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Report on Testing Shrinkage, Creep, and Durability Properties of Fiber-Reinforced Concrete

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CHAPTER 1—INTRODUCTION AND SCOPE

1.1—Introduction

Fiber-reinforced concrete (FRC) has become a choice for many designers and builders for the unique properties and advantages that it provides. From slabs-on-ground to elevated slabs-on-pile and structural precast application, the use of FRC has been expanding. This growth of application has created an urgent need to review existing test methods and, where necessary, develop new methods for determining the performance of FRC with reference to its shrinkage, creep, and durability properties.

1.2—Scope

This report documents the current knowledge on testing of FRC for shrinkage, creep behavior, and transport properties that affect the long-term durability. The objective is to review the test methods available in the literature for evaluating these properties and encourage the use of common test methods. This document has summarized various published experiments; the results are presented to illustrate the test methodologies and should not be taken out of context or used for evaluating specific fibers. The results from the tests and procedures used in this document are not intended to be used for the design of FRC structures. The purpose of this document is to gain a better understanding of the many factors influencing test results for the determination of such properties. This report applies to the shrinkage, durability, and transport properties of conventionally mixed and placed FRC, self-consolidating FRC, or fiber-reinforced shotcrete (FRS) using steel, glass, synthetic/polymeric, and cellulose/natural fibers. Some newer test methods and evaluation procedures under development are not included in this report.
CHAPTER 2—NOTATIONS AND DEFINITIONS

2.1—Notations

ARS = FRC average residual strength, psi (MPa)

CMOD = crack mouth opening displacement in a beam test, in. (mm)

Ct = creep coefficient at time t

Ct = ultimate creep coefficient

fl = first-peak strength in a beam test, psi (MPa)

fR,i = FRC flexural residual strength at CMODi, psi (MPa)

QC = correction factor for creep

2.2—Definitions

Please refer to the latest version of ACI Concrete Terminology for a comprehensive list of definitions. Definitions provided herein complement that resource.

diffusion—movement of species (ions, gas, or vapor) from an area of higher concentration to an area of lower concentration independent of the bulk motion of a fluid.

CHAPTER 3—SHRINKAGE OF FRC

3.1—Shrinkage in concrete and FRC

Shrinkage of concrete is the reduction in volume caused by the loss of water or chemical reactions and can take place under various circumstances such as plastic shrinkage, autogenous shrinkage,
carbonation shrinkage, and drying shrinkage (ACI 224R). From a structural point of view, however, there is no need to separate drying shrinkage from other kinds of shrinkage mentioned previously. When concrete is fresh (plastic), the tensile strength of the material is too low, and cracking can occur as a result of small shrinkage-induced stresses. A typical value for the final shrinkage strain of hardened concrete is approximately $600 \times 10^{-6}$ (0.06 percent). The tensile strain capacity of cured concrete is approximately $150 \times 10^{-6}$; therefore, cracking will occur in restrained concrete (unless a low-shrinkage mixture is used). There is some degree of uncertainty in predicting the shrinkage of concrete structures as it depends on many parameters such as concrete composition, source of aggregates, ambient relative humidity, specimen geometry, and the ratio of the exposed surface to the volume of the structural element. If not controlled, drying shrinkage can lead to serviceability problems, such as excessive deflections and durability problems such as more severe freezing-and-thawing deterioration and steel corrosion at cracks. Proper mixture design and construction practices can minimize the amount of cracking in concrete. Unwanted cracks can be controlled using adequate reinforcement such as reinforcing bars or macrofibers, as well as cutting contraction (control) joints. Schematics of free (unrestrained) and restrained shrinkage are shown in Fig. 3.1.
Concrete tends to shrink due to drying whenever its surface is exposed to air with low relative humidity or high winds. Because various kinds of restraint prevent the concrete from contracting freely, cracking should be expected, unless the ambient relative humidity is kept near 100 percent. When properly proportioned, mixed, and placed, fiber-reinforced concrete would be an ideal solution for crack control in concrete against shrinkage. Fibers have minimal effect on the free shrinkage value of concrete under free (unrestrained) conditions. Under restrained condition, however, which is the case for all concrete applications, fibers can provide excellent crack control. Fibers hold (bridge) the cracks and prevent them from growing, resulting in higher number of cracks but with smaller widths (typically hairline cracks). Monofilament synthetic microfibers and cellulose fibers can be beneficial for controlling the plastic shrinkage cracks. Fibrillated synthetic microfibers and glass fibers can be used to control the cracks caused by plastic shrinkage, drying shrinkage, or thermal stresses. Macrofibers (synthetic or steel) can be used for controlling the cracks from shrinkage or temperature changes, as well as crack control under applied service loads.
3.2—Autogenous shrinkage

