPa. While maximum strain at failure was 2.7 ± 0.17 , which is higher than the minimum value of 1.2% (CSA-S807, 2010).

Table 5.14: Tensile properties of BASA BFRP bars of 16 mm in diameter, (Area = 201.14 mm²)

Specimen No.	Ultimate Load (kN)	Ultimate Stress (kN)	Tensile Modulus (GPa)	Ultimate Strain (%)
1	364.41	1812	62.31	2.91
2	345.43	1717	60.88	2.82
3	338.58	1683	65.98	2.55
4	360.68	1793	66.46	2.70
5	325.20	1617	61.49	2.63
6	343.65	1709	68.98	2.48
7	341.20	1696	68.95	2.46
8	354.91	1764	62.93	2.80
Average	346.76	1724	64.75	2.67
SD	12.79	63.61	3.27	0.17
COV (%)	3.69	3.69	5.05	6.26



Figure 5.164: Mode of failure of BASA BFRP bars of 16 mm in diameter

5.3.2.2 Transverse and interlaminar shear and flexural strengths properties

RWS BFRP bars No. 4, No. 5, and No. 8

The test results of 18 RWS BFRP bars (Figure 5.6) were presented (6 specimens for each bar size, No. 4, No. 5, and No. 8, nominal area = 129, 199, 510 mm² respectively). The transverse shear properties of RWS BFRP bars with different bar size were calculated from equation 5.10. The shear strength before and after conditioning were determined as shown in Table 5.15. The average transverse shear strength of unconditioned and conditioned specimens were an average of 210±14.6, 208±9.8 and 179±3.4 MPa and 182±8.4, 194±7.5 and 176±6.2 MPa for bars No. 4, 5, and 8, respectively, which indicate that the test results were higher than specified limit according to (CSA S807, 2010) (>160 MPa). Furthermore, the transverse shear strength after the immersion in alkaline solution at 60°C for 2000 hrs showed a decrease in the transverse shear

strength by an average of 13, 7, and 2% for bars No. 4, 5, and 8, respectively. A typical mode of failure for the test specimens is presented in Figure 5.165.



Figure 5.165: Reference and aged BFRP bars No.5 after shear tests

Table 5.15: Average shear test results for BFRP bars reference and conditioned in alkaline solution at 2000 hrs.

Bar No.	d_b (mm)	Area (mm ²)	Reference		Conditioned at 2000 hrs		Shear strength retention (%)
			Load (kN)	Stress (MPa)	Load (kN)	Stress (MPa)	
No. 4	12.7	129	54.3±3.8	210±14.6	47.0±2.2	182±8.4	87
No. 5	15.9	199	82.9±3.9	208±9.8	77.2±3.0	194±7.5	93
No. 8	25.4	510	182.0±3.4	179±3.4	180.0±6.4	176±6.2	98

Prestressing BFRP bars 7-mm in diameter

Thirty-two prestressing BFRP specimens of 7-mm diameter were tested in transverse and interlaminar shear and flexural tests until failure. Twenty specimens were tested as reference specimens (without conditioning) and after conditioning in alkaline solution at 60°C for 1,000 hrs due to transverse and interlaminar shear test. The test specimens were cut at approximate length of 200 mm and 47 mm for transverse and interlaminar shear test, respectively. On the other hand, twelve specimens were tested as reference (without conditioning) and after conditioning at 60°C for 1,000 hrs under flexural test. The test was conducted on specimens have a 160 mm-long over a simply supported span equal to 16 times the bar diameter, as shown in Figure 5.166. The nominal cross-section area of (Area = 38.48 mm²) was used in the calculations of stress.

Table 5.16 reported the transverse shear test results of unconditioned and conditioned specimens while a typical mode of failure is shown in Figure 5.167. The average transverse shear strength of the unconditioned and conditioned specimens were an average of 343±17.6 and 327±7.6 MPa, respectively, which indicate that the test results were higher than specified limit according to (CSA S807, 2010) (>160 MPa). Furthermore, the experimental results showed a slight decrease in the transverse shear strength after the immersion at 60°C during 1,000 hrs by an average of 4.7%. On the other hand, the unconditioned BFRP specimens provided an average interlaminar shear strength of 63 MPa, compared with 55 MPa for conditioned ones. Hence, the conditioned

specimens revealed a decrease in the interlaminar shear strength by an average of 12.7 % compared with the unconditioned specimens. Table 5.17 shows the test results of unconditioned and conditioned specimens and Figure 5.168 shows the mode of failure of the test specimens. Table 5.18 provides the flexure strength test results of the unconditioned and conditioned specimens. A typical mode of failure is shown in Figure 5.169. Test results showed a flexural strength of 1790 ± 91.3 and 1763 ± 60.9 MPa for unconditioned and conditioned specimens, respectively, which indicates a loss of resistance by 1.53%.



Reference specimens



Conditioned specimens

Figure 5.166: Tested Specimens



Figure 5.167: Transverse shear failure of prestressing BFRP bars 7-mm



Figure 5.168: Interlaminar shear failure prestressing BFRP bars 7-mm

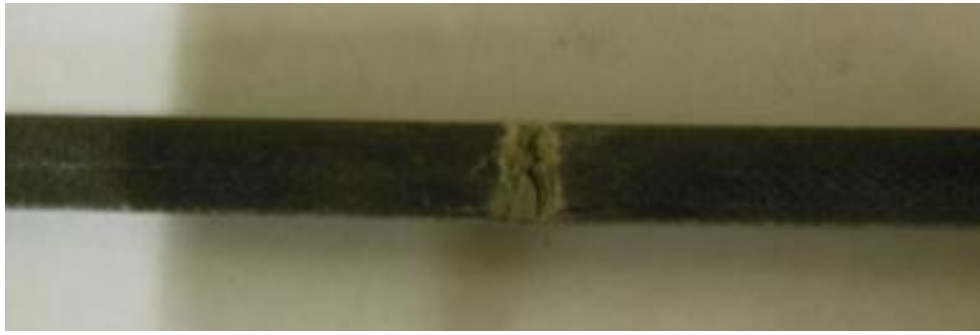


Figure 5.169: A typical flexural mode of failure (Compression failure)

Table 5.16: Transverse shear strength of prestressing BFRP bars 7 mm-in diameter (Area = 38.48 mm²)

Specimen No.	Reference		Conditioned of 1000 hrs	
	Ultimate Load (kN)	Ultimate Shear Stress (MPa)	Ultimate Load (kN)	Ultimate Shear Stress (MPa)
1	28.57	371.232	24.82	322.51
2	26.83	348.623	25.75	334.59
3	26.06	338.617	25.89	336.41
4	25.10	326.143	24.56	319.13
5	25.60	332.64	25.01	324.97
Average	26.43	343.45	25.20	327.52
SD	1.35	17.59	0.58	7.60
COV%	51.23		23.21	

Table 5.17: Interlaminar shear strength of unconditioned and conditioned prestressing BFRP bars 7 mm-in diameter (Area = 38.48 mm²)

Specimen No.	Length (mm)	Span, L (mm)	Diameter, d (mm)	L/d	Reference		Conditioned 1000 hrs	
					Ultimate Load, P (kN)	Apparent horizontal shear strength, S (N/mm ²)	Ultimate Load, P (kN)	Apparent horizontal shear strength, S (N/mm ²)
1	46.0	39.0	7.0	5.57	3.44	59.6	3.39	58.7
2	47.0	39.0	7.0	5.57	3.82	66.1	3.12	54.0
3	47.0	39.0	7.0	5.57	3.72	64.4	3.30	57.1
4	48.4	39.0	7.0	5.57	3.52	60.9	3.18	55.0
5	49.3	39.0	7.0	5.57	3.72	64.4	2.97	51.4
Average					3.64	63.1	3.19	55.3
SD					0.15	2.73	0.16	2.8
COV (%)					4.32	4.32	5.08	5.08

Table 5.18: Flexure properties of unconditioned prestressing BFRP bars 7 mm-in diameter (Area = 38.48 mm²)

No.	Span, L (mm)	Diameter, d (mm)	L/d	Ultimate Load, P (kN)	Mid-span deflection, Y (mm)	Flexure strength, S (MPa)	Modulus of elasticity, E_b (GPa)	Maximum fiber strain, ϵ (mm/mm)
1	120	7	17.1	1.9	6.05	1692.0	95.9	0.0176
2	120	7	17.1	1.93	6.54	1718.7	90.2	0.0190
3	120	7	17.1	1.94	7.02	1727.6	84.4	0.0204
4	120	7	17.1	2.13	7.23	1896.8	89.9	0.0211
5	120	7	17.1	2.03	6.95	1807.8	89.2	0.0202
6	120	7	17.1	2.13	7.00	1896.8	92.9	0.0204
Average				2.01	6.80	1790.0	90.4	0.0198
SD				0.10	0.43	91.3	3.8	0.0012
COV (%)				5.10	6.34	5.10	4.26	6.34

Table 5.19: Flexure properties of 1,000-hrs alkaline conditioned prestressing BFRP bars 7 mm-in diameter (Area = 38.48 mm²)

No.	Span, L (mm)	Diameter, d (mm)	L/d	Ultimate Load, P (kN)	Mid-span deflection, Y (mm)	Flexure strength, S (MPa)	Modulus of elasticity, E_b (GPa)	Maximum fiber strain, ϵ (mm/mm)
1	120	7	17.1	1.97	6.61	1754.4	91.0	0.0193
2	120	7	17.1	2	6.47	1781.1	94.4	0.0189
3	120	7	17.1	1.91	6.09	1700.9	95.8	0.0178
4	120	7	17.1	2.1	7.17	1870.1	89.4	0.0209
5	120	7	17.1	1.92	6.23	1709.8	94.1	0.0182
6	120	7	17.1	1.98	6.47	1763.3	93.4	0.0189
Average				1.98	6.51	1763.3	93.0	0.0190
SD				0.07	0.38	60.9	2.4	0.0011
COV (%)				3.46	5.76	3.46	2.54	5.76

BASA BFRP bars 8-mm in diameter (code 43 and 51)

A total of 62 specimens of BFRP bars 8-mm in diameter type (BASA code 43 and 51) (unconditioned and conditioned) were tested in transverse shear test while 72 and 60 specimens of each type were tested in interlaminar shear and flexural tests. The conditioned specimens were immersed in alkaline solution at 22°C, 40°C, and 60°C for one and/or three months. Figure 5.170 and Figure 5.171 show the BFRP bars before testing by direct application of transverse shear. Figure 5.172 and Figure 5.173 show a typical mode of failure of transverse shear test specimens. Besides, Figure 5.174 and Figure 5.175 to Figure 5.176 show the test specimens were conducted

under horizontal shear test and mode of failure of the test specimens. Moreover, test specimens during flexural test and typical mode of failure are presented in Figure 5.177 and Figure 5.178. The nominal cross-sectional area ($\text{Area} = 50.29 \text{ mm}^2$) and nominal diameter of the BFRP bars (8 mm-diameter) were used in the calculation of stress in different tests.

Table 5.20 provides the test results for both types of BASA BFRP bars. The test result showed an average transverse shear strength for unconditioned and conditioned specimens at 22°C, 40°C, and 60°C for one and/or three months of 315 ± 37.5 , 305.7 ± 20.6 , and 282.7 ± 13.7 (for BASA BFRP bars 8-mm code 43) and 293.0 ± 27.7 , 326.5 ± 4.5 , 286.0 ± 25.5 , and 259.5 ± 13.6 (for BASA BFRP bars 8-mm code 51), respectively. The test results were higher than specified limit according to (CSA S807, 2010) ($>160 \text{ MPa}$). Moreover, the transverse shear strength retention did not provide any effect at 22°C even for 1 or 3 months. The transverse shear strength, however, were loss of resistance at higher temperature degrees at 40°C and 60°C for 1 or 3 months by an average of 3.1 and 10.3% (for BASA BFRP 8-mm code 43) and 2.4 and 11.2% (for BASA BFRP 8-mm code 51), respectively. On the other hand, the unconditioned BFRP specimens provided an average interlaminar shear strength of 72 and 60 MPa, compared with 68, 67, and 69 MPa and 58, 57, 55, and 56 MPa for conditioned BASA BFRP bars code 43 and 51, respectively. Whereas, the conditioned specimens revealed a decrease in the interlaminar shear strength by an average of 6.3 % and 5.7 % compared with the unconditioned specimens (BASA code 43 and 51), respectively. Table 5.17 shows the test results of unconditioned and conditioned specimens.

Table 5.22 and Table 5.23 provide the flexure test results of the unconditioned and conditioned specimens. A typical mode of failure is shown in Figure 5.178. All bars failed with a compression failure in compression side of the bars. Test results showed a flexural strength of 2060 ± 215.8 and $1587.4 \pm 110.7 \text{ MPa}$ for unconditioned BFRP code 43 and 51, respectively, while the test specimens did not evidence any loss into their resistance at 22°C. For BASA BFRP bars 8-mm code 43 showed a slight decrease in flexural strength by average of 0.7%. On other hand, BASA code 51 showed a loss of resistance by an average of 4.4% for all specimens.



Figure 5.170: BASA BFRP bars of sizes 8 mm, BASA code 43 and 51 for transverse shear tests (Before conditioning)

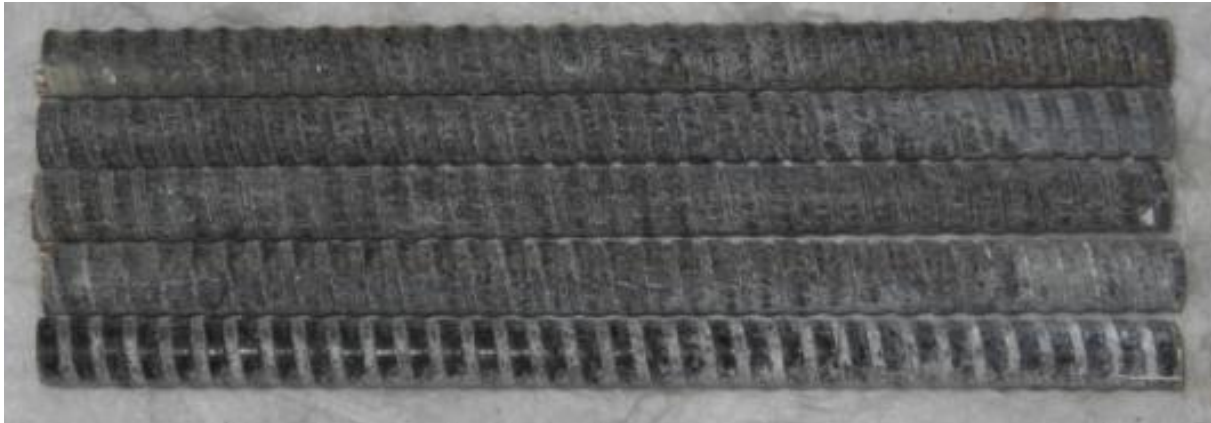


Figure 5.171: BASA BFRP bars of sizes 8 mm, BASA code 43 and 51 for transverse shear tests (After conditioning)



Figure 5.172: A typical shear failure of the tested unconditioned specimens



Figure 5.173: A typical mode of shear failure of the tested conditioned specimens



Before conditioning



after conditioning

Figure 5.174: BASA BFRP bars of 8 mm- in diameter code 43 and 51



Figure 5.175: Specimens during loading



Figure 5.176: A typical apparent horizontal shear failure of the tested BASA BFRP bars

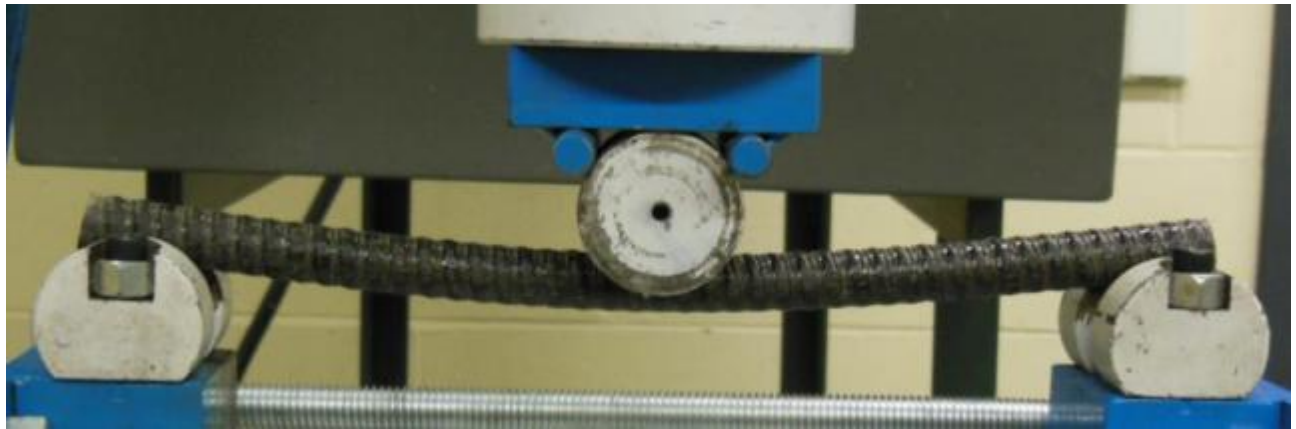


Figure 5.177: Test specimen under testing



Figure 5.178: A typical mode of failure of the tested specimens (Compression failure)

Table 5.20: Average transverse shear strength of BASA (code 43&51) - BFRP bars of 8 mm diameter (Area= 50.29 mm²)

Bar Status	BASA 43		BASA 51		Transverse Shear Strength retention of BASA 43 (%)	Transverse Shear Strength retention of BASA 51 (%)
	Transverse Shear Failure Load (kN)	Transverse Shear Strength (MPa)	Transverse Shear Failure Load (kN)	Transverse Shear Strength (MPa)		
Reference	31.7±3.8	315.3±37.5	29.5±2.8	293.0±27.7	-	-
Conditioned at 22 °C for 1 month	35.1±3.5	349.2±34.3	31.9±3.1	317.0±30.4	107.5	108.2
Conditioned at 22 °C for 3 months	-	-	33.8±6.2	336.1±61.8	-	114.7
Conditioned at 40 °C for 3 months	30.8±2.1	305.7±20.6	28.8±2.6	286.0±25.5	96.9	97.6
Conditioned at 60 °C for 1 month	28.4±1.4	282.7±13.7	26.7±1.1	265.6±11.3	89.7	90.7
Conditioned at 60 °C for 3 months	-	-	25.6±1.6	254.9±15.9	-	87.0

Table 5.21: Average apparent horizontal shear strength of BASA (code 43&51) - BFRP bars of 8 mm diameter (Area= 50.29 mm²)

Bar Status	BASA 43		BASA 51		Apparent horizontal shear strength retention of BASA 43 (%)	Apparent horizontal shear strength retention of BASA 51 (%)
	Ultimate Load, P (kN)	Apparent horizontal shear strength, S (MPa)	Ultimate Load, P (kN)	Apparent horizontal shear strength, S (MPa)		
Reference	5.5±0.2	72.4±2.9	4.5±0.2	59.9±2.4	-	-
Conditioned at 22 °C for 1 month	5.2±0.2	68.3±4.2	4.4±0.3	58.0±3.5	94.3	96.8
Conditioned at 22 °C for 3 months	-	-	4.3±0.2	56.7±2.6	-	94.7
Conditioned at 40 °C for 3 months	5.1±0.2	67.0±3.1	4.2±0.2	55.1±2.4	92.5	92.0
Conditioned at 60 °C for 1 month	5.2±0.2	69.0±3.0	4.2±0.2	56.2±2.7	95.3	93.8

Table 5.22: Average flexural strength of BASA (code 43) - BFRP bars of 8 mm diameter (Area= 50.29 mm²)

Bar Status	BASA 43				
	Ultimate Load, P (kN)	Flexure strength, S (MPa)	Modulus of elasticity, E_b (GPa)	Maximum fiber strain, ϵ (%)	Flexural strength retention (%)
Reference	3.03±0.3	2060.9±215.8	84.6±8.6	2.5±0.47	-
Conditioned at 22 °C for 1 month	3.15±0.2	2145.2±124.5	83.7±2.5	2.6±0.22	104.1
Conditioned at 40 °C for 3 months	3.01±0.2	2045.6±199.1	75.3±5.9	2.7±0.42	99.2
Conditioned at 60 °C for 1 month	3.01±0.3	2047.0±197.1	72.7±2.7	2.8±0.34	99.3

Table 5.23: Average flexural strength of BASA (code 51) - BFRP bars of 8 mm diameter (Area= 50.29 mm²)

Bar Status	BASA 51				
	Ultimate Load, P (kN)	Flexure strength, S (MPa)	Modulus of elasticity, E_b (GPa)	Maximum fiber strain, ϵ (%)	Flexural strength retention (%)
Reference	2.33±0.2	1587.4±110.7	74.0±4.2	2.2±0.20	-
Conditioned at 22 °C for 1 month	2.37±0.2	1615.0±125.7	75.5±5.6	2.2±0.27	101.7
Conditioned at 22 °C for 3 months	2.39±0.1	1627.1±47.5	71.9±5.2	2.3±0.12	102.5
Conditioned at 40 °C for 3 months	2.32±0.2	1580.3±105.7	63.2±2.6	2.5±0.15	99.6
Conditioned at 60 °C for 1 month	2.22±0.2	1507.4±156.9	63.7±1.3	2.4±0.23	95.0
Conditioned at 60 °C for 3 months	1.90±0.2	1295.5±153.5	63.0±1.7	2.1±0.28	81.6

BASA BFRP bars 12-mm in diameter (code 43)

A total of 18 specimens of BASA BFRP bars 12-mm in diameter (code 43) were tested under transverse shear, interlaminar shear and flexural test (6 specimens each test). Figure 5.179 and Figure 5.180 depict the test specimens conducted for shear and flexural tests and interlaminar shear test, respectively. The nominal cross-sectional area of 113.14 mm^2 was used in the calculations of the tensile, shear, and flexural strengths. Table 5.24, Table 5.25, and Table 5.26 report the test results of the test specimens while a typical mode of failure is shown in Figure 5.181, Figure 5.182, and Figure 5.183 for transverse shear, interlaminar shear, and flexural test, respectively. The average transverse shear strength of the test specimens was an average of 281 ± 25.1 , which indicate that the test results were higher than specified limit according to (CSA S807, 2010) ($>160 \text{ MPa}$). On the other hand, BASA BFRP specimens with 12-mm in diameter provided an average interlaminar shear strength of 65 MPa and a flexural strength of $1721 \pm 156.4 \text{ MPa}$.