The internal volume reduction associated with the hydration reactions in a cementitious material is typically 1.66 to 1.94 in.³/lb (6 to 7 mL/100 g) of fully hydrated cement (Jensen and Hansen 1996). Chemical shrinkage is rather a molecular-level volume change and creates the underlying driving force for the occurrence of autogenous shrinkage that is the macroscopic bulk deformation of a closed, isothermal, cementitious material system not subjected to external forces. Autogenous shrinkage has been shown to be equal to chemical shrinkage as long as the paste is liquid. Around the time of setting, however, a solid skeleton is formed that allows empty pores to form while the resulting autogenous shrinkage becomes much smaller than the underlying chemical shrinkage. The part of the autogenous shrinkage that occurs after setting is self-desiccation shrinkage. Autogenous shrinkage due to self-desiccation typically increases with decreasing w/cm. When external curing is not readily available, self-desiccation will occur and a set of empty pores will be created within the microstructure. This will produce capillary stresses that result in a measurable physical shrinkage of the material. Autogenous shrinkage may be thought of as a special class of drying shrinkage where the moisture is removed from the pore structure through internal chemical and physical reactions rather than external drying. (ACI 231R). The typical development of chemical shrinkage and autogenous shrinkage are shown in Fig. 3.2a.
The major difference between the environments used for chemical shrinkage and autogenous shrinkage measurements is that in chemical shrinkage, an external water source provides the additional water absorbed by the cementitious material. In an autogenous shrinkage test, the sample is sealed, and there is no moisture exchange between the sample and the environment. Autogenous shrinkage may be measured in terms of either volume change or length change. Linear measurements are usually carried out by placing the sample in a rigid mold with low friction. The length change of the cement paste is recorded by a displacement transducer at the end of the specimen, as shown in Fig. 3.2b. There are, however, a few limitations in the linear measurements. For example, the geometry of the sample, the friction between the sample and mold, and the measurement might not be carried out before the concrete sample has set. In another approach, volumetric measurements are performed by taking the mass of a submerged sample sealed in a thin rubber membrane (Bjontegaard 1999). The procedure consists of filling a rubber membrane with fresh cement paste or mortar and placing it in a rigid tube. This tube is then immersed in a temperature-controlled water bath and continuously rotated to avoid bleeding.

*Fig. 3.2a—Development of chemical shrinkage and autogenous shrinkage (Sant et al. 2009).*
Autogenous strain of cement paste and mortar is determined following ASTM C1698. The test method was developed based on the experimental work conducted by (Jensen and Hansen 1995) and measures the bulk strain of a sealed cement paste or mortar specimen at constant temperature and not subjected to external forces. Starting at the time of final setting the length of the specimen measured using a dilatometer and recorded until the designated age (28 days). The change in length and original length of the specimen are used to compute the autogenous strain.

Wood derived fibers and powders contain both free and bound water. The free water in large pores and in the lumen and weakly bound water may be released into the surrounding cement matrix over time, providing relief from self-desiccation and subsequent autogenous shrinkage. Mohr et al. (2006) evaluated wood derived materials as internal curing agents for high performance mortar with $w/cm = 0.30$, which were effective in terms of reducing the autogenous deformation. Although wood-derived materials show degradation in a cementitious matrix during wet-dry cycles, the authors believe that these materials can be used at early ages, prior to fiber degradation.

![Diagram of linear measurement of autogenous shrinkage](image)

*Fig. 3.2b—Linear measurement of autogenous shrinkage (Tazawa 1999).*

3.3—Testing FRC for free (unrestrained) shrinkage

Drying shrinkage may produce cracks in concrete if the shrinkage is restrained internally, externally, or both. As drying progresses from the concrete surface inward, pore humidity and the
corresponding free drying shrinkage are generally distributed nonuniformly along the structure’s thickness, with the largest strain at surface and decreasing as the distance from the free surface increases. For the case of drying from both sides, the drying shrinkage stress is obtained through the consideration of internal restraint, and the strain should be uniform over the cross section. Many researchers found that peak tensile stress in the surface layer occurs in the beginning or during the first day of drying, and surface microcracking could be observed in concrete and cement paste. Microcracking in cement paste has been observed even within 1 minute after exposure to drying (Higgins and Bailey 1976).