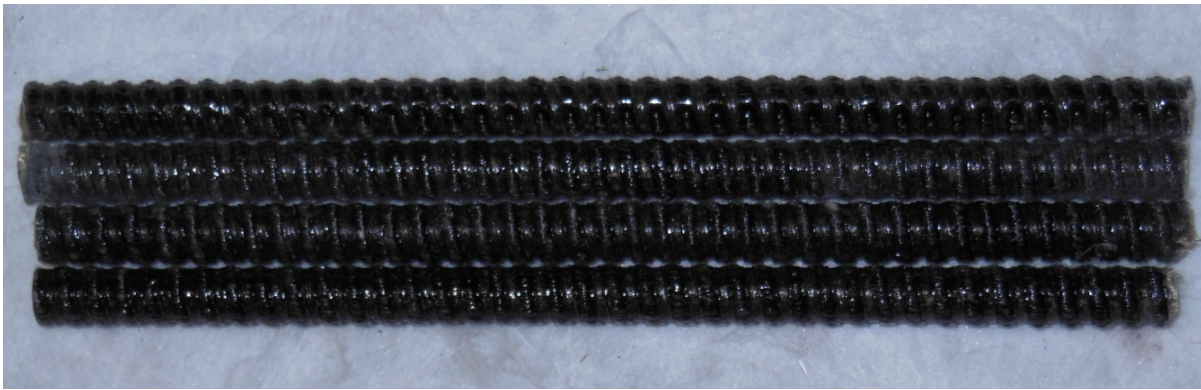


Figure 5.179: BASA BFRP bars of sizes 12 mm (code 43), for shear and flexural tests



Figure 5.180: BASA BFRP bars of sizes 12 mm (code 43)



Figure 5.181: Transverse shear failure of the tested BASA BFRP bars



Figure 5.182: Apparent horizontal shear failure of the tested BASA BFRP bars



Figure 5.183: Typical flexure mode of failure of the tested BASA BFRP bars (compression failure)

Table 5.24: Transverse shear strength of BASA BFRP bars -12 mm diameter (BASA 43) – (Area= 113.14 mm²)

No.	Size (mm)	Area (mm ²)	Transverse Shear Failure Load (kN)	Transverse Shear Strength (MPa)
1	12	113.14	56.14	248.09
2			67.77	299.49
3			65.76	290.61
4			59.26	261.88
5			61.49	271.74
6			71.36	315.35
		Average	63.63	281.19
		SD	5.68	25.09
		COV %	8.92	8.92

Table 5.25: Apparent horizontal shear strength of BASA 43 bars of size 12 mm (Area= 113.14 mm²)

Length (mm)	Span, L (mm)	Diameter, d (mm)	L/d	Ultimate Load, P (kN)	Apparent horizontal shear strength, S (MPa)
83.44	72	12	6.00	11.56	68.2
83.57	72	12	6.00	10.51	62.0
83.44	72	12	6.00	11.31	66.7
83.54	72	12	6.00	10.81	63.7
83.60	72	12	6.00	10.41	61.4
82.94	72	12	6.00	11.41	67.3
Average				11.00	64.86
SD				0.49	2.89
COV (%)				4.46	4.46

Table 5.26: Flexure properties of BASA 43 of size 12 mm (Area= 113.14 mm²)

No.	Length (mm)	Span, L (mm)	Diameter, d(mm)	L/d	Ultimate Load, P (kN)	Mid-span deflection, Y (mm)	Flexure strength, S (MPa)	Modulus of elasticity, E_b (GPa)	Maximum fiber strain, ϵ
1	270	246	12	20.5	4.25	15.31	1540.1	84.6	0.0182
2	270	246	12	20.5	4.96	19.26	1797.4	78.4	0.0229
3	270	246	12	20.5	5.45	19.36	1974.9	85.7	0.0230
4	270	246	12	20.5	4.37	15.27	1583.6	87.1	0.0182
5	270	246	12	20.5	4.70	17.39	1703.2	82.3	0.0207
6	270	246	12	20.5	4.77	17.53	1728.5	82.9	0.0209
Average					4.75	17.35	1721.3	83.5	0.0206
SD					0.43	1.80	156.4	3.06	0.0021
COV (%)					9.09	10.37	9.09	3.67	10.37

5.3.2.3 Bond strength properties

Fifteen specimens were cast using a concrete batch of 35 MPa compressive strength after 28 days and reinforced with prestressing BFRP bars 7-mm and BASA BFRP bars 8-mm (code 43 and 51). For all the specimens were failed under pure pull-out failure whereas the bar is pulled out of the specimen without any splitting/cracking in the concrete. Figure 5.184 and Figure 5.185 show the mode of failure of the test specimens of prestressing BFRP bars 7-mm and BASA BFRP bars 8-mm (code 43 and 51), respectively. As can be seen that the 7-mm and 8-mm-diameter prestressing and BASA BFRP bars (code 43 and 51) exhibited an average bond stress of 7.0 ± 0.9 , 25.4 ± 2.3 , and 27.2 ± 2.4 MPa, which are higher than the specified limit of (8 MPa) as specified by (CSA S807, 2010), except for prestressing BFRP bars 7-mm. The calculated bond stress of the prestress bars of size 7 mm and BASA BFRP bars (code 43 and 51) are listed in Table 5.27. The bond stress-free end slip relationships of the tested specimens are shown in Figure 5.186, Figure 5.187, and Figure 5.188.



Figure 5.184: Mode of failure of the pullout specimens of prestressing BFRP bars 7-mm in diameter

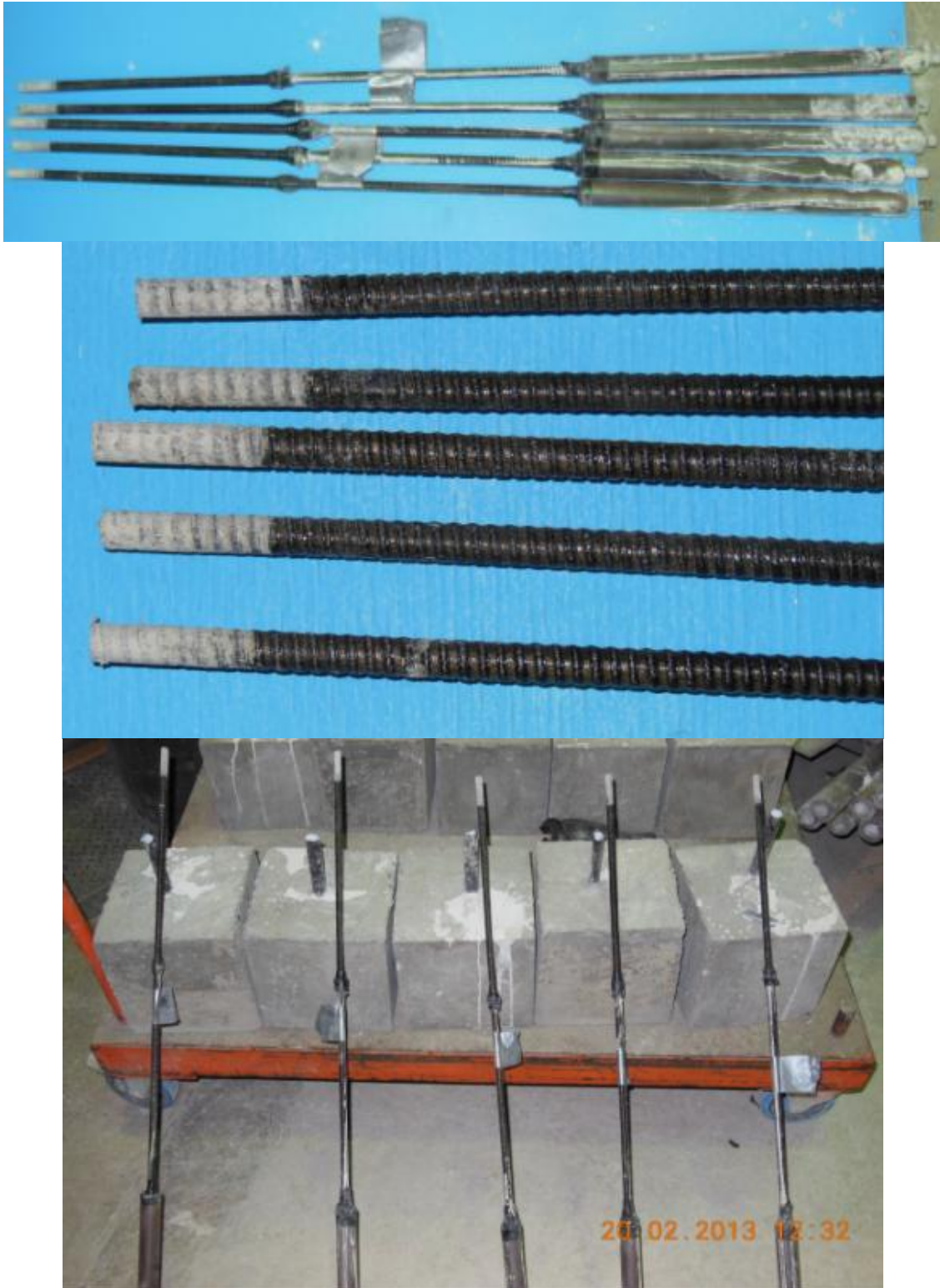


Figure 5.185: Typical mode of failure of the pullout specimens of BASA BFRP bars 8-mm (code 43 and 51)

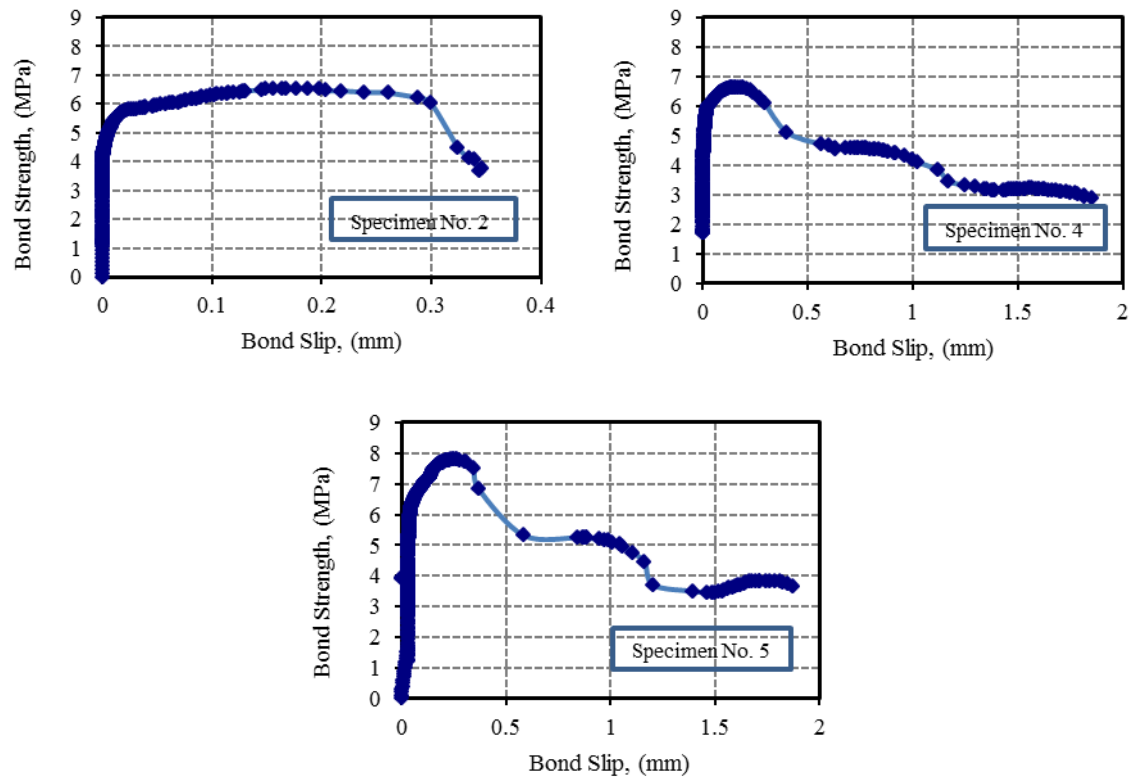


Figure 5.186: Bond stress-free end slip relationship for prestress BFRP bars of size 7 mm

Table 5.27: Bond stress of prestressing BFRP bars 7-mm and BASA BFRP bars 8-mm (code 43 and 51)

Specimens	Prestressing 7-mm		BASA (Code 43)		BASA (Code 51)	
	Failure Load (kN)	Bond Stress (MPa)*	Failure Load (kN)	Bond Stress (MPa)*	Failure Load (kN)	Bond Stress (MPa)*
1	6.12	7.95	26.86	26.70	26.22	26.07
2	5.04	6.55	24.71	24.57	29.76	29.59
3	4.48	5.81	23.32	23.19	28.02	27.86
4	5.12	6.65	23.97	23.84	29.00	28.84
5	6.01	7.81	28.80	28.63	23.89	23.75
Average value	5.35	6.95	25.53	25.39	27.38	27.22
SD	0.70	0.91	2.26	2.25	2.36	2.35
C.O.V (%)	12.99	13.04	8.85	8.85	8.62	8.62

* Note: calculated based on 7 or 8 mm diameter and embedment length of $(5 d_b)$

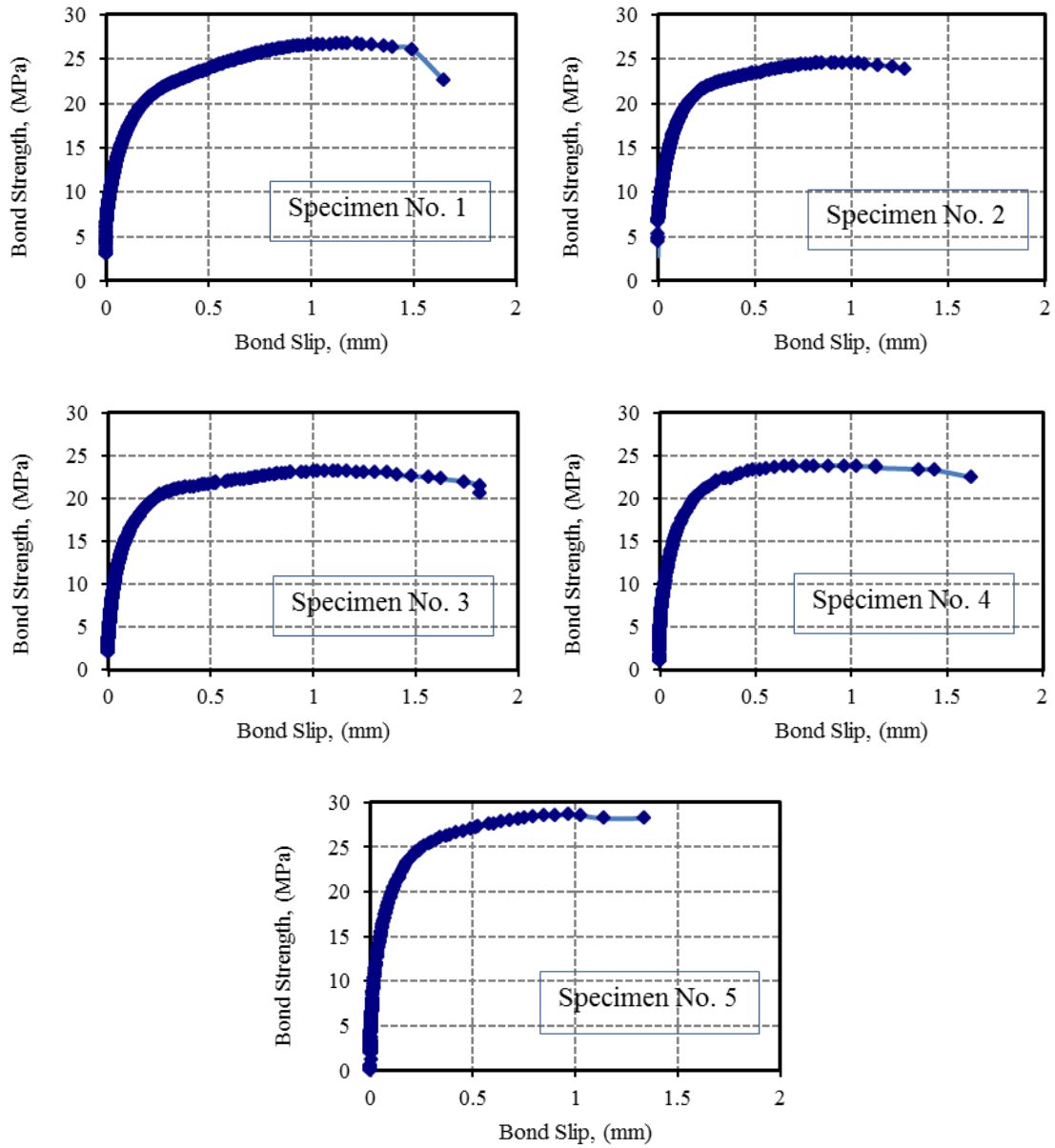


Figure 5.187: Bond stress-free end slip relationship for BASA BFRP bars of size 8 mm (Code 43)

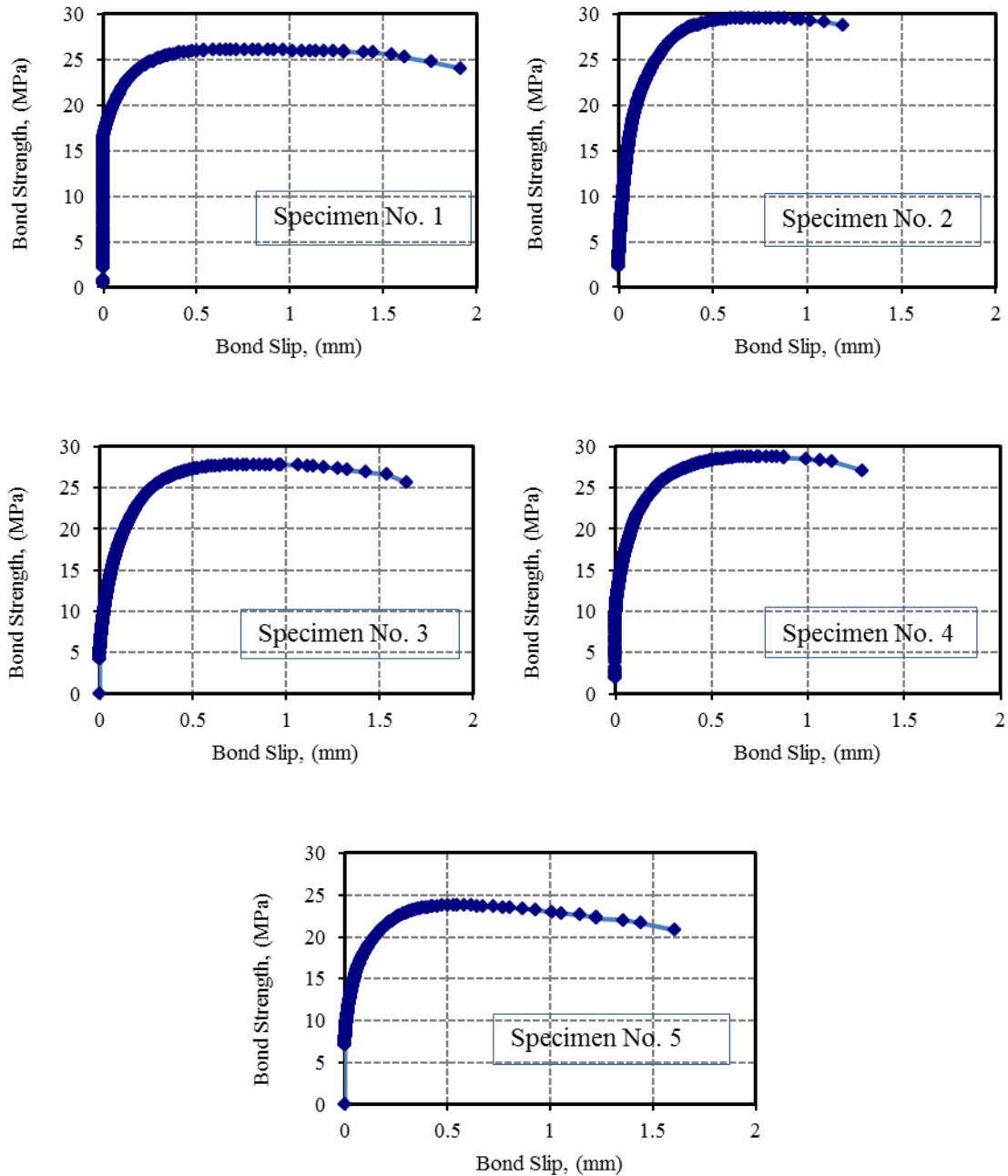


Figure 5.188: Bond stress-free end slip relationship for BASA BFRP bars of size 8 mm (Code 51)

5.4 Bond Strength of (BFRP) Bars Type I and J of Beam-End specimens

This section summarizes the test results of beam-end specimens used to determine the bond length for two types of basalt fiber reinforced polymer (BFRP) bars. The objectives of this preliminary test were to investigate the bond characteristics of the BFRP bars and to obtain estimate values of the respective development lengths of each bar type. Bar types “I” (known commercially as Reinforce Tech bars) and J are shown in Figure 5.189 and Figure 5.190, respectively.



Figure 5.189: BFRP bar type I



Figure 5.190: BFRP bar type J

5.4.1 Fabrication of the Test Specimens

The selection of the development length, l_d , of each bar type was made based on the following expression developed by (ACI440.1R, 2006) for glass and carbon bars:

$$\ell_d = \frac{\alpha \frac{f_{fr}}{\sqrt{f'_c}} - 340}{13.6 + \frac{C}{d_b}} d_b \quad (5.15)$$

Where f_{fr} is the guaranteed rupture strength of the BFRP bar (rupture strength reduced by three times the standard deviation) and d_b is the effective diameter of the bar. C represents the concrete cover measured from the surface of the concrete to the center of the BFRP bar and α is an adjustment for “top cast” bars (taken as 1.0 in this case since the specimens were cast with the BFRP bars at the bottom of the forms). For the size of bars considered in this study, the predicted development length was 30”. Based on this preliminary assessment, four specimens were de-

signed with bonded lengths of 15", 24", 40", and 52" for each type of bar, resulting in a total of eight specimens.

Four bars, from each type, were randomly selected from stock. A 15" steel tube was attached to one end of each BFRP bar and bonded to the bar using epoxy to provide a "grip" mechanism during the applied tension loads. All bars were then cut to a length of 86" (including the 15" steel tube grip). Two PVC pipe sections were then slipped over each BFRP bar to act as bond breakers at the appropriate locations. The first pipe section prevented bond over the first four inches of concrete nearest the loaded end to avoid a localized cone-type failure of the concrete. The second pipe section varied in length according to the desired bonded length for each specimen. The interior ends of each PVC bond breaker were sealed with caulk to prevent seepage of concrete into the debonded region.

A concrete beam-end length of 60" was chosen to accommodate the longest bonded length. A beam depth of 24" was selected to provide adequate distance between the bonded bar and the compression stresses developed during testing. A width of 12" was used to provide ample concrete areas to support the reactions developed during testing. Figure 5.191 depicts the position of the BFRP bar within the concrete.

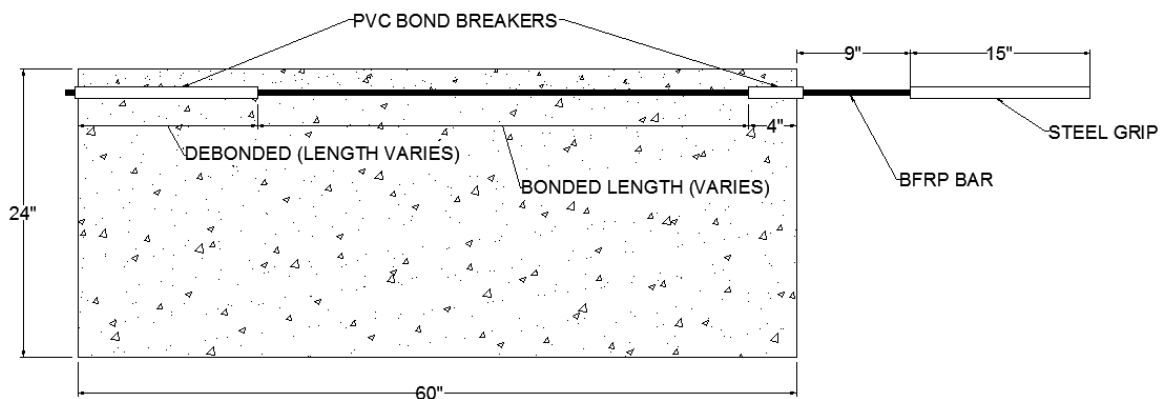


Figure 5.191: Elevation view of specimen

Longitudinal reinforcement was provided at the top and bottom of the beam section to carry and distribute the tensile stresses induced by the tension force in the BFRP bar and to assist the concrete in resisting the compressive stresses induced by the support reactions. Closed stirrups were placed parallel to the long dimension of the beam and used as transverse reinforcement. The orientation of the transverse reinforcement provided shear strength without confining the concrete surrounding the BFRP bar, as the effect of confinement was not a test variable to be investigated. Two lifting bars were placed perpendicular to the long dimension of the beam specimen for handling and positioning of the specimens. Figure 5.192 and Figure 5.193 illustrate the steel reinforcement details in relation to the BFRP bar.

Specimens were cast using wooden forms as shown in Figure 5.194 and Figure 5.195. The forms consisted of $\frac{3}{4}$ " plywood and 2x4's (nominal lumber dimensions) and were constructed with holes drilled in the appropriate locations for the BFRP and lifting bars. Reinforcement was tied

together outside of the form in cage assemblages and then placed into the form. The BFRP bars were inserted through the holes and tied into place.

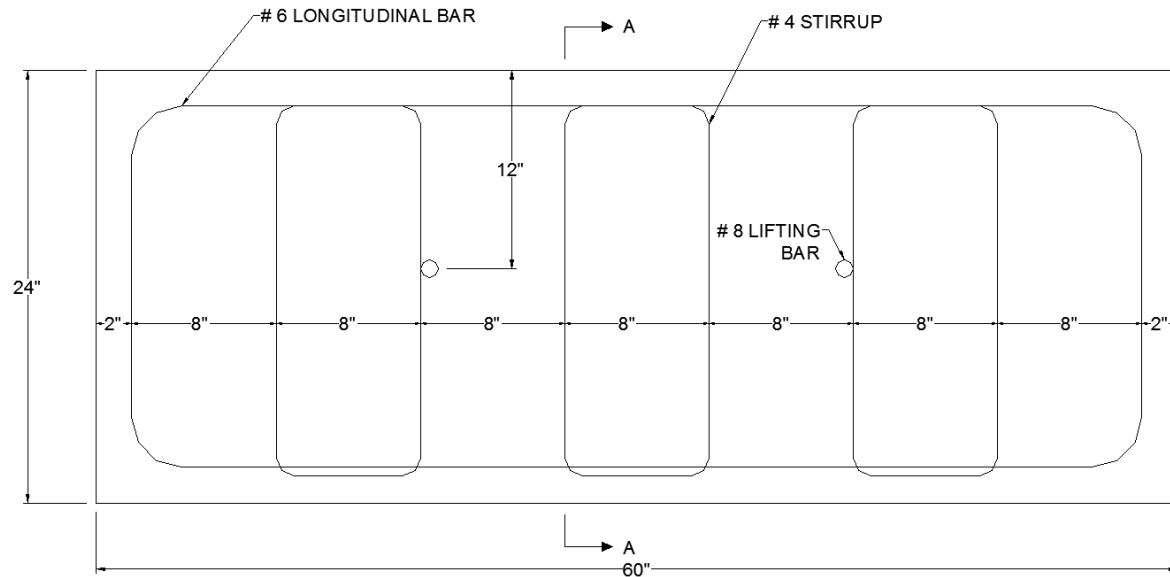


Figure 5.192: Reinforcement details of the test specimen (BFRP bar not shown)

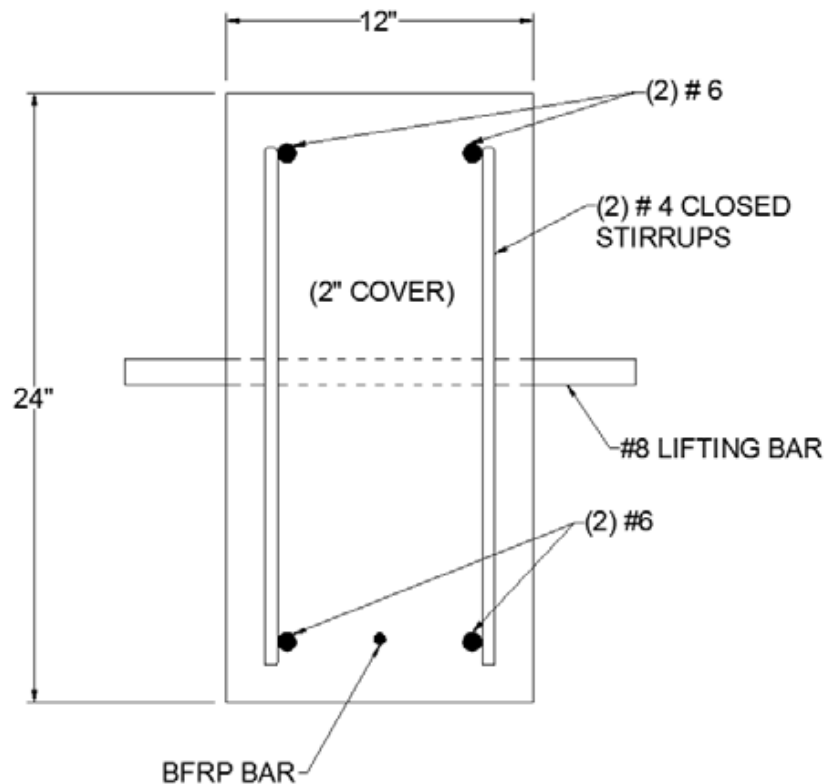


Figure 5.193: Cross-section A-A



Figure 5.194: Completed formwork

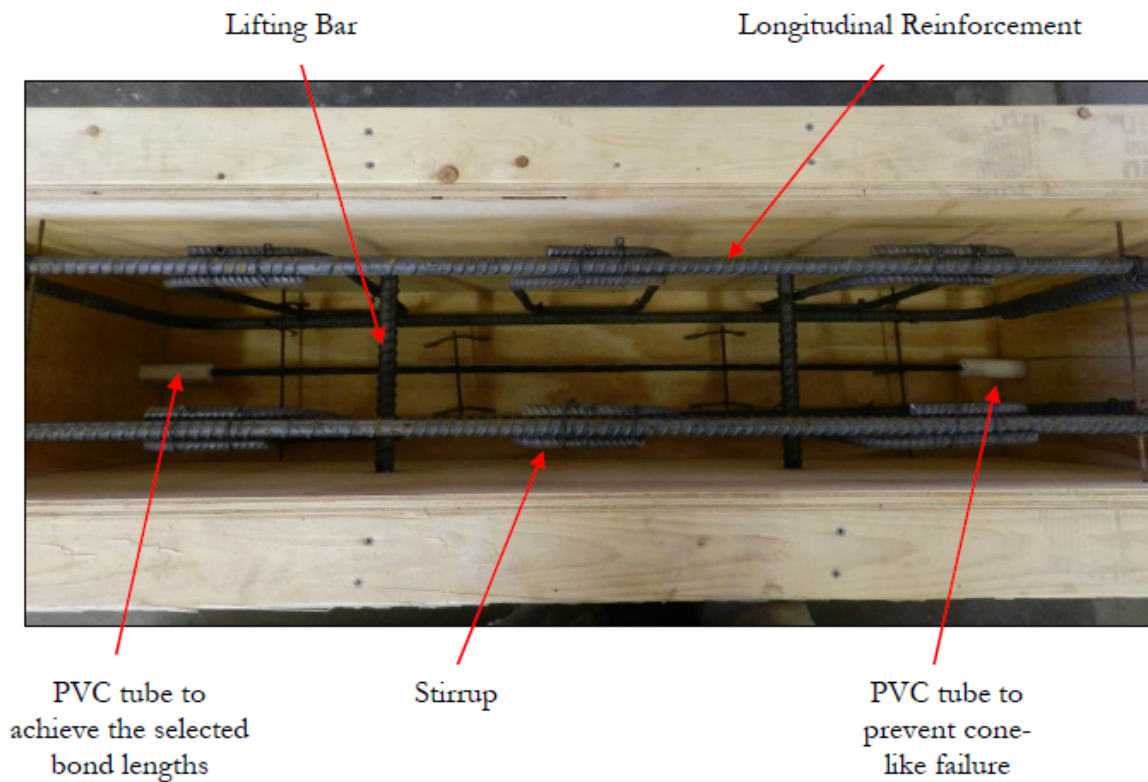


Figure 5.195: Detail of the steel reinforcement and BFRP bar

Ready mix concrete with a nominal compressive strength of 4000 psi was supplied from a local supplier. The specimens were cast in two lifts. After each lift, the concrete was compacted with a vibrator. Following the vibration of the second lift, the concrete was levelled with a screed and finished with a trowel. Several 4" x 8" concrete test cylinders were simultaneously cast from the same batch of concrete to determine the compressive strength of the concrete on respective days of testing. Figure 5.196 shows the specimens after casting. The formwork was stripped from the specimens 10 days after casting. Upon stripping the formwork, concrete honeycombing was visi-

ble at some corners of the test specimens due to inadequate consolidation of the concrete. These areas were filled with grout prior to testing.



Figure 5.196: Casted specimens

5.4.2 Test Setup

A test setup was designed to hold the concrete portion of the specimen stationary while the axial tension load was applied to the grip-end of the BFRP bar. Load was applied by a hydraulic jack and measured by a load cell. Two linear potentiometers were positioned at the loaded face of the specimen and attached to the BFRP bar. Similarly, two linear potentiometers were positioned at the opposite face of the specimen and attached to the free end of the BFRP bar. An extensometer was attached to the loaded end of the BFRP bar between the grip and the concrete face. The specimen rested on a steel beam with a flange width greater than the width of the specimen. An HSS member was used to tie down the specimen and prevent rotation induced from the moment couple caused by the applied force. A 2-in steel roller was welded to the test setup to provide a consistent horizontal reaction point for all tests. Figure 5.197 and Figure 5.198 shows the test setup in detail.

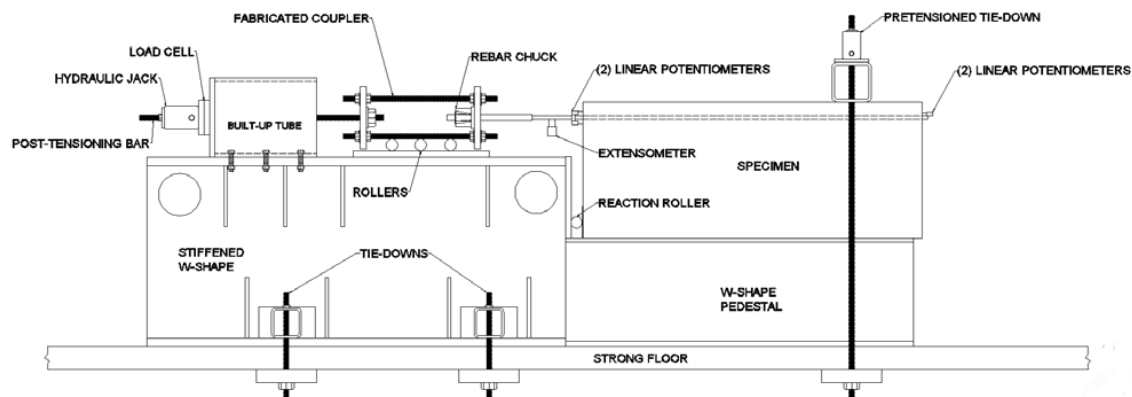


Figure 5.197: Schematic of the test setup

Care was taken in the positioning and aligning of the specimen in the test setup to minimize eccentricity of the threaded bar used to apply the load to the BFRP bar. The end of the beam was post-tensioned to the strong floor of the lab to provide the vertical reaction.



Figure 5.198: Elevation view of the test setup

5.4.3 Test Procedure

The specimen was properly positioned in the test setup and the BFRP bar was loaded in tension. A hand pump was used to increase the pressure within the hydraulic jack thereby incrementing the tension applied to the BFRP bar. Load was applied at a steady rate in each test, and the average load rate of all tests was 3.7 kips per minute. Slip of the free end of the BFRP bar was based on an average of the two free end potentiometer readings. Each specimen was tested until significant slip was detected at the free end of the BFRP bar or until the bar ruptured.

5.4.4 Test Results

The specimens were marked according to the type of BFRP bar and the bonded length as given in Table 5.28. For example, specimen A15 corresponds to BFRP bar type A with a bonded length of 15". Table 5.51 summarizes test results of all tested beams and includes the concrete compressive strength on the day of testing (average of three cylinders). Average values of rupture stress, rupture strain, and elastic modulus from tension testing are included in Table 5.28.

The measured maximum stress was determined based on the maximum measured load at failure and the bar cross-sectional area. The strain was measured by an extensometer located at the loaded end of the bar between the concrete face and the steel grip. Figure 5.199 and Figure 5.200 show the measured strain and stress for type I and J BFRP bars, respectively. In both figures, the dashed line represents the results obtained during tension tests of the bars.

In order to evaluate the development length of the BFRP bars, the maximum measured load and bonded length are shown in Figure 5.201 and Figure 5.202 for type I and J BFRP bars, respectively. In both figures, the dashed horizontal line represents the average maximum load measured during coupon tension testing for that particular bar type.