The structural responses to drying may be different from other types of shrinkage deformations, such as autogenous shrinkage and thermal contraction, because drying often occurs differentially throughout the member thickness. For the case of a concrete slab resting on the ground, drying proceeds from the top surface when humidity at the bottom is fairly constant due to contact with the ground. This humidity gradient will cause a nonlinear distributed shrinkage profile along the slab depth. Two visible deformations develop as a result of the differential shrinkage: 1) the axial movement, which is conventionally treated as drying shrinkage; and 2) warping. If restrained, these two deformations may cause axial stress and bending stress, respectively, as shown in Fig. 3.3a. Typically, only uniaxial drying shrinkage is measured to assess the potential effects of drying shrinkage. The uniaxial drying shrinkage is a time-dependent longitudinal contraction of a prismatic or cylindrical specimen under drying conditions. The magnitude of uniaxial drying shrinkage depends on the size of concrete members because the drying process is limited to a surface region of a few centimeters thickness. While fibers may not affect the magnitude of drying shrinkage, they may slow down the drying process by deviating the moisture pass and increasing the tortuosity, resulting in a lower rate of drying and warping. Fibers, however, can control the
cracks caused from the drying and warping stresses. It has been shown (Tsukamoto and Womer 1991) that concrete reinforced with steel macrofibers reduced the flow rate in a cracked section up to 90% compared to plain concrete.

![Diagram: Non-linear Deformation Gradient](image)

Fig. 3.3a—Deformations and stresses caused by a nonlinear drying-shrinkage gradient (Springenschmid and Plannerer 2001).

Linear specimens have the advantage of a relatively straightforward data interpretation; however, these test methods are generally not used for quality-control procedures, partially due to difficulties associated with providing sufficient end restraint (Weiss and Shah 1997). Linear specimens have been used by many researchers for comparing different mixtures (Springenschmid et al. 1985; Kovler et al. 1993; Toma et al. 1999). The free shrinkage test is performed according to ASTM C157/C157M as shown in Fig. 3.3b. Measurement of length change permits assessment of the potential for volumetric contraction (shrinkage) of concrete due to drying. This test method is particularly useful for comparative evaluation of shrinkage potential in different hydraulic cement mortar or concrete mixtures. Because the length of the specimen is much larger than the cross sectional dimensions, shrinkage can be assumed linear. In this method, the test specimens are cured.
in the molds covered with a plastic sheet for 24 hours for proper initial curing. Upon removal of the specimens from the molds, the initial length reading is taken using a digital comparator dial. After the initial comparator reading, the specimens are stored in the drying room at a relative humidity of 50 percent and a temperature of approximately 73°F (23°C). Because the specimens are not restrained in this test method, this test cannot be an indicator of cracking performance of cement systems against shrinkage and does not differentiate the contribution of different materials such as fibers in controlling shrinkage cracks.

Fig. 3.3b—Typical ASTM C157/C157M free (unrestrained) shrinkage test (Zhuang 2009).

3.4—Testing for restrained shrinkage

Ring tests evaluate the performance of concrete or mortar mixtures when they are restrained from shrinking freely. In the ring test, concrete (or cement paste) is cast around a steel ring. If unrestrained, the concrete would shrink freely; however, the steel ring restrains this movement, resulting in the development of circumferential tensile stresses. Due to its simplicity and economy, the ring test has been developed into both AASHTO PP 34 and ASTM C1581/C1581M. The main difference between these standards is the relative ratio of the concrete to steel ring thickness, which
influences the degree of restraint provided to the concrete. A similar restrained shrinkage test using a steel ring was done in the 1980s (Carlson and Reading 1988). More recently, to better quantify early-age cracking tendency of cementitious material, instrumented rings have been used by researchers to measure the magnitude of tensile stresses that develop inside the material (Grzybowski and Shah 1990; Shah et al. 1997; Hossain and Weiss 2004). Figure 3.4a is a schematic illustration of the restraint stress distribution in typical ring tests for ring specimens that experience drying from the sides only because the top is sealed.

Fig. 3.4a—Ring test geometry according to ASTM C1581/C1581M.

Typical steel ring strain versus time data, starting from the time after casting the test specimen, is shown in Fig. 3.4b. The sudden decrease in compressive strain in the steel ring indicates the point of cracking. Figure 3.4c shows the development of drying stresses with time for various dosages of AR-glass fibers. GRC1.5, GRC3, and GRC4.5 refer to 2.5, 5.0, and 7.7 lb/ft³ (1.5, 3, and 4.5 kg/m³) of glass fiber dosages, respectively. For plain concrete and for FRC at moderate dosages, one main crack is expected to form in the concrete ring. Images should be acquired during with time to capture the crack width development. Image analysis tools can be used to measure the
average crack width for various mixtures. Figure 3.4d shows typical cracks in plain concrete and FRC. Figure 3.4e presents the increase of crack width with drying time for plain concrete (control) as well as GFRC mixtures.