Table 5.28: Results of beam-end tests compared with tension tests values

Specimen	Failure Mode	Maximum Stress (ksi)	Calculated Modulus (ksi)	Concrete Compressive Strength (ksi)
A15	Bar Rupture	132	6294	7.56
A24	Bar Rupture	117	5919	7.56
A40	Bar Rupture	129	6770	7.46
A52	Bar Rupture	126	5952	7.46
Average of Type A Tension Tests (Low-High)	Bar Rupture	150 (146-156)	6076 (5848-6248)	---
B15	Slip/Bar Rupture	152	6169	7.78
B24	Bar Rupture	131	7534	7.05
B40	Bar Rupture	145	7334	7.05
B52	Grip Failure	118	6270	NA
Average of Type B Tension Tests (Low-High)	Bar Rupture	151 (143-163)	7033 (6585-8124)	---

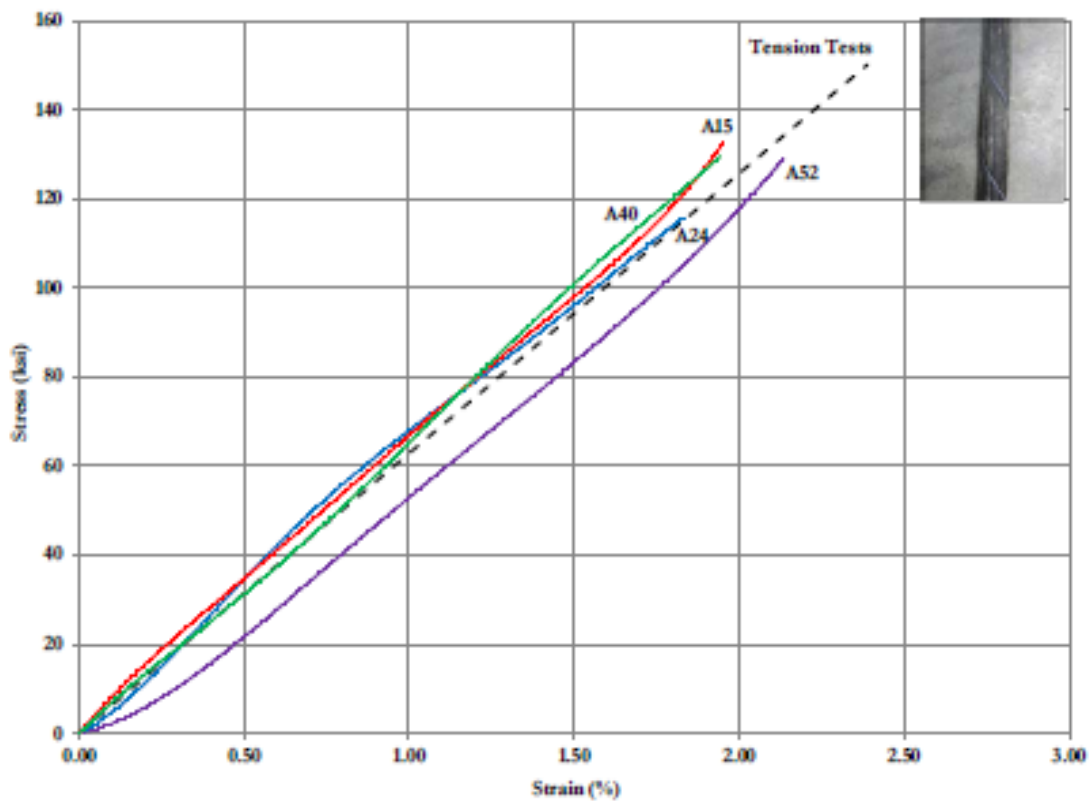


Figure 5.199: Stress-Strain response of type A BFRP bars

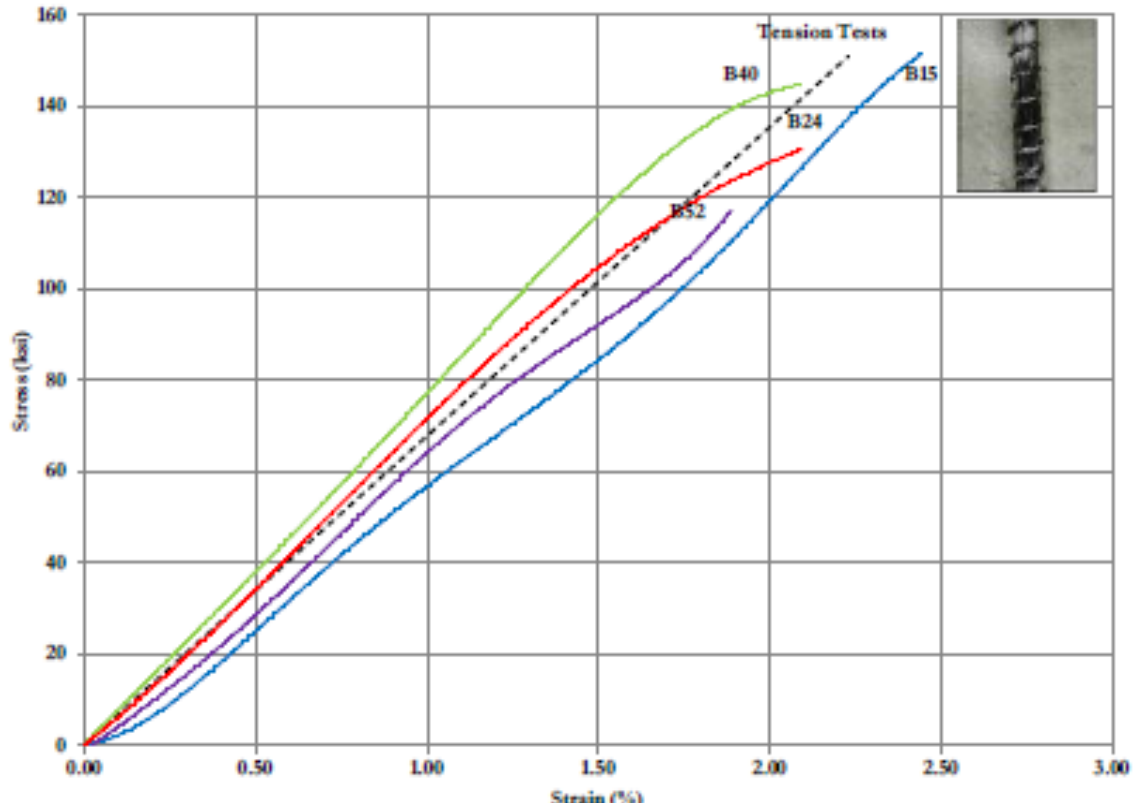


Figure 5.200: Stress-Strain response of type A BFRP bars

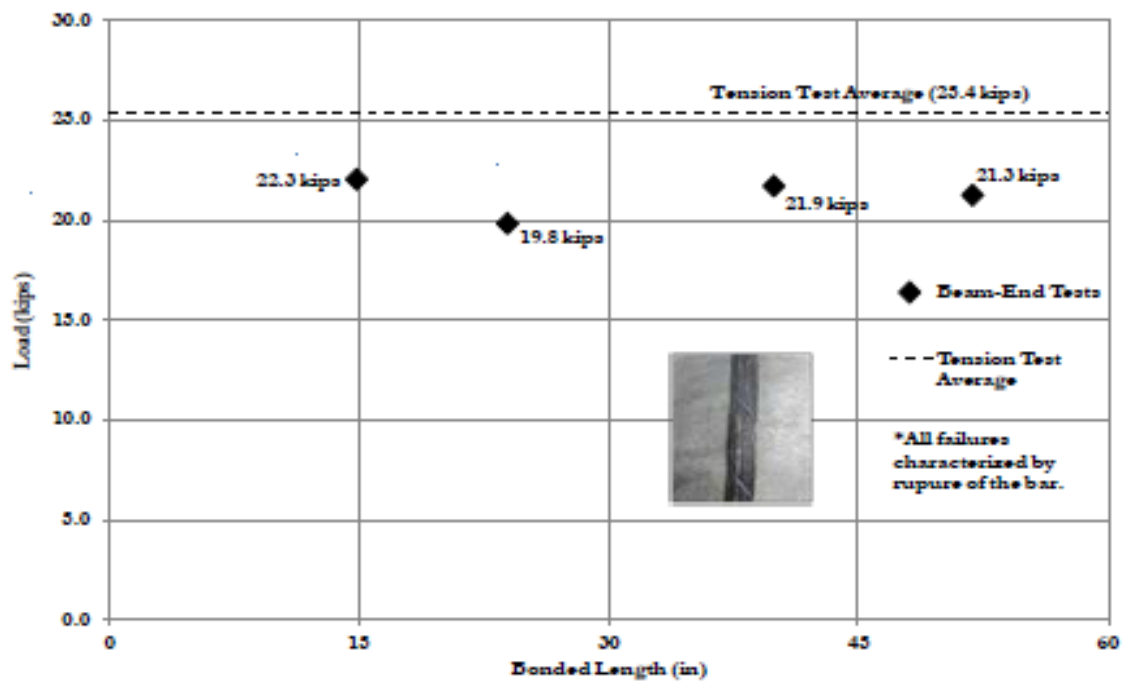


Figure 5.201: Load vs. bonded length of bar type I

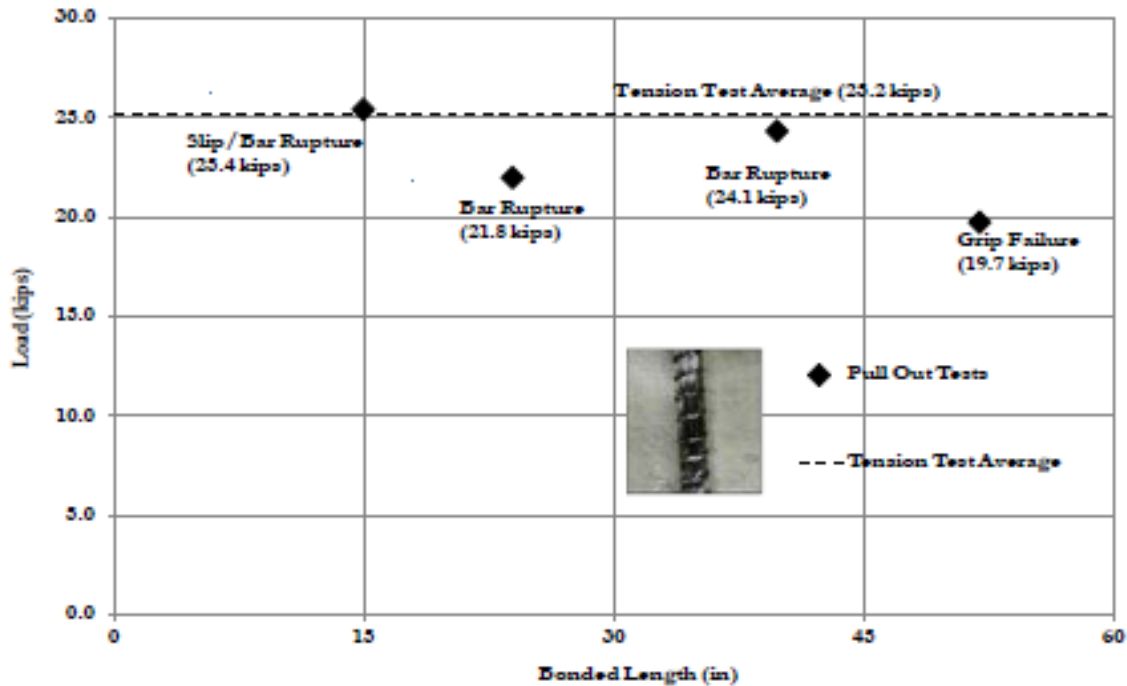


Figure 5.202: Load vs. bonded length of bar type J

5.4.5 Mode of Failure

The observed mode of failure for type A BFRP bars was rupture for all the bonded lengths used in this preliminary test. Failures of bar type A exhibited very similar qualities characterized predominantly by the gradual rupture of bar fibers at the loaded end of the specimen as shown in Figure 5.203. Specimen B52 experienced slip of the steel grip before rupture or slip of the bonded length was reached. Specimens B24 and B40 failed by bar rupture, severing completely into two pieces instantly as rupture occurred as shown in Figure 5.204. Specimen B15 exhibited slip at the free end of the BFRP bar, however, the bar sustained load until failure due to rupture of its exterior fibers at a load level equal to the ultimate tensile strength of the bar.

5.4.6 Conclusions

- Despite the difficulty experienced in casting the specimens and gripping the BFRP bars, test results suggest that the bond strength to concrete of the bars under consideration is sufficient to achieve the full strength of the bars.
- The relationship between the maximum measured load and the failure mode of the BFRP bars suggests that the development length is approximately 15" for both types of BFRP bars. This value for development length is less than the value predicted by ACI based on the bar diameter and the tensile strength of the bar.



Figure 5.203: Observed failure of type A BFRP bars



Figure 5.204: Rupture of type J BFRP bar

5.5 Experimental Testing of Concrete Beams Reinforced with Basalt (BFRP) Bars

Investigations were conducted to evaluate the short- and long-term characteristics of the new BFRP bars. This investigation aims at characterizing newly developed basalt fiber-reinforced polymer (BFRP) bars and evaluating the bond-dependent coefficient (k_b) of these bars. The investigation included physical and mechanical characterization of sand-coated BFRP bars of 10, 12, and 16 mm diameters. In addition, three beams reinforced with sand-coated BFRP bars of 10, 12, and 16 mm diameters were constructed and tested to evaluate the bond-dependent coefficient (k_b) and compare it with the current design recommendations of the FRP design codes and guides.

5.5.1 Experimental Program Outline

This study includes an experimental investigation to characterize newly developed basalt FRP bars (BFRP). The bars have a sand-coated surface over helical wire wrapping as shown in Figure 5.205. The investigation includes complete physical and mechanical characterization of three diameters of these BFRP bars (10, 12, and 16 mm). The characterization is conducted in accordance with the (ACI 440, 2008) and (CSA-S807, 2010). In addition, the bond-dependent coefficient (k_b), as one of the important design parameters, is evaluated and compared with the available design recommendations for FRP bars. The evaluation of k_b is conducted according to Annex S of the (CSA-S806, 2012) “*Test Method for Determining the Bond-Dependent Coefficient of FRP Rods.*”

5.5.2 Mechanical Characteristics of BFRP Bars

Table 5.29 shows that the tested BFRP have a modulus of elasticity close to that of GFRP bars of Grade I (CSA S807, 2010). In addition, the BFRP bars exhibited maximum strain at failure ranging from 2.4 to 2.7% which is higher than the minimum value of 1.2% (CSA S807, 2010). Furthermore, the BFRP bars showed average transverse shear strength of 234.2 ± 4.9 and 225.9 ± 14.2 MPa for 12 and 16 mm-diameter, respectively. These values are higher than the 160 MPa limit provided by (CSA S807, 2010). The bond strength was also satisfied because the three tested diameters (10, 12, and 16 mm) showed bond strengths higher than the 8 MPa specified by (CSA S807, 2010). Thus, it could be concluded that the tested BFRP bars meet the mechanical properties requirements of (CSA S807, 2010).



Figure 5.205: Basalt FRP bars

Table 5.29: Mechanical properties of BFRP bars

Diameter (mm)	Area A_f (mm ²)	Tensile modulus E_f (GPa)	Ultimate strength (MPa)	Ultimate strain (%)	Shear strength (MPa)	Bond strength (MPa)
10	79	44.4 ± 0.3	1189 ± 74	2.7 ± 0.2	---	18.0 ± 0.2
12	113	45.3 ± 0.1	1162 ± 26	2.6 ± 0.1	245.2 ± 4.9	13.8 ± 1.9
16	201	48.7 ± 0.4	1173 ± 49	2.4 ± 0.1	225.9 ± 14.2	13.5 ± 1.6

5.5.3 Bond-Dependent Coefficient (k_b)

Crack width provisions

Calculation of crack width involves a common term that is included in the predicting equation—the bond-dependent coefficient (k_b)—to account for the degree of bond between FRP bars and the surrounding concrete. Different values for k_b were introduced by the available FRP design codes and guides for the different FRP reinforcing bars. Table 5.30 provides the available k_b values from design codes and guidelines. The following are the currently available equations in North American codes and guides for predicting the crack width in FRP-reinforced concrete members:

Table 5.30: Design recommendations for k_b values of different design codes and guides

(ACI 440, 2006)	(ISIS M-03, 2007)	(CSA-S6, 2010)
k_b values ranging from 0.60 to 1.72 (mean of 1.10) Conservative value of $k_b = 1.4$ (excluding smooth bars and grids)	In the absence of significant test data $k_b = 1.2$	Sand-coated FRP: $k_b = 0.8$ Deformed FRP: $k_b = 1.0$

ACI 440.1R-06 (ACI-440, 2006)

The maximum probable crack width for FRP-reinforced concrete members is calculated as follows:

$$w = 2 \frac{f_r}{E_f} \frac{h_2}{h_1} k_b \sqrt{d_c^2 + (s/2)^2} \quad (5.16)$$

An analysis of crack width data by ACI Committee 440 on a variety of concrete cross-sections and FRP bars, fiber types, resin formulations, and surface treatments, yielded average k_b values ranging from 0.60 to 1.72, with a mean of 1.10.

CSA S6.1S1-10 (CSA-S6, 2010)

The crack width has to be verified when the maximum tensile strain in FRP reinforcement under full service load exceeds 0.0015 using the following equation:

$$w = 2 \frac{f_r}{E_f} \frac{h_2}{h_1} k_b \sqrt{d_c^2 + (s/2)^2} \quad (5.17)$$

The value of k_b should be determined experimentally, but, in the absence of test data, it may be taken as 0.8 for sand-coated and 1.0 for deformed FRP bars. In calculating d_c , the clear cover shall not be taken greater than 50 mm.

ISIS M-03 (2007)

ISIS design manual No. 3 predicts the crack width using Eq. [3]:

$$w = 2.2 \frac{f_r}{E_f} \frac{h_2}{h_1} k_b \sqrt{d_c^2 A} \quad (5.18)$$

In the absence of significant test data, $k_b = 1.2$ is recommended.

5.5.4 Beam Specimens

The bond-dependent coefficient (k_b) was determined through testing of three simply-supported beams reinforced with sand-coated BFRP of 10, 12, and 16 mm-diameter. The specimens and tests were designed in according with Annex S "Test Method for Determining the Bond-Dependent Coefficient of Fiber-Reinforced Polymer (FRP) Rods" of the CSA S806 (2012). The beams measured 3,100 mm long, 200 mm wide, and 300 mm deep. The beams were reinforced in tension with two BFRP bars (10, 12, and 16 mm-diameter) and in compression with two 10M steel bars. The beams were provided with closely spaced steel stirrups (10M @ 100 mm) in the shear spans to avoid shear failure. The constant moment zone was kept without stirrups to minimize the confinement contribution. Figure 5.206 shows the geometry and reinforcement details

of the tested beams. The mechanical properties of the BFRP were provided in Table 5.29. However, Table 5.31 provides the details of the beam specimens.

Table 5.31: Details of beam specimens

Beam	f'_c	f_t	Reinforcement	ρ_f (%)	ρ_{fb} (%)	ρ_f/ρ_{fb}	$E_f A_f$ (kN)
N2 No.3	42.5±0.4	2.83±0.05	2-10 mm bars	0.3	0.2	1.4	6952
N2 No.4			2-12 mm bars	0.4	0.2	1.8	10170
N2 No.5			2-16 mm bars	0.8	0.3	3.1	19698

The beams were constructed using a normal-weight, ready-mixed concrete with an average compressive strength of 42.5±0.4 MPa, determined from three 150×200 mm cylinders at the day of test. The tensile strength was also measured at the day of test through splitting test of three 150×200 mm cylinders and it was equal to 2.83±0.05 MPa.

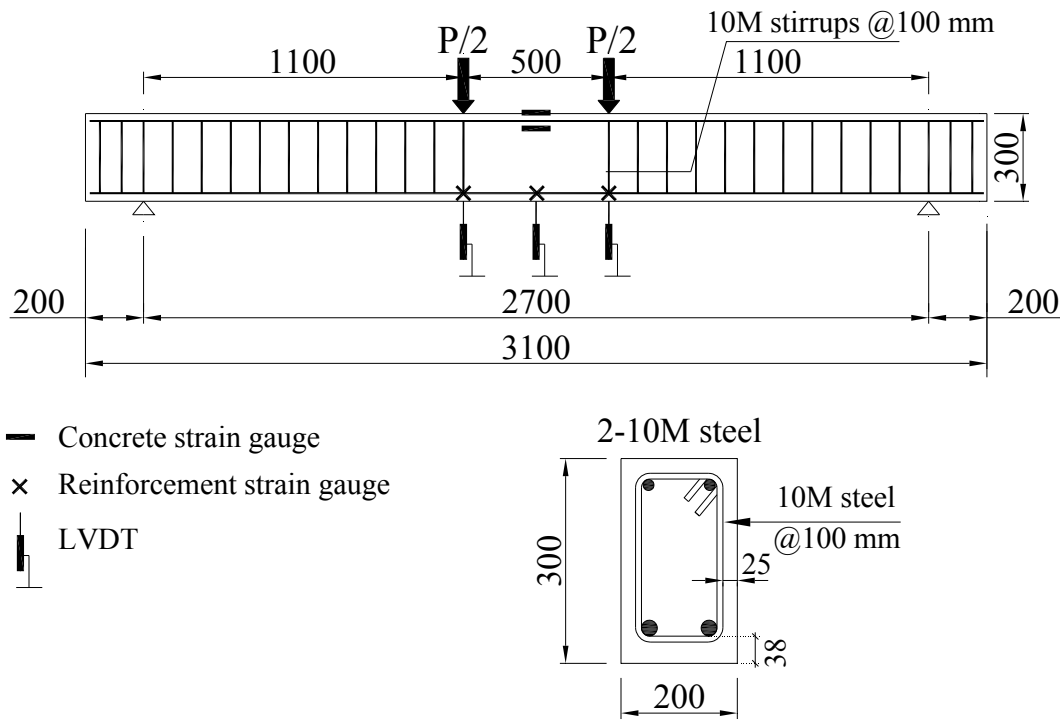


Figure 5.206: Details of beam specimens: geometry, reinforcement details, and instrumentation

5.5.5 Instrumentation and Testing

Electrical-resistance strain gauges (10 mm and 60 mm long) were used to measure the tensile strains in the reinforcing bars and the concrete compressive strains at desired locations, respectively. Reinforcement strains gauges were glued to the BFRP bars at the beam midspan and at the two loading points. Concrete strain gauges, however, were glued to the top concrete surface at beam midspan. The midspan deflection was measured with two Linear Variable Displacement Transducers (LVDTs) with a 100 mm stroke, fastened to each side of the beam while the deflection of the beam at loading points were also measured using another two LVDTs. Figure 5.206

also shows the instrumentation of the beam specimens. In addition, the widths of the first three cracks were measured with a 50 mm stroke LVDTs. A data-acquisition module monitored by a computer was used to record the readings of the strain gauges, LVDTs and load cells at one reading per second rate. During loading, crack formation along the front face of the beams was marked and the corresponding loads were recorded. The beams were tested in four-point bending over a simply supported clear span of 2,700 mm and a shear span of 1,100 mm as shown in Figure 5.206. The load was monotonically applied using a 500 kN hydraulic actuator at a stroke-controlled rate of 0.6 mm/min. The test was paused when the first three flexural cracks appeared. Their initial crack widths were measured manually with a 50X hand-held optical microscope, and three LVDTs were installed to capture the crack width evolution with the load increase. Figure 5.207 shows the test setup during a beam test.



Figure 5.207: Test setup during a beam test

5.5.6 Test Results and Discussion

Figure 5.208 shows the three tested beams at failure. As the main objective of this paper is limited to the characterization and bond-dependent coefficient (k_b) determination, the presented results and discussions in this section will be limited only to the crack widths and k_b evaluation.

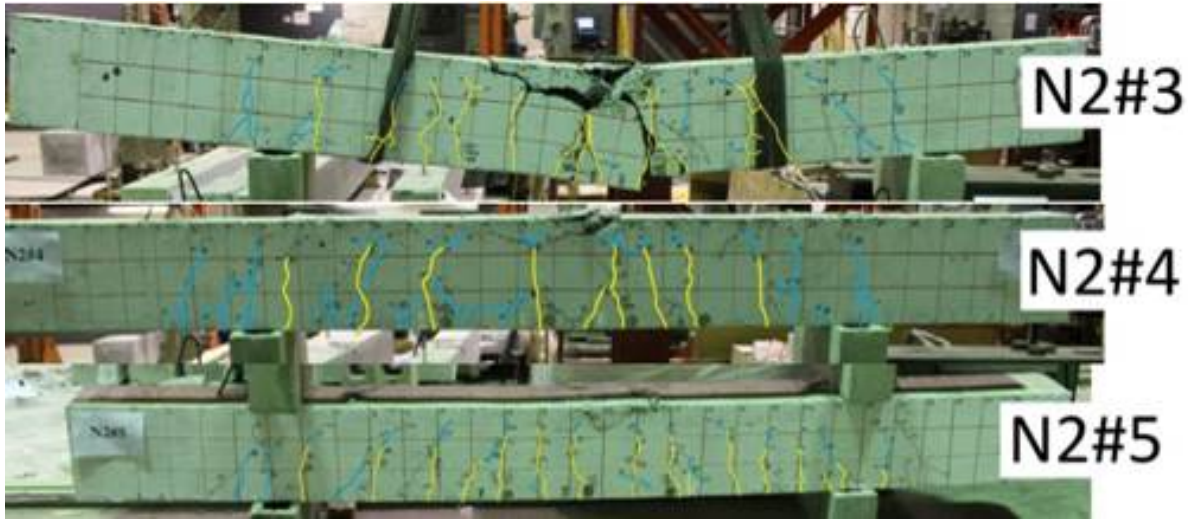


Figure 5.208: Tested beams at failure

Figure 5.209 shows the moment-crack width relationships of the three tested beams. The figure also introduces a comparison with the predicted crack widths using the available equations of (ACI 440, 2006), (ISIS M-03, 2007) and (CSA-S6, 2010). As mentioned earlier, three cracks in the constant moment zone were monitored and their widths were measured. The three cracks were used in the comparison for verifications. The k_b values that were used in the prediction were 1.4, 1.2, and 0.8 for (ACI 440, 2006), (ISIS M-03, 2007) and (CSA-S6, 2010), respectively. The comparisons in this figure indicate that (ACI 440, 2006) and (ISIS M-03, 2007) overestimate the crack widths in these beams. On the other hand, (CSA-S6, 2010) yielded good predictions for the three tested beams and the moment-crack width relationships were very close to the experimentally measured relationships.

The measured crack widths of the three tested beams were used in determining the bond-dependent coefficient (k_b) using Eqs. (5.16) to (5.18). The calculations were made at 30% of the nominal capacity ($0.3M_n$) of the testes specimens as this value was reported as the service load level by many researchers (Mota et al. 2006; Bischoff 2009; El-Nemr et al. 2011). The k_b values were calculated according to the three cracks of each beam and the average value was introduced as shown in Table 5.32. The results indicate that the k_b value of the tested BFRP bars is close to the 0.8 recommended by the (CSA-S6, 2010). Thus, the predicated crack widths using (CSA-S6, 2010) were very close to the experimentally measured ones.

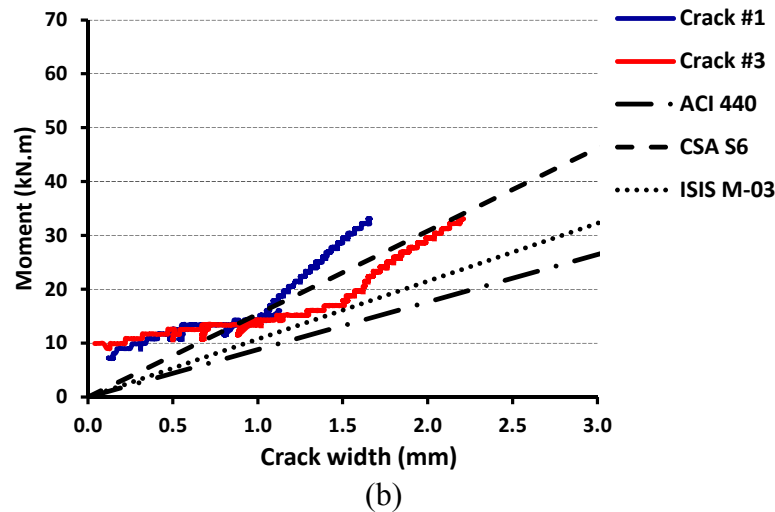
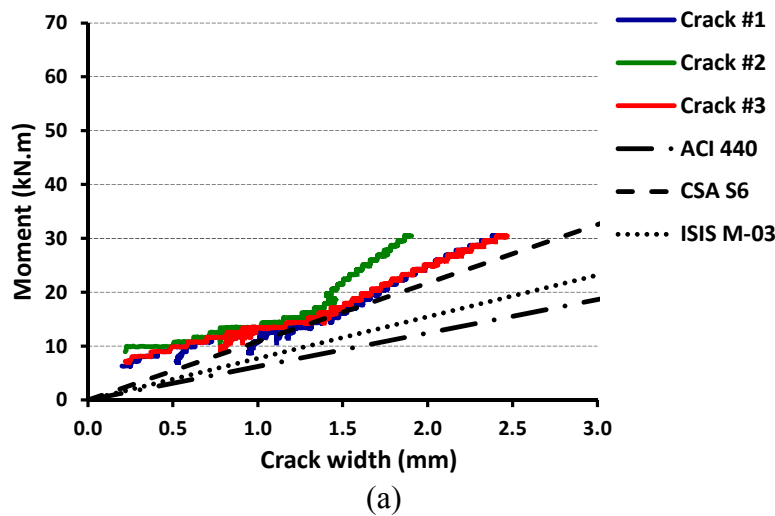
Table 5.32: Experimentally determined k_b values for BFRP bars at $0.3M_n$

Beam	Calculated k_b value			
	Crack #1	Crack #2	Crack #3	Average
N2 No.3	0.74	0.64	0.71	0.70
N2 No.4	0.79	N/A	0.97	0.88
N2 No.5	0.54	0.88	0.60	0.67
			Average	0.74

5.5.7 Concluding remarks

This study presented the result of an investigation conducted to characterize newly developed sand-coated basalt fiber-reinforced polymer (BFRP) bars and evaluate their bond-dependent coefficient (k_b). Based on the test results presented herein, the following conclusions are drawn:

- This preliminary study confirms that the developed basalt FRP (BFRP) bars meet the requirements of ACI 440 (2008) and CSA S807 (2010) concerning their physical and mechanical properties. The long-term performance of these bars in different environments and under different exposure conditions, however, has to be investigated.
- The measured bond-dependent coefficient (k_b) for the tested BFRP bars was 0.74. This value is very close to that of the CSA S6 (2010) for sand-coated FRP bars ($k_b=0.8$). Consequently, the predicted crack widths using CSA S6 (2010) provisions were very close to the experimentally measured ones.



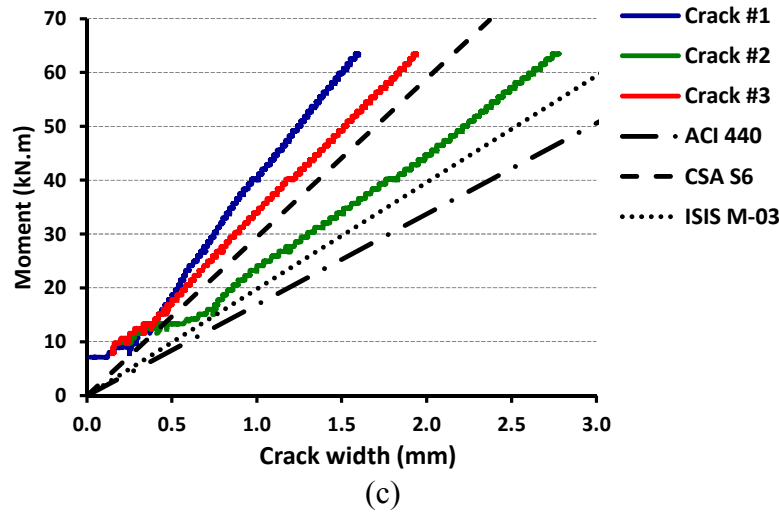


Figure 5.209: Experimental and predicted moment-crack width relationships: (a) Beam N2 No.3; (b) Beam N2 No.4; (c) Beam N2 No.5

5.6 Flexural Behavior of Concrete Members Reinforced with BFRP Bars

A total of six one-way concrete slabs reinforced with basalt fiber-reinforced polymer, BFRP, bars were cast at the Constructed Facilities Laboratory at North Carolina State University. The slabs were reinforced with varying BFRP reinforcement ratios and tested in a four-point bending configuration to study the flexural behavior of concrete members reinforced with BFRP reinforcing bars. The tests examined the serviceability and strength of concrete members reinforced with BFRP bars. The flexural behavior was assessed in terms of the load-deflection relationship, load-strain relationship, and failure mode of each specimen. Load, deflection, and strain data were recorded during testing and compared to predicted values recommended by the (ACI 440.1R, 2006) design guidelines. Test results indicate that the measured failure loads compared well with nominal flexural capacities predicted using (ACI 440.1R, 2006) equations and detailed layered-sectional analyses. The measured short-term deflections at service load level were higher than the values predicted by (ACI 440.1R, 2006). Mid-span deflections at service load level were also predicted using a numerical method of integrating curvature along the span length. The values of mid-span deflection predicted by the numerical approach compared well with the measure values. Test results indicate that the (ACI 440.1R, 2006) equations can be used safely to predict the nominal moment capacity of concrete flexural members reinforced with BFRP bars. However, it was found that the (ACI 440.1R, 2006) method under-estimated the deflection at service load level by 20% to 60%. Test results and examination of the tested specimens suggest possible localized bar slip of the BFRP reinforcement within the maximum moment region.

5.6.1 BFRP Reinforcing Bars Used for Testing

The two BFRP bars considered in this study are known commercially as “GeoTech Rebar” and “BasBar” as shown in Figure 5.210 and Figure 5.211, respectively. Ten tension coupons from each bar type were tested to determine their mechanical properties. The measured rupture strengths of the two BFRP bars were essentially identical. The selected bar for this study is the

GeoTech Rebar due to its higher modulus of elasticity. The average tensile strength of the GeoTech Rebars, $f_{u,ave}$, is 151 ksi and the average modulus of elasticity, $E_{u,ave}$, is 7,000 ksi. The average rupture strain of the bars, $\epsilon_{u,ave}$, is 2.2%. The stress-strain relationship of the GeoTech Rebars is shown in Figure 5.212. The measured average area of the cross-section of the bars is 0.167 in².



Figure 5.210: “GeoTech Rebar”



Figure 5.211: “BasBar”

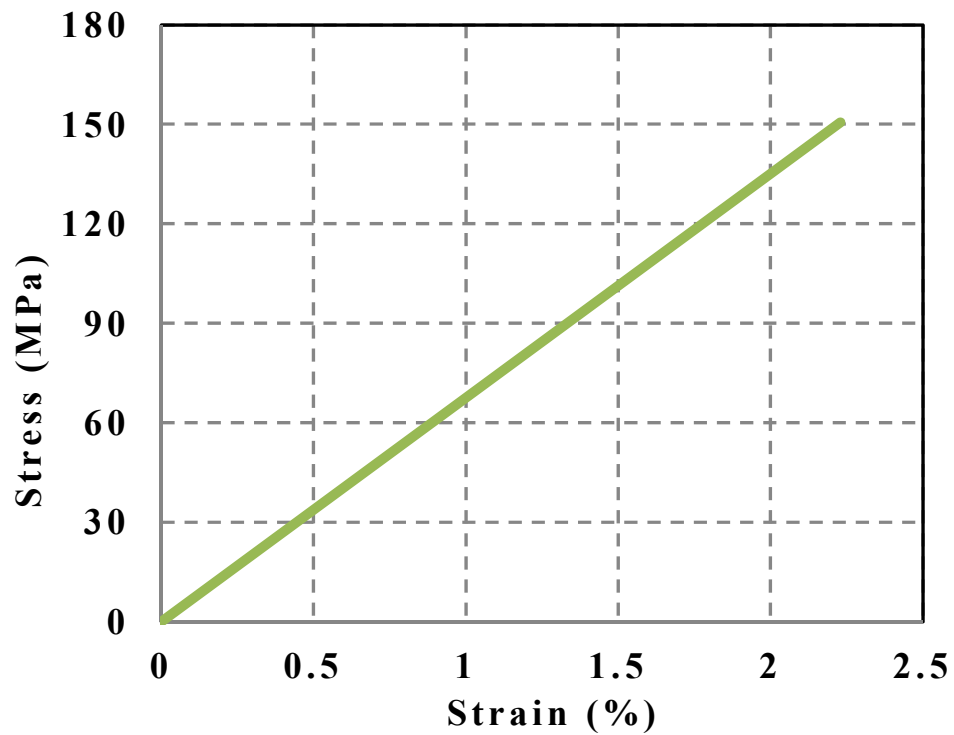


Figure 5.212: Stress-strain behavior of BFRP bars used in the flexural specimens

5.6.2 Design of the BFRP Flexural Specimens

This section describes overall dimensions and the various reinforcing ratios used in the test specimens.

Geometry of the specimens

All tested slabs shared the same cross-sectional dimensions. The height and width of each specimen were 6 in. and 24 in., respectively. The overall length of the specimens was 12 ft.

Flexural reinforcement

The BFRP bars were placed with a concrete cover of 1 inch and distributed across the width of the concrete section. Steel lifting hooks were placed at the outside quarter spans in order to transport the slabs to the test setup. No shear reinforcement was used.

The design nominal concrete compressive strength was 10 ksi. The balanced reinforcement ratio was determined based on the measured tensile properties of the BFRP bars and the selected nominal concrete strength. The reinforcement ratio and the number of specimens for each case are given in Table 5.33. Figure 5.213 shows a cross-sectional view of a specimen with balanced reinforcement.

Table 5.33: Reinforcement ratios and number of specimens.

Number of BFRP Bars per Specimen	Longitudinal Reinforcement Ratio	Number of Specimens	Designed Failure Mode
7	0.0081	2	Compression (over-reinforced)
4	0.0046	2	Balanced
3	0.0035	2	Tension (under-reinforced)

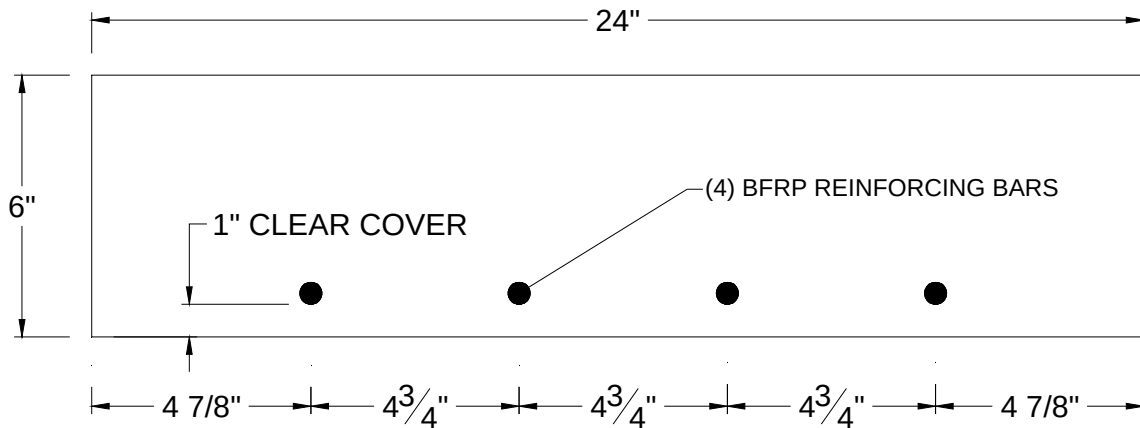


Figure 5.213: Cross-section of a flexural specimen with balanced BFRP reinforcing

5.6.3 Fabrication of the Test Specimens

This section provides a detailed description of the formwork used for casting the specimens and the placement of the BFRP bars within the formwork. It also provides the casting and curing process used. The formwork was assembled at the Constructed Facilities Laboratory and prepared for casting. The BFRP bars were trimmed $\frac{1}{2}$ in. from each end to fit into the 12 ft. long forms. The BFRP bars were supported by slab bolsters at each end and wheel spacers in-between to provide clear concrete cover along the length as shown in Figure 5.215. The No. 4 steel lifting hooks were placed near the end quarter spans.

Local ready-mix concrete was used as shown in Figure 5.214. A maximum nominal aggregate size of $\frac{3}{4}$ in. was specified to ensure satisfactory consolidation of the concrete within the concrete cover. The concrete was carefully consolidated with an internal vibrator to avoid touching the BFRP bars. The casted specimens were covered with wet burlap and plastic sheeting to cure. The formwork was stripped after one week. Multiple concrete cylinders were also cast from the same mix in order to evaluate the concrete's compressive strength at 28 days and at the time of testing.

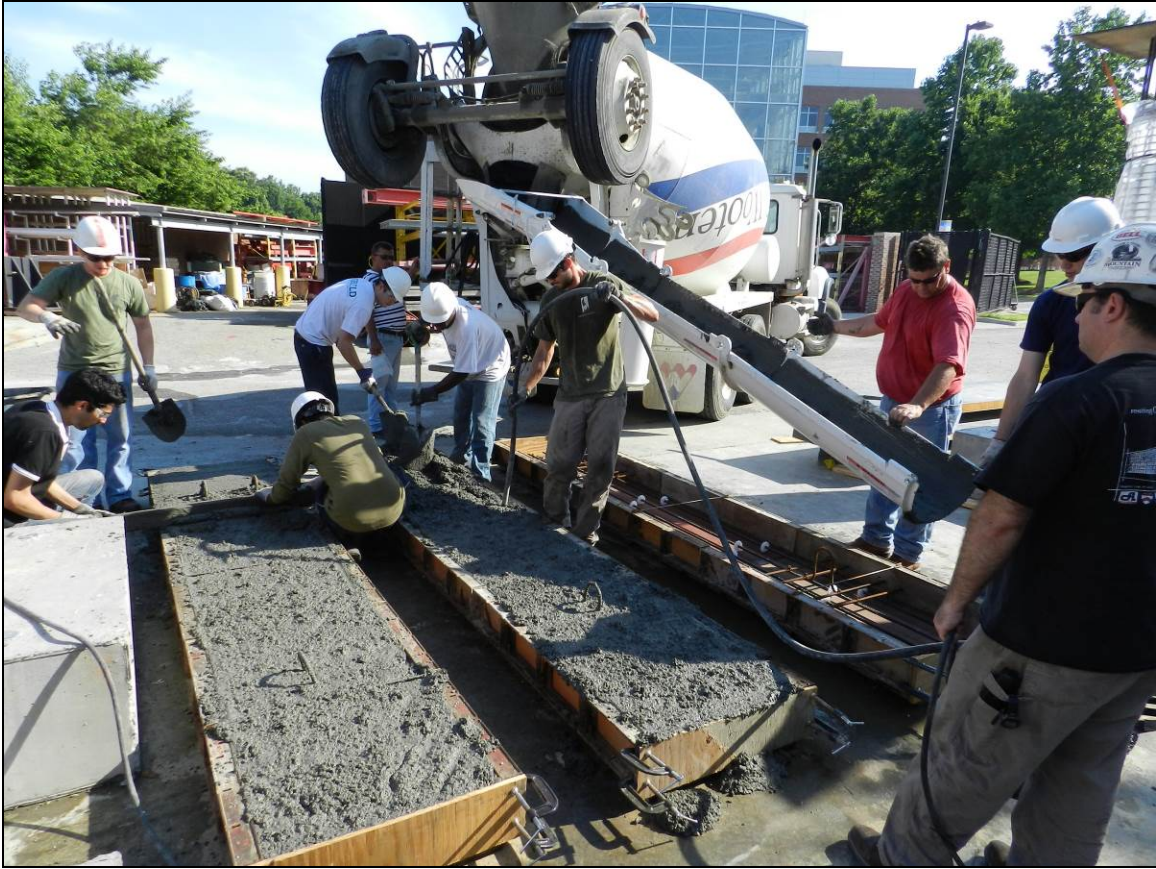


Figure 5.214: Casting of test specimens



Figure 5.215: Assembled concrete form with BFRP bars and steel lifting hooks

5.6.4 Test Setup

Four-point bending was used to investigate the flexural behavior of the test specimens. Test specimens were supported at their ends allowing a test span of 11 ft. Two concentrated loads were applied at using a hydraulic actuator. Figure 5.216 and Figure 5.217 show the loading configuration used for testing and a schematic of the test setup, respectively.