![Typical cracks in plain concrete and FRC](image1)

**Fig. 3.4b—Typical result of strain development for a plain concrete sample (Bakhshi 2011).**

![Effect of glass fibers on the strain development](image2)

**Fig. 3.4c—Effect of glass fibers on the strain development in the ring test (Bakhshi 2011).**

![Crack width over time](image3)

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Fig. 3.4d—Typical shrinkage crack images of control and GFRC samples (Bakhshi 2011).

Fig. 3.4e—Mean and SD of shrinkage crack widths at different times of drying (Bakhshi 2011).

The ring test is a practical tool for assessing time of cracking and cracking sensitivity in
cementitious materials, especially for FRC. Swamy and Stavarides (1979) and Krenchel and Shah
(1987) used the ring test for fiber-reinforced materials. The restrained ring test has also been used
to examine the influence of new materials (for example, SRAs) (Shah et al. 1998; Folliard and
Berke 1997) and mixture proportions on the cracking potential of concrete (Krauss et al. 1996).

The effect of fiber type, aspect ratio, and dosage on the crack width in this has also been
investigated by Voigt et al. (2004). The restrained shrinkage behavior of strain-hardening FRC and
ordinary mortar using ring tests was studied by Wittmann et al. 2005. The results showed that, in
the case of ordinary mortar, a single crack was formed with a maximum crack width of 8.5 mm.
In contrast, approximately 10 cracks were observed in the strain hardening FRC ring specimens
with maximum crack width in the range of 0.05 to 0.11 mm depending on fiber volume fractions.

A new test method was recently developed to monitor the drying of fresh cement paste and the
formation of two-dimensional cracks under accelerated evaporation (Bakhshi and Mobasher
2011). This drying method uses a vacuum system and imposes a one-dimensional moisture flow
through the thickness as shown schematically in Fig. 3.4f. The test method was shown to be very
useful in studying the effectiveness of fibers in controlling the plastic shrinkage cracks. The
development of crack patterns during drying is documented using time-lapse photography. Results
show that 22 and 61 percent reduction in areal fraction of cracks were observed by adding 5 and
10 lb/yd³ (3 and 6 kg/m³) fibers to the cement paste, respectively. Also, maximum crack widths of

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samples associated with 5 and 10 lb/yd³ (3 and 6 kg/m³) fibers were 47 and 71 percent less than corresponding value for the control sample, respectively. These results can be observed in the processed images shown in Fig. 3.4g.

![Diagram](image.png)

**Fig. 3.4f**—Schematic of vacuum drying test setup (Bakhshi and Mobasher 2011).

![Images](image.png)

**Fig. 3.4g**—Crack pattern of cement paste specimens after 24 hours of drying: control, 3 kg/m³ GFRC and 6 kg/m³ GFRC mixtures from left to right (Bakhshi and Mobasher 2011).

**CHAPTER 4**—CREEP PROPERTIES OF FRC

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4.1—Creep behavior of concrete

Creep is defined as time-induced increase of strains in a solid material while the stresses are kept constant (sustained loads). In more practical terms, creep is the increased deformation of a structural element under a constant load over time. Concrete is known to deform considerably under constant loading and normal service conditions. It has been discussed (Powers 1965) that creep and shrinkage in concrete are two different names for the same phenomenon with the only main differences being that shrinkage occurs whether loading occurs or not, but creep is dependent on an external load. Wittmann (1982) argues that shrinkage and creep are based not only on the diffusion of water but on several other mechanisms in the ultra-microscopic scale, namely expansion of single cell particles, expansion of pores, and displacement of gel particles.

Compressive creep strain in conventional concrete can be 1.30 to 4.15 times the initial elastic strain under standard service conditions (Troxell et al. 1958). For certain nonstandard conditions, such as low ambient relative humidity or high ambient temperature, creep strain can be even greater. Over time, these large creep strains result in shortened compression members and increased deflections in bending members. The creep coefficient $C_t$ is defined as the ratio of creep strain to initial elastic strain and is dependent upon the time $t$ after application of stress. The method suggested in ACI 209R is most commonly used for predicting the creep behavior of concrete, using the following expression that is applicable for normal to low density concrete where $C_t$ is the creep coefficient at time $t$ (days) after application of stress; $C_u$ is the ultimate creep coefficient; and $Q_{cr}$ is a correction factor to modify for nonstandard conditions. Figure 4.1 shows a typical creep behavior of concrete presenting its viscoelastic properties.

$$C_t = \frac{t^{0.6}}{10 + t^{0.6}} C_u Q_{cr}$$

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Fig. 4.1—Creep strain versus time graph presenting the viscoelastic properties of concrete (Mouton 2012).