Two round steel bars were used to support the slab. Both steel bars were 2 in. in diameter. One support acted as a pin by fixing the bar between two steel angles and the other support behaved as a roller by allowing a larger gap between the steel angles. Figure 5.218 and Figure 5.219 show the pin and roller supports, respectively.

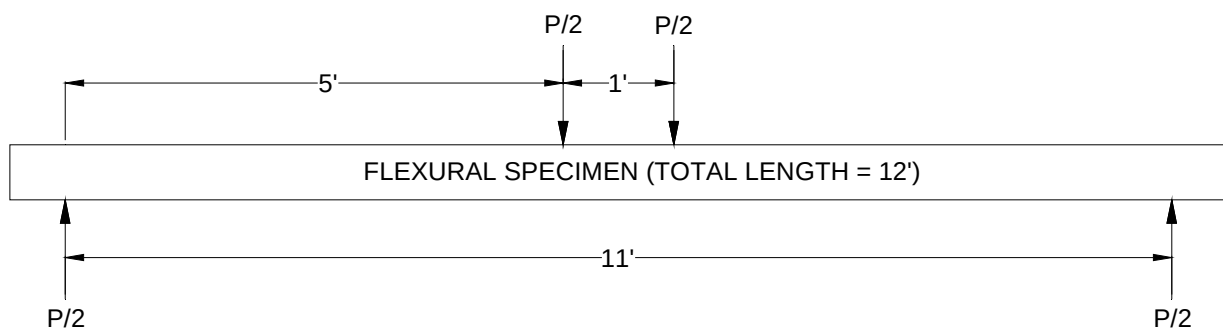


Figure 5.216: Loading configuration used for flexural testing

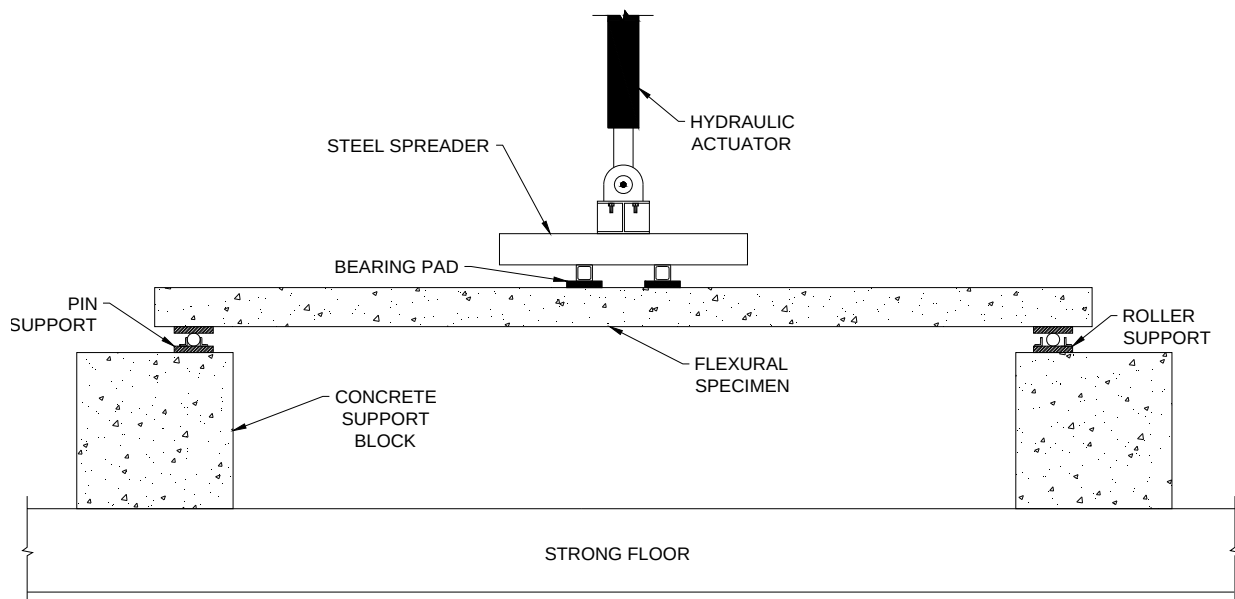


Figure 5.217: Schematic of the test setup



Figure 5.218: Pin support



Figure 5.219: Roller support

Instrumentation

Deflection at mid-span was measured using two string potentiometers. Four Pi strain gauges were positioned at mid-span in order to measure strain at the top and bottom surfaces of the concrete. An additional Pi strain gauge was positioned on the side of the test specimen at the same level as the BFRP reinforcing bars to measure tensile strain at the level of the reinforcement. Figure 5.220 shows the mid-span cross-section of the test specimen including the instrumentation.

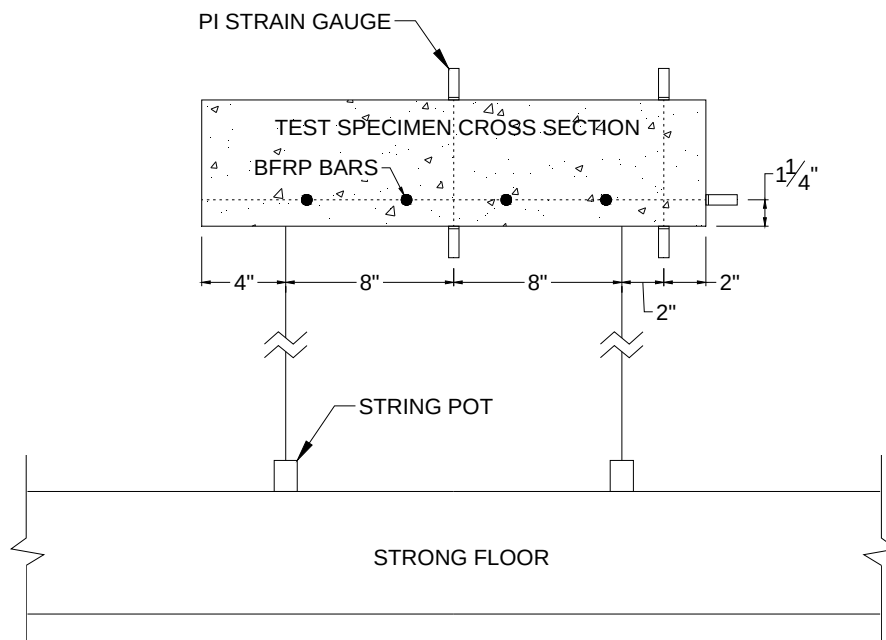


Figure 5.220: Mid-span instrumentation of the test specimen

5.6.5 Test Procedure

Load was applied using a stroke controlled actuator at a displacement rate of 0.25 in. per minute up to failure. Flexural cracks and the corresponding applied load were marked along the test specimen during testing. Displacement and strain data were recorded using an electronic data acquisition system. Test data was recorded at 2 Hz up to failure, characterized by a significant loss of applied load due to rupture of the BFRP bars and/or crushing of the concrete in the compression zone.

5.6.6 Test Results

This section summarizes all test results including compressive strength of the concrete, initiation of the first cracks, load-deflection behavior, and the ultimate load capacity of the test specimens.

Concrete compressive strength

The compressive strength of six concrete cylinders was measured at 28 days in accordance with ASTM C39: *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*. The concrete's average measured compressive strength at 28 days was 9418 psi. The compressive strength of 16 additional concrete cylinders was measured throughout the duration of flexural testing. The concrete's average measured compressive strength at the time of testing was 10456 psi. Table 5.34 and Table 5.35 give the compressive testing results of the concrete cylinders. It should be noted that the concrete's measured compressive strength at the time of testing was very close to the nominal design strength.

Table 5.34: Measured concrete compressive strength at 28 days

Cylinder #	Age (days)	Compressive Strength (psi)	Average (psi)
1	28	1,0000	9418
2	28	9530	
3	28	9560	
4	28	9120	
5	28	9030	
6	28	9270	

Table 5.35: Measured concrete compressive strength at time of flexural testing

Cylinder #	Age (days)	Compressive Strength (psi)	Average (psi)
7	57	10520	10456
8	57	10690	
9	57	10080	
10	60	10610	
11	60	10440	
12	61	10720	
13	61	10390	
14	62	10720	
15	62	10260	
16	62	10460	
17	62	10640	
18	62	10690	
19	63	10070	
20	63	10920	
21	63	10180	
22	63	9900	

Initiation of the first crack and ultimate flexural strength

The applied load and corresponding deflection at first cracking for each flexural specimen are given in Table 5.36. The maximum applied load, corresponding deflection and failure mode are also given for each flexural specimen in Table 5.36.

Table 5.36: Cracking load and deflection

Specimen		# of No. 4 BFRP Bars	First Cracking		Ultimate Capacity		Failure Mode
Design Type	ID		Load (kips)	Deflection (in.)	Load (kips)	Deflection (in.)	
Under Reinforced	UR1	3	2.38	0.05	10.65	7.11	Bar rupture
	UR2	3	3.19	0.09	10.24	7.10	
Balanced Reinforcement	BR1	4	1.89	0.07	13.33	7.63	Crushing of concrete and partial bar rupture
	BR2	4	2.25	0.06	14.00	7.72	
Over Reinforced	OR1	7	2.24	0.09	18.85	6.39	Crushing of concrete
	OR2	7	1.90	0.05	18.23	6.18	

Load-deflection behavior

The measured load-deflection response for each of the slabs is shown in Figure 5.221. Test results suggest that initiation of the first crack occurred at approximately the same load level for all tested slabs. After cracking, the behavior was linear for all tested specimens and the flexural rigidity was proportional to the BFRP reinforcement ratio. The failure load was sudden the mode of failure was brittle for all three BFRP reinforcement ratios used in this program. The following section discusses in detail the various modes of failure.

Failure mode

Failure of the under-reinforced specimens, UR1 and UR2, was due to rupture of the BFRP bars as shown in Figure 5.222 and Figure 5.223. This failure was sudden and caused complete separation of the slab at the maximum moment zone as shown in Figure 5.223. Failure of the over-reinforced specimens, OR1 and OR2, was due to crushing of the concrete in the compression zone of the maximum moment region as shown in Figure 5.225. This type of failure is a classical and desirable type of failure for concrete members reinforced with FRP bars. Failure of the two test specimens with balanced reinforcement, BR1 and BR2, was due to crushing of the concrete

compression zone of the constant moment region; however, partial rupture of the BFRP bars within these two specimens was visible as shown in Figure 5.224. This type of failure falls in the same category as the failure of the under-reinforced slabs and is ultimately undesirable.

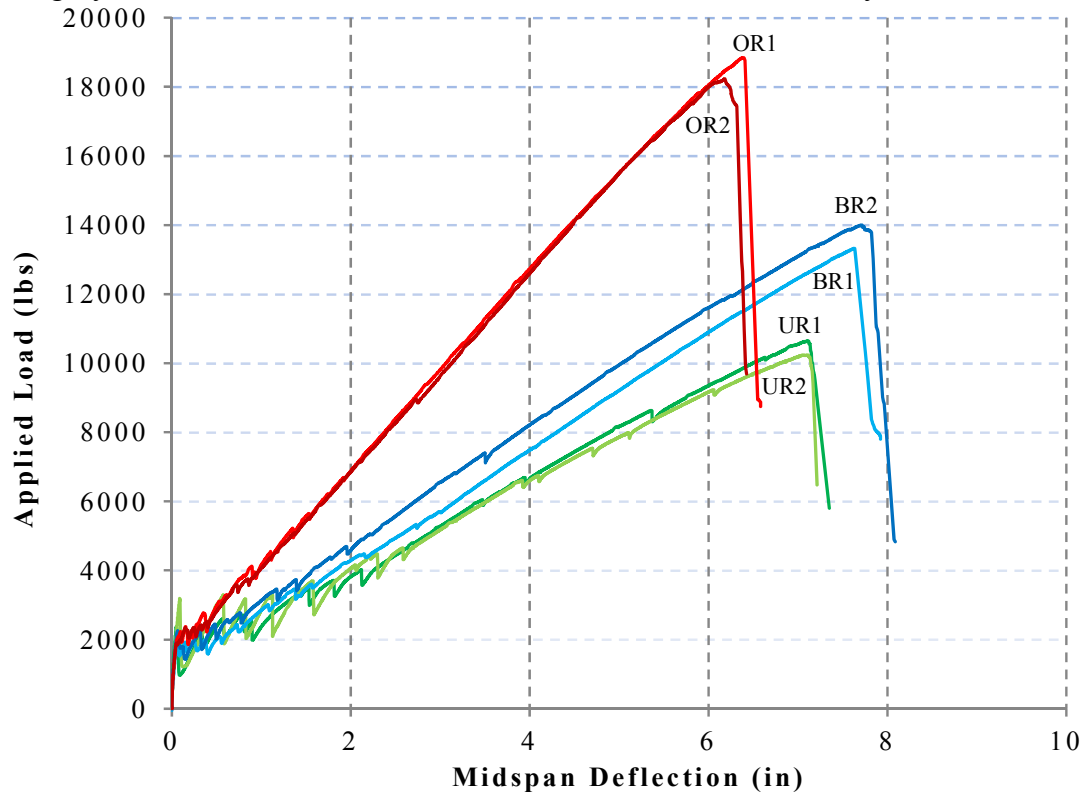


Figure 5.221: Load-deflection behavior of test specimens



Figure 5.222: Specimen UR1 following rupture of the BFRP Bars



Figure 5.223: Total collapse of the under-reinforced slab and failure of the BFRP reinforcement



Figure 5.224: Specimen BR1, crushing of concrete in compression

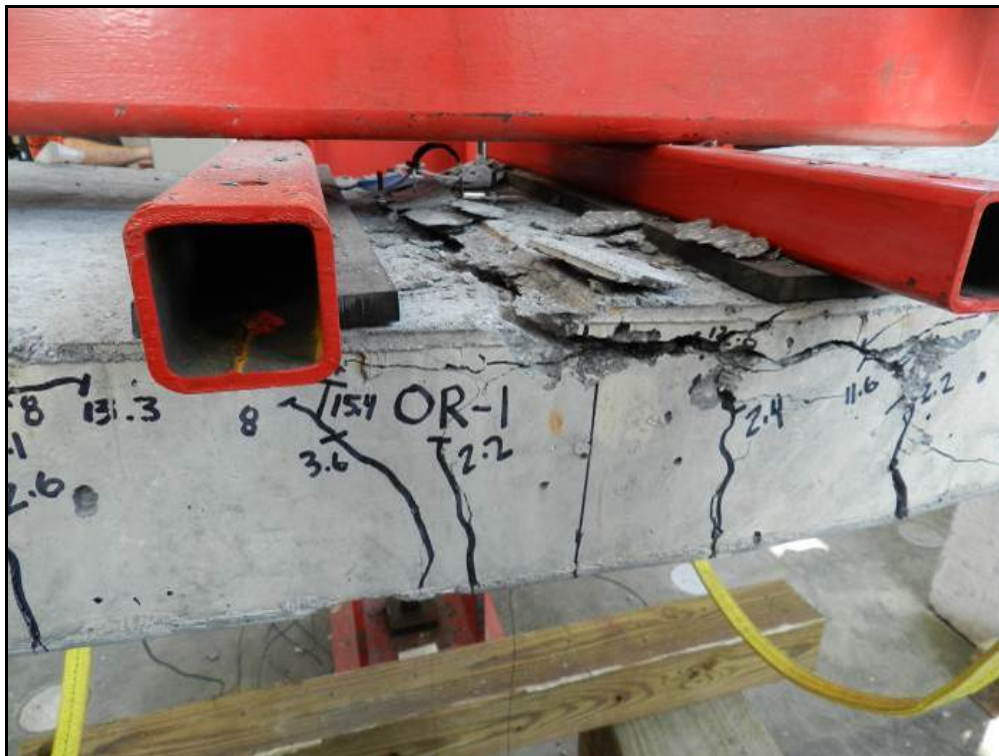


Figure 5.225: Specimen OR1, crushing of concrete in compression

5.6.7 Analysis of the Test Results

This section provides an analysis of the test results and compares the measured response to predicted values.

Cracking load and deflection

The predicted cracking moment of each flexural specimen was calculated based on the predicted rupture strength of the concrete, f_r , which was estimated as follows:

$$f_r = 7.5\sqrt{f_c} \quad (5.19)$$

Since all the specimens have the same concrete cross-sectional dimensions, the gross or uncracked moment of inertia is almost the same for all of the specimens. Therefore, the cracking moment can be estimated as follows:

$$M_{cr} = f_r \cdot S \quad (5.20)$$

where S is the elastic section modulus. The predicted cracking moment, M_{cr} , was 9.2 k-ft. The measured cracking moment is the sum of the moment due to the applied loading that initialized the first crack and the moment due to self-weight of the specimen. The predicted and the measured cracking moment for the six slabs are given in Table 5.37. The results suggest that the modulus of rupture is approximately $6.9\sqrt{f_c}$, which is within the range recommended by the ACI code of $6\sqrt{f_c}$ to $7.5\sqrt{f_c}$. Therefore, the use of current code is quite adequate.

The load-deflection behavior of the flexural specimens up to first cracking was predicted to be linearly elastic. The predicted mid-span deflection at first cracking is 0.05 in. Table 5.37 gives also a comparison of the predicted and the measured mid-span deflections at first cracking which are reasonably close.

5.6.8 Ultimate Load Capacity

Two approaches were used to predict the ultimate capacities of the test specimens. The first approach was consistent with the ACI 440.1R-06 calculation of nominal moment capacity. The second approach is based on the layered-sectional analysis.