4.2—Creep behavior of fibers and FRC

The combined use of conventional steel reinforcement and fibers has been shown to considerably reduce the deferred deformations under sustained loads, as well as reduce the crack widths. Discontinuous fibers and continuous reinforcing bars both provide tensile capacity after concrete cracks. However, discontinuous fibers differ from continuous bars in tension properties due to the influence of the concrete properties. As a result, it is not possible to separate or disconnect the behavior of the concrete from that of the fibers the way conventional bar-reinforced members are analyzed or designed. FRC should be treated as a composite material with unique properties.

The creep behavior of FRC depends on various parameters of the fibers and concrete. These parameters include the material properties for fibers and the matrix, bond strength, crack width, stress level, and ambient temperature. Under normal service temperatures, steel or synthetic fibers do not show temperature-dependent creep. Steel fibers are considered an elastic material and can resist creep at higher temperatures of up to approximately 700°F (370°C). Synthetic fibers are considered a viscoelastic (viscous and elastic) material and are more susceptible to creep at higher temperatures. Creep of individual fibers is modulus-dependent and therefore higher-modulus fibers show less creep than those that are lower-modulus. The choice of fiber material and material
properties of the chosen fiber material depends on the specific application and the desired or chosen creep behavior for the analysis or design. For FRC elements with lower stress levels or structures with continuous support (such as slabs-on-ground or shotcrete), creep may not be a factor in the design or analysis (Plizzari and Serna 2018).

The study of creep behavior of cracked FRC and the conditions for a stable residual response represent a key point of interest because serviceability of the material will depend on its capacity to transfer the sustained stresses through the fibers and the stability of the cracks. The results of creep tests are especially significant for elements reinforced solely with fibers that tend to present cracks in service state. However, there are no standard test methods and there is limited information on the long-term behavior of cracked FRC elements under load, with only few reports dealing with the subject (Banthia and Pigen 1989; Balaguru and Kurtz 2000; MacKay 2002; Bernard 2004; Mouton 2012; Zerbino et al. 2016; Babafemi 2015; Plizzari and Serna 2018).

4.3—Testing FRC for creep

The majority of creep experiments have been performed on FRC beams (ASTM C1609/C1609M; C1399/C1399M; BS EN 14651) or round panels (ASTM C1550). Typically, these specimens are precracked and tested for creep under different levels of residual strength over time. The increased deflections or crack widths are determined as a part of creep measurement. Zerbino et al. (2016) have recently studied the creep behavior of FRC beams both under three-point and four-point loading configurations shown schematically in Fig. 4.3a. Steel fibers and two types of synthetic macrofibers were used. The modulus of synthetic fibers used in this study is unknown. They used notched beams according to BS EN 14651 with crack mouth opening displacement (CMOD) used as the control signal of a closed-loop servo-hydraulic system. One set
of beams was tested for measuring the first-peak strength ($f_L$) and the residual flexural strengths at 0.5, 1.5, 2.5, and 3.5 mm of CMOD ($f_{R1}, f_{R2}, f_{R3},$ and $f_{R4}$, respectively). A second group of beams was used for creep tests. To induce the cracks in concrete and considering that the residual stress $f_{R1}$ is used for the verification of serviceability limit state, each beam was loaded until a CMOD of 0.5 mm was achieved. To analyze the conditions for stable creep development, different sustained stress levels were applied on sets of three beams, the highest up to 70 percent $f_{R1}$ and the lowest up to 50 percent $f_{R1}$. They recommended the application of sustained loads for periods in the order of 90 days. Their test results showed that FRC with synthetic macrofibers presented higher creep deformations and higher creep rate than steel FRC; variations in creep rate in an order of magnitude were observed. Major differences were found between the two macro synthetic fibers. The changes seen in the residual strength of FRC, after creep tests, were found insignificant.

Fig. 4.3a—Schematics of creep test on FRC beams Zerbino et al. (2016).