Ultimate capacity based on ACI 440.1R-06

The balanced BFRP reinforcement ratio, ρ_{fb} , was calculated according to the ACI 440.1R-06 equation:

$$\rho_{fb} = 0.85\beta_1 \frac{f_c}{f_{fu}} \frac{E_f \epsilon_{cu}}{E_f \epsilon_{cu} + f_{fu}} \quad (5.21)$$

where:

f_c = Concrete compressive strength at the time of testing, ksi

E_f	=	Elastic modulus of the BFRP bars, ksi
f_{fu}	=	Design rupture strength of the BFRP bars, ksi
ε_{cu}	=	Maximum concrete strain, 0.003
β_1	=	Effective concrete depth factor, 0.65 for $f'_c > 8$ ksi

Table 5.37: Comparison of predicted and measured values at first cracking

Specimen Design Type	Specimen ID	Number of No.4 BFRP Bars	Predicted Crack-ing Moment (kip-ft)	Meas-ured Crack-ing Moment (kip-ft)	Predicted Mid-span Deflection at Crack-ing (in.)	Measured Mid-span Deflection at Crack-ing (in.)
Under Reinforced	UR1	3	9.2	8.2	0.05	0.05
	UR2	3	9.2	10.2	0.05	0.09
Balanced Reinforcement	BR1	4	9.2	7.0	0.05	0.07
	BR2	4	9.2	7.9	0.05	0.06
Over Reinforced	OR1	7	9.2	7.9	0.05	0.09
	OR2	7	9.2	7.0	0.05	0.05

For specimens with a BFRP reinforcement ratio greater than or equal to ρ_{fb} the flexural capacity was governed by crushing of the concrete in compression. Therefore, the stress distribution of the concrete in compression could be approximated by the ACI rectangular stress block. Based on equilibrium of forces and strain compatibility, the nominal ultimate moment capacity, M_n , can be determined as:

$$M_n = A_f f_f \left(d - \frac{a}{2} \right) \quad (5.22)$$

$$a = \frac{A_f f_f}{0.85 f'_c b} \quad (5.23)$$

$$f_f = E_f \varepsilon_{cu} \frac{\beta_1 d - a}{a} \quad (5.24)$$

where:

A_f	=	Area of the FRP reinforcement, in ²
f_f	=	Stress level in the FRP bars, ksi

a = Depth of equivalent rectangular stress block, in

The average rupture strength of the BFRP bars, $f_{u,ave}$, was used as the design rupture strength, f_{fu} , in order to better predict the failure load of the test specimens. Similarly, the average value of elastic modulus, $E_{u,ave}$, was used as the design value of elastic modulus, E_f . Substituting equation (5.23) into equation (5.24) and solving for f_f gives:

$$f_f = \left(\sqrt{\frac{(E_f \epsilon_{cu})^2}{4} + \frac{0.85\beta_1 f'_c}{\rho_f} E_f \epsilon_{cu}} - 0.5 E_f \epsilon_{cu} \right) \leq f_{fu} \quad (5.25)$$

For specimens with ρ_f less than ρ_{fb} the flexural capacity is governed by rupture of the BFRP bars. In this case, the ACI stress block is not applicable and ACI 440.1R-06 recommends using an equivalent stress block to approximate the compressive stress distribution in the concrete at a particular level of strain. The following stress block parameters were used to approximate the compressive stress distribution in the concrete:

$$\alpha\beta = \left(\frac{\epsilon_c}{\epsilon'_c} \right) - \left(\frac{1}{3} \right) \left(\frac{\epsilon_c}{\epsilon'_c} \right)^2 \quad (5.26)$$

$$\beta = \frac{\left[4 - \left(\frac{\epsilon_c}{\epsilon'_c} \right) \right]}{\left[6 - \left(\frac{2\epsilon_c}{\epsilon'_c} \right) \right]} \quad (5.27)$$

where:

- α = Ratio of the average concrete stress to the concrete strength
- β = Ratio of the depth of the equivalent rectangular stress block to the depth of the neutral axis
- ϵ'_c = Concrete compressive strain at peak stress, taken as -0.002
- ϵ_c = Concrete compressive strain at the extreme compression fiber

The nominal moment capacity of the under-reinforced specimens was calculated using the following equation:

$$M_n = A_f f_{fu} \left(d - \frac{\beta c}{2} \right) \quad (5.28)$$

where c is the depth of the neutral axis measured from the top of the cross-section. Using force equilibrium, c is computed as follows:

$$c = \frac{A_f f_{fu}}{\alpha f'_c \beta b} \quad (5.29)$$

From strain compatibility, ϵ_c can be related to the neutral axis depth, c , by the following expression:

$$\varepsilon_c = \frac{f_{fu}c}{E_f(c-d)} \quad (5.30)$$

ε_c is found iteratively by varying the depth of the neutral axis to satisfy force equilibrium.

Ultimate capacity based on layered-sectional analysis

In this approach, the layered-sectional was used to predict the nominal moment capacities of the specimens. A linear stress-strain relationship was used to model the behavior of the BFRP bars and the Hognestad constitutive stress-strain relationship was used to model the behavior of the concrete in compression. The cross-sectional height of each specimen was discretized into 300 thin layers. Using force equilibrium and assuming strain compatibility the moment-curvature response was generated for each specimen by incrementally increasing the level of extreme fiber compressive strain.

Comparison of predicted ultimate capacities

Table 5.38 gives a comparison of the measured and predicted moment capacities of each slab. Comparison of the test results to the predicted moment capacities suggests there is excellent agreement between the measured capacities and the capacities predicted by the ACI 440.1R-06 and the layered-sectional analysis approaches.

Table 5.38: Comparison of measured and predicted nominal moment capacities

Specimen Design Type (ρ_f / ρ_{fb})	Specimen ID	Measured Moment Capacity, kip-ft		Predicted Moment Capacity, M_n kip-ft (Predicted/Measured)	
		Average, $M_{n,exp}$		ACI 440.1R-06	M- Φ Analysis
Under-reinforced (0.74)	UR1	28.9	28.4	28.8	28.9
	UR2	27.9		(1.01)	(1.02)
Balanced Reinforcement (0.98)	BR1	35.6	36.5	37.9	38.0
	BR2	37.3		(1.04)	(1.04)
Over-reinforced (1.72)	OR1	49.4	48.7	48.6	50.1
	OR2	47.9		(1.00)	(1.03)

5.6.9 Deflection at Service Load

Due to the non-ductile behavior of FRP bars, ACI 440.1R-06 recommends using a conservative strength reduction coefficient, ϕ . The strength reduction coefficient varies from 0.55 to 0.65 according to the reinforcement ratio of the section. Assuming a dead load to live load ratio of 2:1, the applied moment at service load level, M_{serv} , can be related to nominal moment of the section as follows:

$$\phi M_n = 1.2 \left(\frac{2}{3} M_{serv} \right) + 1.6 \left(\frac{1}{3} M_{serv} \right) \quad (5.31)$$

Therefore, a reasonable estimate of the applied moment at service load level can be expressed as:

$$M_{serv} = \frac{\phi M_n}{1.33} \quad (5.32)$$

$$= 0.41 M_n$$

(for under-reinforced and balanced reinforcement specimens)

$$= 0.49 M_n$$

(for over-reinforced specimens)

Service load deflection based on (ACI 440.1R, 2006)

According to ACI 440.1R-06, short-term deflections that occur after a concrete member is cracked can be approximated using an effective moment of inertia, I_e . The expression for I_e is based on Branson's equation (1977) and varies between the un-cracked moment of inertia, I_g , and the cracked moment of inertia I_{cr} . The following equation is recommended for the calculation of I_e :

$$I_e = \left(\frac{M_{cr}}{M_a} \right)^3 \beta_d I_g + \left[1 - \left(\frac{M_{cr}}{M_a} \right)^3 \right] I_{cr} \leq I_g \quad (5.33)$$

where β_d is a reduction coefficient related to the reduced tension stiffening exhibited by FRP-reinforced members. β_d varies with the FRP reinforcement ratio as follows:

$$\beta_d = \frac{1}{5} \left(\frac{\rho_f}{\rho_{fb}} \right) \leq 1.0 \quad (5.34)$$

In equation (5.69) M_a represents the un-factored moment in the member and I_{cr} is the cracked cross-sectional moment of inertia. I_{cr} is calculated as follows:

$$I_{cr} = \frac{bd^3}{3} k^3 + n_f A_f d^2 (1-k)^2 \quad (5.35)$$

where n_f is the ratio of the modulus of elasticity of the FRP bars, E_f , to the modulus of elasticity of the concrete, E_c and k is a ratio of the depth of the neutral axis to the depth of the reinforcement. K is given as follows:

$$k = \sqrt{2\rho_f n_f + (\rho_f n_f)^2} - \rho_f n_f \quad (5.36)$$

The moment at service capacity was based on the average measured moment capacity, $M_{n,exp}$. The predicted mid-span deflection at service capacity, Δ_{serv} , was calculated using equation (5.39)

except I_g was replaced by I_e and P_{cr} was replaced by P_{serv} , the predicted applied load service load. $M_{n,exp}$, P_{serv} and Δ_{serv} were calculated as follows:

$$M_{serv} = \frac{M_{n,exp}}{1.5} \quad (5.37)$$

$$P_{serv} = \frac{(M_{serv} - M_{sw})}{a} \quad (5.38)$$

$$\Delta_{serv} = \frac{P_{serv} a (3l^2 - 4k^2)}{48E_c I_e} \quad (5.39)$$

Deflection based on Bischoff's Model

Bischoff proposed an expression for the equivalent moment of inertia of concrete members reinforced with steel bars or FRP bars based on the integration of curvature (Bischoff and Gross 2011). This expression for the effective moment of inertia accounts for the change in flexural stiffness along the length of the member and is given as follows:

$$I_e' = \frac{I_{cr}}{1 - \gamma \eta (M_{cr} / M_a)^2} \leq I_g \quad (5.40)$$

where η is given by:

$$\eta = 1 - I_{cr} / I_g \quad (5.41)$$

and γ is an integration factor that depends on the loading arrangement and the boundary conditions of the member. The expression for γ is given for members with and without tension stiffening by equations (5.42) and (5.44), respectively.

$$\gamma = \frac{3(a' / \ell) - 4\xi(a' / \ell)^3}{3(a' / \ell) - 4(a' / \ell)^3} \quad (5.42)$$

$$\xi = 4(M_{cr} / M_a) - 3 \quad (5.43)$$

$$\gamma = \frac{8(a' / \ell)^3 (M_{cr} / M_a)}{3(a' / \ell) - 4(a' / \ell)^3} \quad (5.44)$$

The service load deflection was calculated using an elastic analysis of the deflection due to the applied load and Bischoff's expression for the equivalent moment of inertia, I_e' .

Prediction of deflection using numerical integration

An approach for predicting the mid-span deflection using numerical integration was presented by Chen and Atsuta (1975). This approach was developed to determine the deflection curve of a

laterally loaded beam-column by numerically integrating the curvature of the member along its length.

A moment diagram was formed for each specimen by setting the moment value at mid-span equal to the service moment. The moment diagram was discretized along the length of the specimen at three inch intervals. A value of curvature from the layered-sectional analysis was assigned at each interval that corresponded to the moment at that point along the span. By selecting an initial value of slope at the specimen's end support, successive values of slope and deflection were computed at each interval based on the previous interval's curvature, slope and deflection. The predicted mid-span deflection was calculated by iteratively adjusting the slope at the specimen's end support until the computed slope at mid-span was zero.

For comparison with the measured mid-span deflection, the mid-span deflection due to the self-weight of the specimen was subtracted from the value calculated by numerical integration.

Comparison of predicted service load deflections

Table 5.39 gives a comparison of the measured and predicted mid-span deflections at service load levels.

Table 5.39: Comparison of measured and predicted mid-span deflections at service load

Specimen ID $\left(\frac{M_{serv}}{M_{cr}} \right)$	Measured mid-span deflection at service load, in. Avg $\frac{\ell}{\Delta}$			Predicted Mid-span Deflection, in. (Predicted / Measured Avg)			
				ACI 440.1R-06	Bischoff's model with tension stiffening	Bischoff's model w/o tension stiffening	Numerical Integration
UR1 (1.27)	1.98	1.9	69	0.8 (0.42)	0.5 (0.26)	1.6 (0.84)	2.0 (1.05)
UR2 (1.27)	1.87						
BR1 (1.63)	2.61	2.4	55	1.3 (0.54)	1.1 (0.46)	2.2 (0.92)	2.5 (1.04)
BR2 (1.63)	2.27						
OR1 (2.58)	2.56	2.6	51	2.1 (0.81)	2.0 (0.77)	2.6 (1.00)	2.7 (1.04)
OR2 (2.58)	2.60						

The deflection at the equivalent service load defined on page 246 is considerably higher than the typical span/deflection limit recommended by the ACI code ($\ell/\Delta \geq 240$). The approach recommended by (ACI 440.1R, 2006) under-predicted the mid-span deflection at service load for all slabs.

Bischoff's model with tension stiffening under-predicted the service load deflection for all slabs. However, using Bischoff's model without tension stiffening predicted service load deflections that were considerably close to the measured deflections.

Numerical integration of each specimen's curvature at service load predicted mid-span deflections that agreed very well with all measured deflections. Test results showed that as the BFRP reinforcement ratio increased, the predicted mid-span deflections were closer to the average measured deflections.

5.6.10 Load-Deflection Behavior

Figure 5.226, Figure 5.227, and Figure 5.228 show the predicted and measured load-deflection responses of the slabs. The service load level is marked in each figure. It should be noted that the (ACI 440.1R, 2006) expression for I_e was calibrated at service loads and is not recommended for use beyond service load levels.

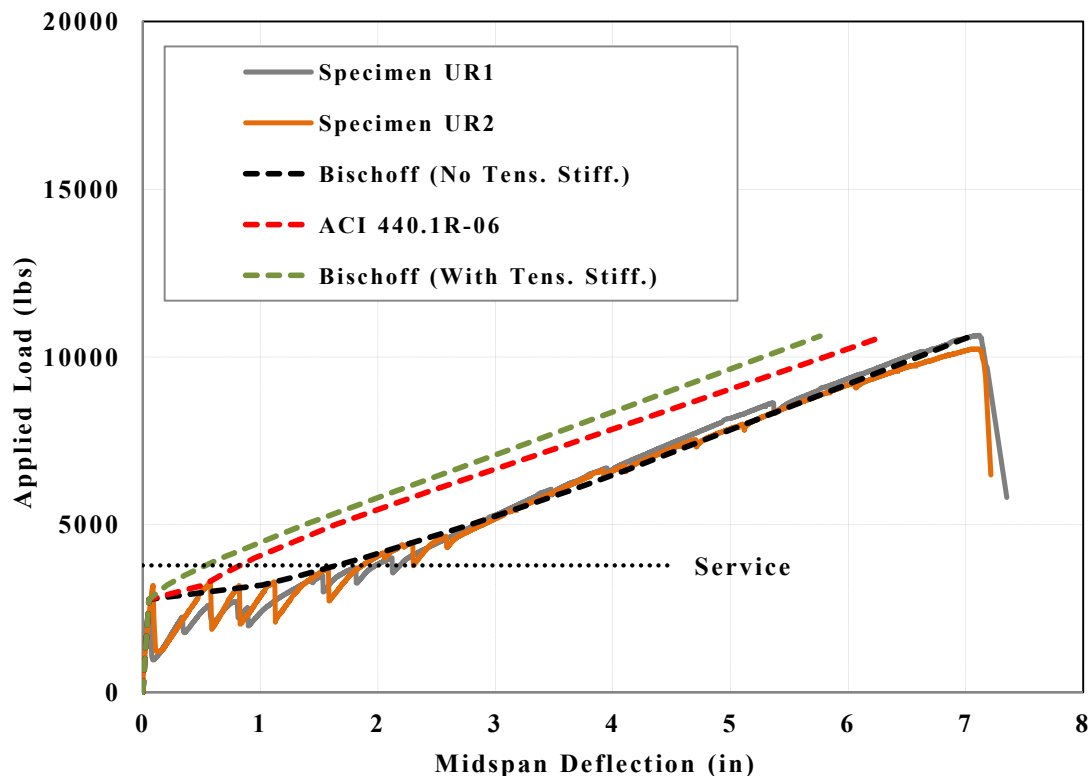


Figure 5.226: Predicted and measured load-deflection behavior (under-reinforced slabs)

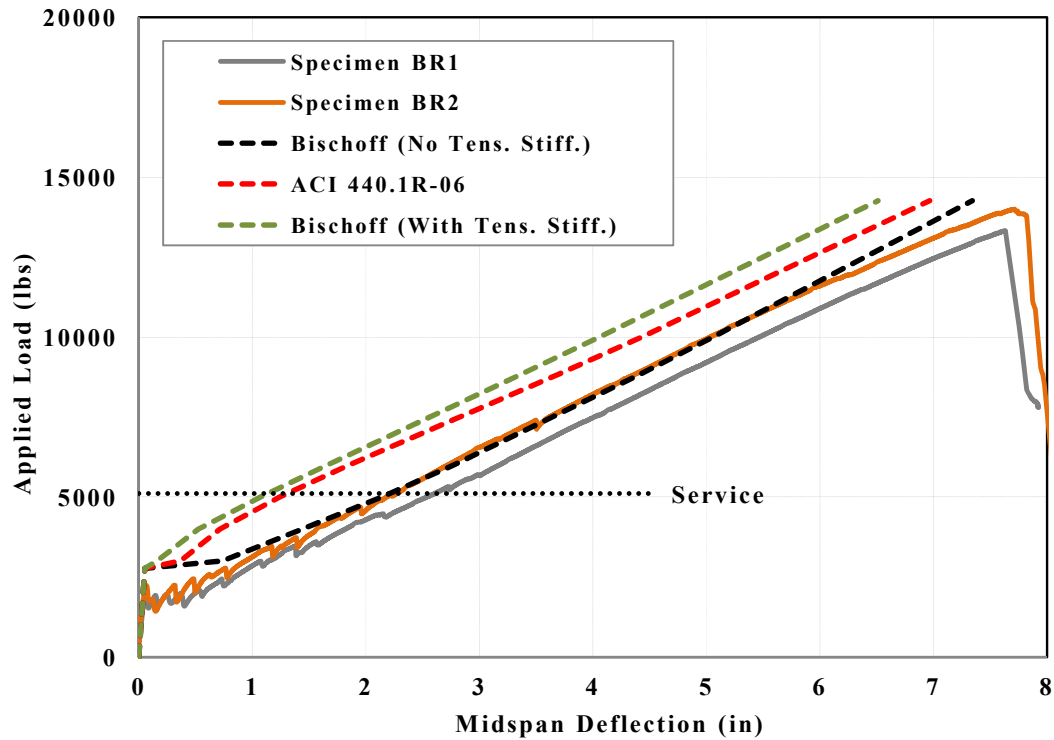


Figure 5.227: Predicted and measured load-deflection behavior (slabs with balanced reinforcing)

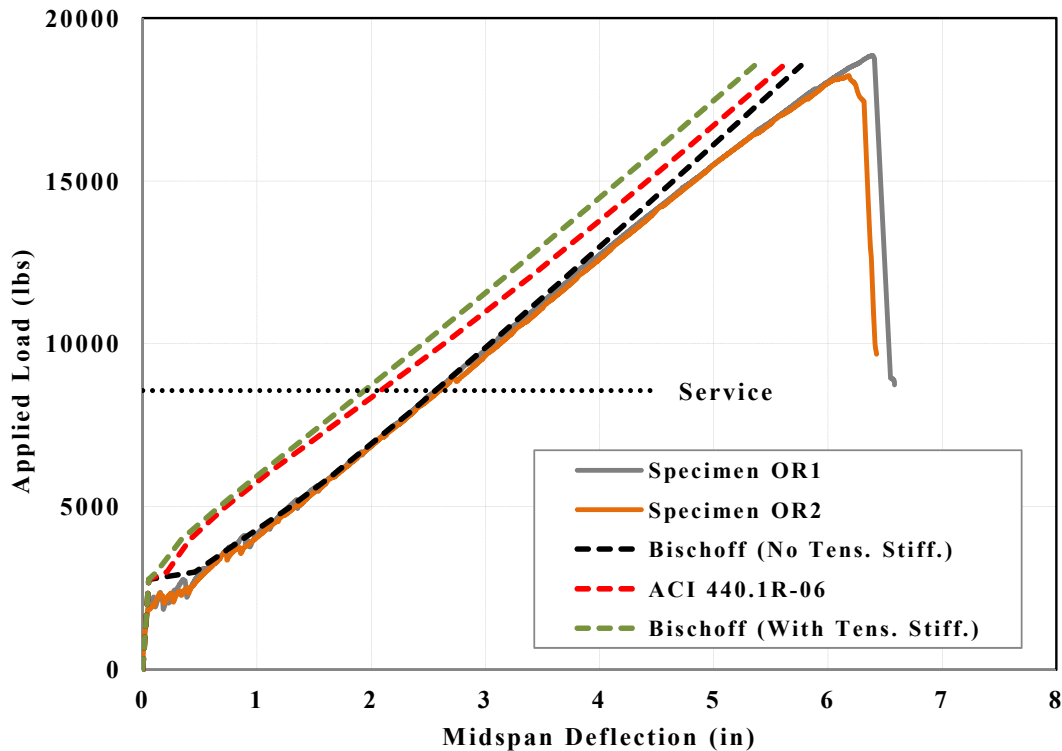


Figure 5.228: Predicted and measured load-deflection behavior (over-reinforced slabs)

The comparison clearly indicates that Bischoff's model without tension stiffening most accurately predicted the full load-deflection response of the slabs as seen in Figure 5.226, Figure 5.227,

and Figure 5.228. The approach suggested by (ACI 440.1R, 2006) and Bischoff's model with tension stiffening gave similar results, both under-predicting deflections after cracking. The flexural rigidity of the prediction models converge to the fully cracked rigidity, $E_c I_{cr}$, near ultimate capacity, as expected.

5.6.11 Load-Strain Behavior of the BFRP Bars

The strain at the depth of the BFRP bars was measured by interpolation between the strains recorded for the top and bottom concrete surfaces at mid-span. Direct measurement of the strain at the level of the bars was very close to the measured values based on the interpolation of the measured concrete strains at the top and bottom surfaces. The BFRP bar strain was predicted according to the detailed moment-curvature analysis and the measured stress-strain behavior of the BFRP bars. Figure 5.229, Figure 5.230, and Figure 5.231 show the measured and predicted load-strain behavior of the slabs.

The measured and predicted BFRP bar strains at ultimate capacity are given in Table 5.40. The measured strain of the BFRP bars was approximately twice $\epsilon_{u,ave}$ for the under-reinforced specimens and the specimens with balanced reinforcement at ultimate capacity. The measured strain of the BFRP bars was equal to $\epsilon_{u,ave}$ for the over-reinforced specimens at ultimate capacity.

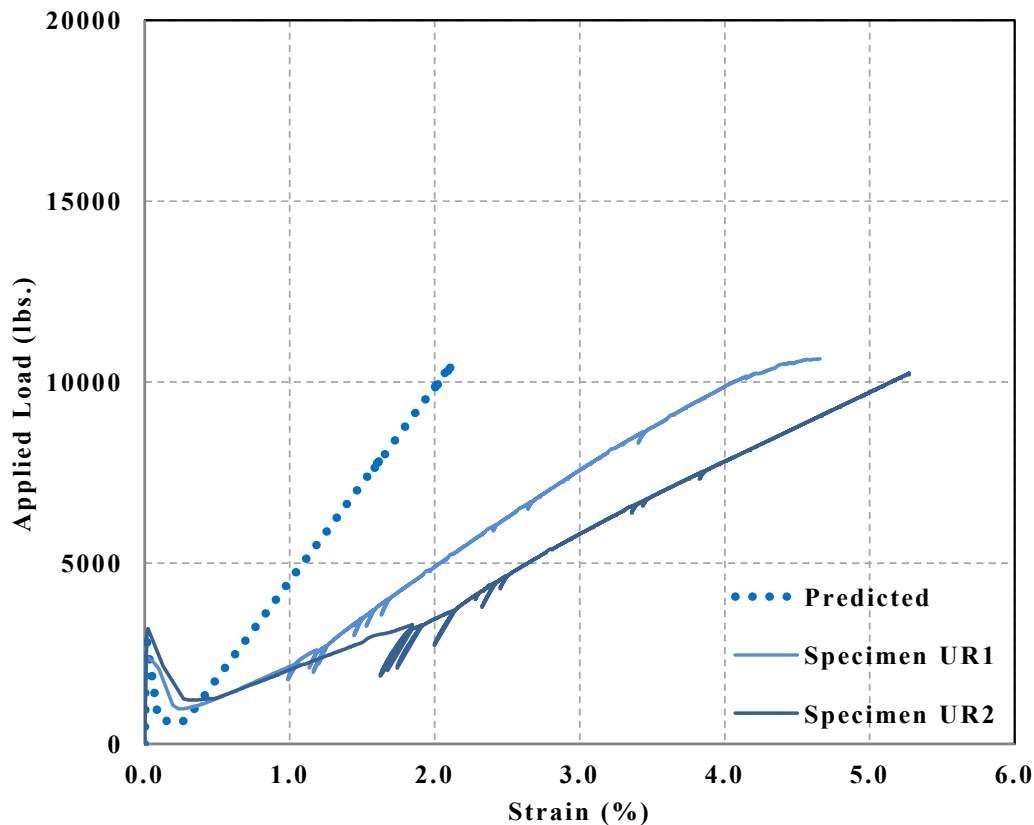


Figure 5.229: Measured and predicted load-strain behavior (under-reinforced)

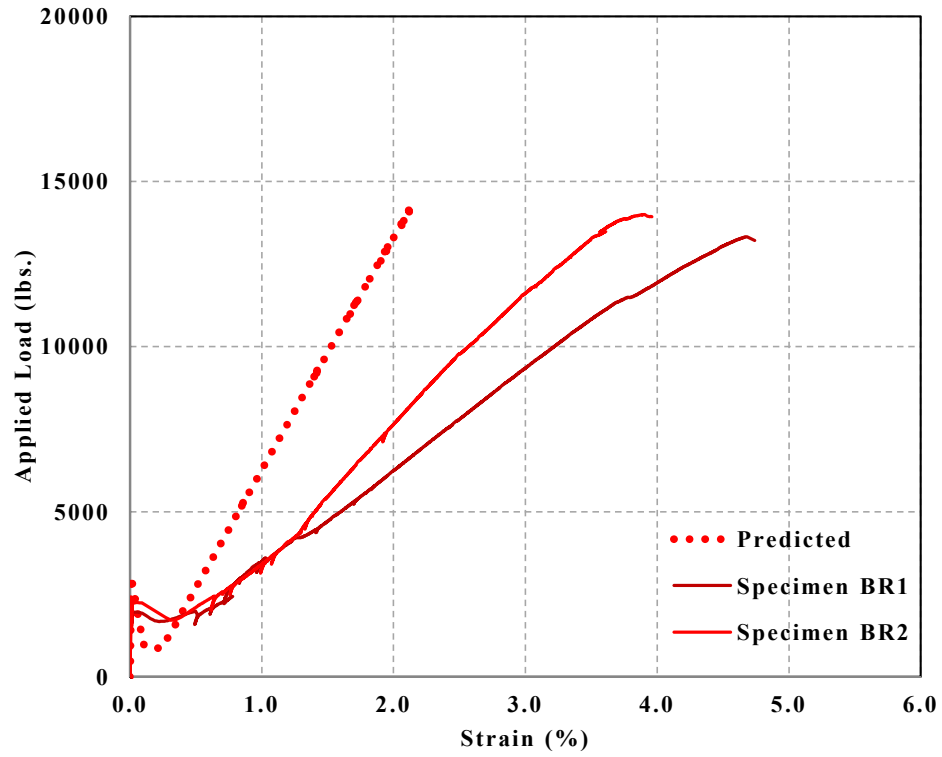


Figure 5.230: Measured and predicted load-strain behavior (balanced reinforcement)

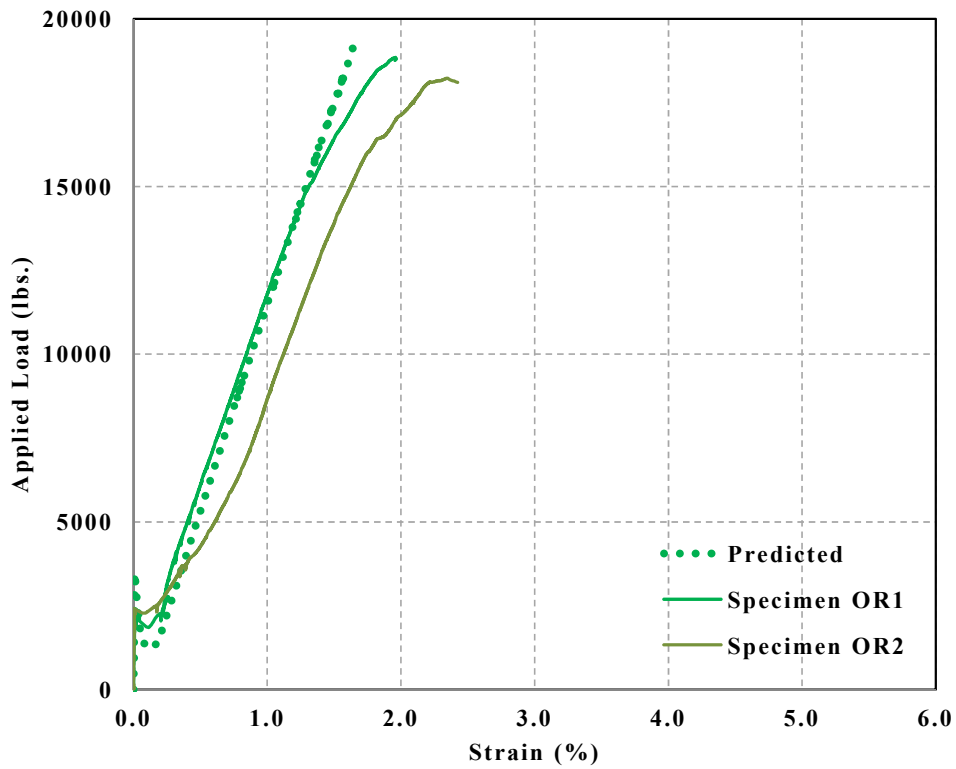


Figure 5.231: Measured and predicted load-strain behavior (over-reinforced)

Table 5.40: Measured and predicted mid-span strains at flexural capacity

Specimen Design Type (ρ_f / ρ_{fb})	Specimen ID	Measured Strain at BFRP Bar Level, (%)		Predicted BFRP Bar Strain (%)	
		Avg.		ACI 440.1R-06	M- Φ Analysis
Under-reinforced (0.74)	UR1	4.9	5.1	2.2	2.2
	UR2	5.3			
Balanced Reinforcement (0.98)	BR1	4.7	4.4	2.2	2.2
	BR2	4.0			
Over-reinforced (1.72)	OR1	2.0	2.2	1.6	1.6
	OR2	2.4			

5.6.12 Slip of the BFRP Bars

Test results suggest that the bars may have experienced localized slip within the maximum moment zone for under-reinforced and balanced-reinforcement specimens. Therefore, the measured strain of the reinforcement is most likely a summation of the strain in the bars and the slip of the bars. Based on the comparison between the predicted and measured load-strain behavior, the amount of bar slip was inversely proportional to the BFRP reinforcement ratio. Figure 5.232 shows the middle bar at mid-span of a tested specimen with the concrete cover removed and a BFRP bar exposed at mid-span. Figure 5.233 shows a bar on the side of tested specimen UR2 after removing the concrete. The bar in Figure 5.232 has clearly experienced slip as indicated by the empty grooves where the spiral wrapping was originally located. Figure 5.233 shows the spiral wrapping debonded from the longitudinal fibers of the BFRP bars. Figure 5.234 shows a tested specimen with the concrete cover removed at the end of a BFRP bar. The figure indicates there is a good bond between the BFRP bar and the concrete at the end of the slab and there is no evidence of slip.

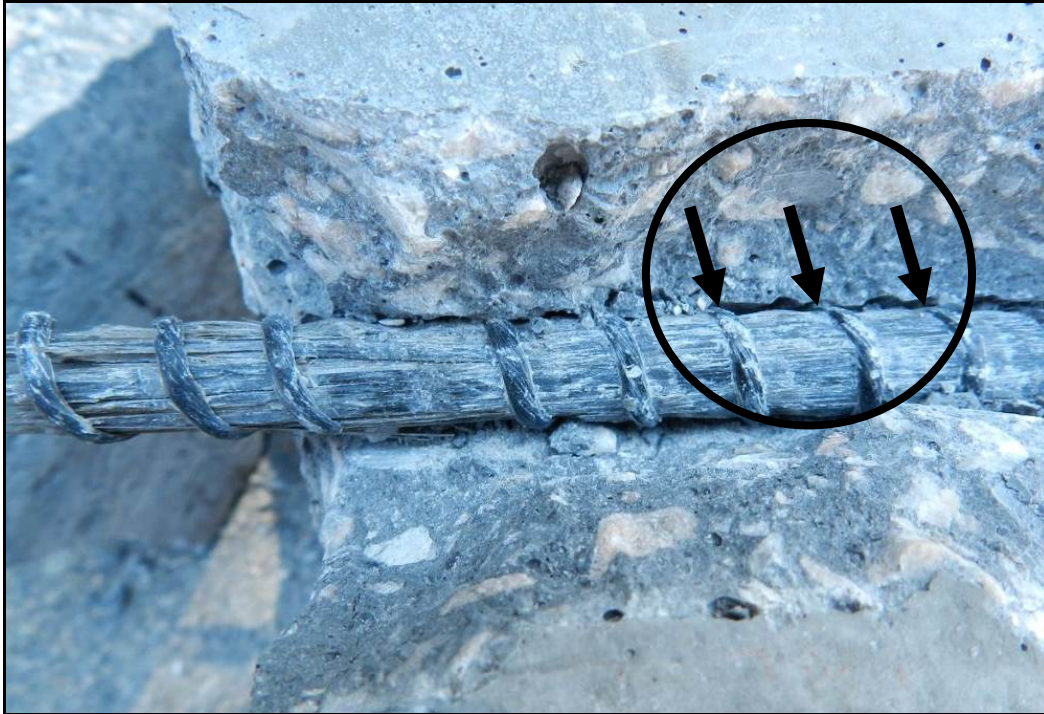


Figure 5.232: Mid-span of a tested specimen with concrete cover removed



Figure 5.233: Exposed BFRP bar with debonded spiral wrapping



Figure 5.234: End of a tested specimen with concrete cover removed

CHAPTER 6

SUMMARY AND CONCLUSIONS

6.1 Conclusions for GFRP Bars

1. The test results of the physical properties of the three grades [I(LM) , II(STD), and III(HM)] tested GFRP bars show that the tested GFRP bars satisfy the ACI and CSA requirements (when applicable) for: (1) Glass fiber content, (2) Transverse coefficient of thermal expansion, (3) Moisture absorption, (4) Cure ratio, and (5) Glass transition temperature.
2. The average tensile strength retention of the tested GFRP bars (Grade I, II, and III) conditioned for 3 months in high pH solution without load at 60°C is over 90% for all the tested GFRP bars.
3. The aging of alkaline solution on GFRP bars is insignificant. Even at high temperature of 60 °C, where the environment is more aggressive, the change in tensile strength is minor.
4. All the tested GFRP bar (Grade I, II, and III) sizes present a value greater than the specified limit for high durability (D1) in CSA S807 (2010) Standard (80%). The elastic modulus of GFRP bars (Grade I, II, and III) is not significantly affected by the immersion in high pH of 13. The value of the elastic modulus retention varied between 89 to 103 %, 95% to 107%, and 92% to 103%, of the reference elastic modulus value, respectively.
5. The tested GFRP bars under conditioning for 3 months with load have shown a tensile capacity retention over 80%. The tested GFRP bars meet the D1 requirement of CSA S807 (2010) for the alkali resistance in high pH solution with load (required limit is 70%).

6.2 Conclusions for Prestressing Carbon Fiber Composite Cables (CFCC)

This study represented an overview on the physical, microstructural analyses, and preliminary durability characterization on CFCC cables. Based on the experimental physical and durability results, the following conclusions were drawn:

1. The test observation indicated that the carbon fiber content is 82% by weight and the water uptake at saturation is equal to 1.1%. The cure ratio of the material is very high (close to 100%) but its glass transition temperature is not clearly visible by DSC.
2. Optical microscopy and electronic scanning microscopy analysis showed that strands presented various levels of porosity. However, no major defect was detected in the tested CFCC material.
3. The results indicate that the strengths of the CFCC specimens were slightly affected by increasing the immersion duration at higher temperature levels. After 7,000 hours of immersion in the alkaline solution at 60°C, test result indicated that almost 7% degradation in the average tensile strength occurred. However, the reduction in the average modulus of elasticity after, 7,000 hours of immersion in the alkaline solution at 60°C, was only 2.6%. The strength reduction could be attributed to an interface degradation that is usually caused by a sizing problem. Further investigations are required on this topic.
4. The average tensile strength retention of the tested Tokyo Rope cable conditioned for up to 7,000 hours in alkaline solution (high pH of 12.8) without load at 22, 40, 50 and 60°C is at least 92.8%. Moreover, the average modulus of elasticity retention of the tested Tokyo prestressing bars conditioned for 7,000 hours in alkaline solution (high pH) without load at 22, 40, 50 and 60°C is at least 97%.
5. The elastic modulus of the CFCC was slightly reduced by about 2.6% after the immersion in alkaline solution for 7,000 hours at 60°C. Based on the regression analysis, the long-term predictions revealed that the effect of alkaline conditioning at 50°C and 60°C could be more significant on the guaranteed tensile capacity retention than that at 22°C and 40°C.
6. For the practical use of CFCC cables in prestressing applications, the cables should be pre-tensioned, assuming that 40 to 50% of the strength capacity remains after prestressing to allow the flexural member to deform and crack before failure (ACI.440.4R, 2004).

Hence, the effect of alkaline solution under sustained load on the tensile properties could be more harmful than without load. Therefore, researchers recommend conducting additional research to investigate the long-term durability effect of alkaline solution with loads on the tensile properties of these cables at different exposure conditions.

7. In conclusion, the Tokyo Rope (CFCC) showed excellent performance maintaining very high guaranteed tensile strength retention and elastic modulus retention after conditioning for 7,000 hours in alkaline solution (high pH of 12.8) at 60°C.

6.3 Conclusions for BFRP Bars

1. This preliminary study reporting results of the physical properties of eight different types (A to H) of BFRP bars (Rock rebars) of different diameters showed that the tested BFRP bars satisfy the CSA and ACI requirements (when applicable) for: (1) Fiber content, (2) Transverse coefficient of thermal expansion, (3) Moisture absorption, (4) Cure ratio, and (5) Glass transition temperature.
2. The SEM analysis has shown that porosity and voids are present in the tested basalt bars. Researchers recommend that improvement in the manufacturing process should be implemented to reduce and/or eliminate these defects.
3. Results of the physical properties of the ReforceTech basalt FRP reinforcing bars No.4 (12 mm-designated diameter) show that this first generation of tested basalt FRP bar does not satisfy the CSA S807 and ACI 440 requirements (when applicable). In particular, the thermal properties (T_g and cure ratio) are very low and not acceptable.
4. The average value of fiber content by weigh is around 73%. An increase in fiber content (above 80%) is recommended to be considered by the manufacturer.
5. The SEM analysis of ReforceTech BFRP has shown that porosity, large voids, and microcracks are present in the tested basalt FRP bars. Researchers recommend that improvement in the manufacturing process should be implemented to reduce and/or eliminate these defects.
6. The test results of investigating the mechanical properties in terms of shear strength of basalt FRP bar (RWS BFRP) showed that the tested BFRP bars satisfy the ACI and CSA requirements ≥ 160 MPa.

7. The average ultimate tensile strength of the two bar types (Type I and J) is nearly identical at roughly 150 ksi.
8. The test results suggest that the bond strength of BFRP bars to concrete is sufficient to achieve the full strength of the bars. The relationship between the maximum measured load and the failure mode of the BFRP bars suggests that the development length is approximately 15 inches for both types of BFRP bars (Type I and J). This value for development length is less than the value predicted by ACI based on the bar diameter and the tensile strength of the bar.
9. Physical properties of BASA Prestressing bar met the requirements of ACI 440 and CSA S807. T_g is a little low and could be increased by increasing the curing temperature or duration to complete the curing and reach the maximum T_g around 123°C.
10. SEM analysis shows that the material is not porous and the interface between the fibers and resin matrix is good, in spite of the presence of certain debonded fibers.
11. From the performed SEM analysis, the comparison of reference and samples immersed in alkaline solution at elevated temperature for 1,000 hours did not show significant signs of degradation. Only the veil covering the bar has been affected by the conditioning.
12. Reductions of transverse and horizontal Interlaminar shear strength due to conditioning in alkaline solution for 1,000 and 3,000 hours were detected.
13. A reduction of flexure strength due to conditioning in alkaline solution was recorded after conditioning for 3,000 hours with exposure to alkaline solution at 60°C.
14. The bond strength obtained for BASA-Prestress bars (7mm diameter) is in agreement with that for steel tendons. However, SEM analysis of BASA-Prestress bars (8mm diameter) shows that conditioned samples present a certain degree of debonding, which is generally higher with samples conditioned in alkaline solution at highest temperatures and long period of time.
15. Further investigations should be conducted on Basalt material. Processing defects in the material should be avoided. The quality of the interface and fiber bonding are very important parameters that should be optimized by using the most performing coupling/sizing agent for the basalt fiber/vinylester system. Resin systems offering a very low shrinkage must be used because curing shrinkage may create free gaps around fibers and reduce the degree of bonding.

16. For concrete beams reinforced with sand-coated basalt fiber-reinforced polymer (BFRP) bars, the developed basalt FRP (BFRP) bars meet the requirements of ACI 440 (2008) and CSA S807 (2010) concerning their physical and mechanical properties.
17. For concrete beams reinforced with BFRP, the measured bond-dependent coefficient (k_b) for the tested BFRP bars was 0.74. This value is very close to that of the CSA S6 (2010) for sand-coated FRP bars ($k_b=0.8$). Consequently, the predicted crack widths using CSA S6 (2010) provisions were very close to the experimentally measured ones.
18. Test results for Flexural behavior of concrete members reinforced with BFRP bars indicate that the ACI 440.1R-06 equations are appropriate to predict the flexural capacity of concrete members reinforced with BFRP bars.
19. The measured deflection at the selected service load was significantly higher than the typical limit specified by ACI. This is due to the low elastic modulus of the BFRP bars and possible slip of the bars. Therefore, the design of members reinforced with BFRP bars should be governed by serviceability (deflection) requirements rather than the load demand.
20. Slip of the BFRP bars results in larger deflections due to a reduction of tension stiffening. The β_d term used in the ACI 440.1R-06 calculation of I_e accounts for the reduced tension stiffening exhibited by members reinforced with FRP bars. Test results suggest that the BFRP reinforced specimens examined in this study demonstrated less tension stiffening than accounted for by the β_d reduction factor. Over-estimating the degree of tension stiffening results in an under-estimation of curvature and deflection for a given load level after cracking. This could explain why the deflection values predicted using I_e , as recommended by ACI 440.1R-06, under-predicted the average measured service load deflections.

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