In another recent study, Babafemi (2015) investigated the time-dependent crack opening response of FRC using macro synthetic fiber (length of 40 mm and aspect ratio of 50). The modulus of synthetic fibers used in this study is unknown. Compressive strength, uniaxial tensile strength, and uniaxial tensile creep tests were performed at 30 to 70 percent stress levels of the average residual strength. This draft is not final and is subject to revision. Do not circulate or publish: confidential.
tensile strength. Fiber tensile test, single fiber pullout rate test, time-dependent fiber pullout test, and fiber creep test were performed. Under sustained load at different stress levels, significant crack opening has been recorded for a period of 8 months. Creep fracture of specimens occurred at 60 and 70 percent of stress level, indicating that these stress levels are not sustainable for cracked FRC with the specific fiber used. Flexural creep results have shown that the crack opening increases over time. After 8 months of investigation, the total crack opening was 0.2 and 0.5 mm at 30 and 50 percent stress levels, respectively. Figure 4.3b shows the different creep tests for FRC, including fiber pull-out and FRC tensile and flexural tests.

Figure 4.3b—Creep tests performed on fibers (pull-out) and FRC (tension and flexure) Babafemi (2015).

Balaguru and Kurtz (2000) studied the creep behavior of FRC beams using steel and synthetic macrofibers. The modulus of synthetic fibers used in this study is unknown. The magnitude of load applied to a specific specimen during creep testing was based on the results of average residual strength (ARS) tests determined using ASTM C1399/C1399M. Prior to creep testing, the beams were cracked to a deflection of 0.01 in. (0.2 mm). FRC beams were creep tested at loads nominally...
equivalent to 20, 40, and 60 percent of the ARS. The study concluded that, at similar loading levels, cracked synthetic FRC can be expected to experience creep coefficients twice that of FRC with steel fibers. In a similar study, MacKay (2002) investigated the creep response of FRC beams according to ASTM C1399/C1399M. He used hooked-end steel fiber at a dosage rate of 40 lb/yd$^3$ (24 kg/m$^3$) as well as a novel synthetic fiber at a dosage rate of 7.8 lb/yd$^3$ (4.6 kg/m$^3$). After precracking, sustained loads were applied for 90 days at ARS levels equal to 20, 40, 60, and 80 percent (for steel fibers only). The steel and synthetic FRC beams performed similarly in terms of residual strength. Creep deflections, however, varied between 1.5 and 7.2 times the initial elastic deflections after 90 days of loading. These values are referred to as the creep coefficient $C_t$. The main difference observed between the steel and synthetic FRC was the creep occurring at a constant rate, which took place after the initial rapid creep. This constant creep rate tended to be higher for the synthetic FRC. Typical creep responses are shown in Fig. 4.3c for both types of fibers tested.

![Creep response for FRC beams using macro synthetic fibers (left) and hooked-end steel fibers (right) for various ARS levels. (MacKay 2002).](image)

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Bernard (2004) studied the creep of cracked FRC round panels based on ASTM C1550. This test method is typically used for fiber reinforced shotcrete. Three types of fibers were used including a high-modulus macro synthetic fiber, low-modulus crimped macro synthetic fiber, and flat-end steel fiber. The dosage rate for steel fibers was 85 lb/yd\(^3\) (50 kg/m\(^3\)) and for macro synthetic fibers approximately 15 lb/yd\(^3\) (9 kg/m\(^3\)) was used. After precracking the specimens, a gravity-controlled test rig was used and the panels were loaded under flexure for 90 days. Different levels of load ratios were used, defined as the ratio of gravity load during the creep test over the static load resistance. These experiments showed that for small crack widths and load ratios of up to 50 percent, the FRC with both steel fibers and high-modulus macro synthetic fibers exhibited similar creep behaviors. Larger creep and even creep failure was observed for higher load ratios.

More recently, Mobasher et al. (2015) proposed a modified technique in performing flexural creep test on FRC. Flexural creep testing systems were built using commercially available 20 ton hydraulic press frames that accommodate two beams in parallel (Fig. 4.3d), unlike some other tests where beams are stacked up. The load is applied using air pressurized pistons that are attached to the press frame through customized mounting hardware. Compressed air is provided through air pipelines while a backup air compressor is used for emergent cases when the main air pipelines are shut off. Pneumatic air cylinders with different capacities are used to achieve a wide range of applied loads. The input air pressure is controlled by air regulator with a gauge. Practical applied loads up to 3000 lb (13.3 kN) were achieved by adjusting the input pressure up to approximately 30 psi (207 kPa) while higher loads can be applied. The crack mouth opening displacement (CMOD) may be measured using linear variable differential transformer (LVDT) or linear potentiometer that is attached across the notch. Load and CMOD data are acquired using a data acquisition system.

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Figure 4.3d—Schematic drawing of the flexural creep testing system.

Figure 4.3e shows the flexural creep test setup developed at Arizona State University. Figure 4.3e(a) and (b) represent the test assemblies for a high capacity cylinder (10,000 psi) and a low capacity cylinder (125 psi), respectively. Load capacity of the two systems are up to 10,000 lb (44.5 kN) and 2000 lbs (8.9 kN), which accommodate the testing demands from strength normal strength concrete to ultra-high-performance concrete (UHPC) with varying fiber volume fractions. The testing systems are assembled in a temperature and humidity-controlled testing room for the study of environmental effects.
Fig. 4.3e—Testing frames and loaded beam specimens of (a) high and (b) low capacity piston.

The three-point bending fixture is designed to test prismatic 6 x 6 x 21 in. (150 x 150 x 533 mm) beam specimens while modifications may be made to accommodate varying specimen size. The beam specimens are cast, notched, precracked, and then tested under flexural loads sustained for a period of time. The first stage (Fig. 4.3f) denotes the precracking of notched beam specimens under a three-point bending test in accordance with RILEM TC162-TDF, with a 450 mm span between supports. The fracture test is performed when the CMOD reaches 0.02 in. (0.50 mm), and the corresponding applied load is defined as \( f_RI \). The specimen is then completely unloaded. Precracked specimens are reloaded and subjected to sustained loads conditions according to the test setup previously shown in Fig. 4.3e. Applied load levels representing service conditions may be selected, for example, 30 and 50 percent of \( f_RI \). Figure 3.3g and 3.3h demonstrate the load-CMOD response of FRC beams in terms of precracking and creep stages, with synthetic

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macrofiber dosed at 7.5 and 15 lb/yd$^3$ (4.5 and 9 kg/m$^3$), respectively. The creep test of FRC beam
with lower dosage was carried out using the low-capacity system while the higher dosage beam
was tested on the high-capacity system, due to the different service loads, need to be applied.

Fig. 4.3f—Experimental setup used for running the monotonic fracture test (precracking).
Fig. 4.3g—Load-CMOD response of a FRC beam reinforced by 7.5 lb/yd$^3$ of chemically-enhanced synthetic macrofiber including precracking and creep stages (low-capacity system).

Bernard (2004) studied the creep behavior of fiber-reinforced shotcrete round panels (ASTM C1550) after precracking. These standard panels have a diameter of 32 in. (800 mm) and a...
thickness of 3 in. (75 mm) are were shot for casting. He compared flattened-end steel fiber at a dosage of 85 lb/yd$^3$ (50 kg/m$^3$) with two types of synthetic macrofiber at a dosage of 15 lb/yd$^3$ (9 kg/m$^3$) for this study. The specimens were first precracked using a servo-controlled test machine and then tested for creep under gravity load up to 3 months. The panels were then retested according to ASTM C1550 up to 1.6 in. (40 mm) deflection. This procedure is shown schematically in Fig. 4.3i. He observed that for narrow initial crack widths and load ratios up to 50 percent of static load capacity, one type of synthetic macrofiber had a similar creep response as the tested steel fiber in the range of 1.5 to 2.5 as the coefficient of creep. Some specimens showed creep rupture under heavier load ratios with fiber pull-out as the mechanism.

![Load-deflection response during initial cracking test](image_url)

**Fig. 4.3i—Schematics of three stages of creep testing for round panels.**

Attiogbe et al. (2016) evaluated the deformation behavior of pre-cracked, polypropylene fiber-reinforced concrete pipes under sustained loading. Full size pipes with internal diameters of 24 in. (600 mm), 36 in. (900 mm), and 48 in. (1200 mm) were used in the lab study and tested in accordance with ASTM C1818. A dead weight was hung at the free end of a cantilevered rod to
apply the required constant service load over the pipe for a duration of 10,000 hours. The test results show that the synthetic fiber-reinforced concrete pipe exhibited sufficient long-term post-crack strength and low creep deformations. Park et al. (2014) also observed similar performance when tested in actual field conditions (Fig. 4.3j through 4.3l).

Fig. 4.3j—Schematic of long-term creep testing.

Fig. 4.3k—Lab test setup for long-term testing of 24 in. (600 mm) diameter pipe.
Fig. 4.31—Deformation vs. time for 24 in. (600 mm) diameter pipes under sustained loading.

CHAPTER 5—DURABILITY PROPERTIES OF FRC

5.1—Chloride penetration and corrosion

Concrete usually provides protection against the corrosion of the embedded steel because of the highly alkaline environment of the portland cement paste. The adequacy of that protection is dependent upon the amount of concrete cover, the properties of the concrete, the details of the construction, and the degree of exposure to chlorides. Transport properties of concrete can have large impacts on the corrosion of steel and service life of the concrete structure (Shekarchi et al. 2010). Corrosion of steel in concrete is usually an electrochemical process that develops an anode where oxidations takes place and a cathode where reduction takes place. Uncarbonated cement paste has a minimum pH of 12.5, and steel will not corrode at that pH. If the pH is lowered (for example, pH 10 or less), corrosion can occur. Carbonation of the cement paste can lower the pH to levels of 8 to 9. When moisture and a supply of oxygen are present, the water-soluble chloride ions above threshold levels of approximately 0.15 percent (0.4 percent calcium chloride) by mass of cement, can accelerate the corrosion (ACI 222R). Other driving forces include couplings of different metals (galvanic corrosion) and stray electrical currents, such as caused by DC current of...
electric railways, electroplating plants, and cathodic systems used to protect other steel systems such as pipes. In each of the preceding situations, a strong electrolyte such as chloride and moisture are needed to promote the corrosion or at least cause it to occur more rapidly (for example, in years instead of decades). If steel in contact with the concrete is exposed due to cracking, even trace amounts of chloride can trigger and accelerate corrosion when moisture and oxygen are present (ACI 201.2R). Methods are available to estimate time to corrosion initiation and corrosion-induced spalling for uncracked reinforced concrete elements. Limited research has been conducted on the influence of cracking on corrosion of reinforcement in reinforced concrete elements that are expected to cracks in service. Studying the effect of fibers on stress corrosion is significant in predicting the service life of reinforced concrete structures. In these tests, FRC beams are precracked and exposed to some corrosive environment (for example, chlorides and sulfates) and the residual strength of the beam is measured as a function of time and stress corrosion (Bonakdar 2010). Back-calculation methods are available for determine the post-crack tensile properties of FRC from the stress corrosion flexural beam tests (Mobasher et al. 2015).

5.2—Testing FRC for chloride diffusion

While the effect of the fibers on the mechanical performance of the concrete is typically evaluated with tensile or flexural testing, the permeability of concrete is evaluated using, for example, the rapid chloride permeability test (RCPT) or water permeability test. The former, which is standardized as ASTM C1202, uses an electric potential to force the transmission of chloride ions through concrete. While the RCPT may be effective for measuring the permeability of uncracked concrete, it is not the best method for examining cracked materials because it is less sensitive to changes in crack width than water permeability (Aldea 1999). It was shown by Lawler et al. (2002)
that a crack surface acts as a free surface for the chloride to penetrate; therefore, lateral penetration also exists from the crack face (Rodriguez et al. 2003). Polypropylene microfibers were shown to reduce the amount of plastic shrinkage cracking that, in turn, increased the time for steel corrosion formation (Qi 2003). According to Sansone and Brown (2007), cracks occur above and parallel to reinforcement 73 percent of the time when cracking is present. This is a major concern because the purpose of the concrete cover is to protect the reinforcing bars for the service life of the structure, but a crack above a bar would reduce that cover, allowing chlorides to reach that bar in a shorter time.

Forgeron et al. (2004) studied the effect of fiber reinforcement on the plastic shrinkage cracking and ultimately on the chloride penetration of concrete. They implemented an accelerated shrinkage test for concrete panels of 63 x 24 x 2 in. size with fans at a speed of 9 mph (evaporation rate of 0.2 lb/ft^2/hr). They used a novel synthetic macrofibers and measured the crack width and area in different mixtures. Concrete cores of 4 in. in diameter were then obtained from these precracked panels and used for a rapid migration test according to NT Build 492 as shown in Fig. 5.2a. Chloride penetration depths were also measured in this study. It was shown that the higher dosages of these synthetic fibers provided smaller crack widths and shorter crack lengths, resulting in lower penetration of chloride ions. Corrosion resistant synthetic fibers have been shown to provide significant residual strength along with significantly improved cracking resistance (Trottier et al. 2002).
Mangat and Ouruasamy (1987) studied the effect of steel fibers with different geometries on the chloride penetration of concrete beams exposed to marine environment. They cast 4 x 4 x 20 in. (100 x 100 x 500 mm) beams and exposed them to tidal zone of salty sea water. Some beams were sounds (uncracked) and some were precracked using a controlled flexural test with cracks ranging from 0.003 to 0.043 in. (0.07 to 1.08 mm). The chloride penetration was measured using profiling and chemical method for chloride content. Fick’s second law of diffusion was used for plotting the concentration profiles. Chloride diffusion characteristics in uncracked and precracked concrete were determined at more than 1000 marine cycles as shown in Fig. 5.2b. The results show that chloride concentrations increase with increasing crack widths although the influence of small crack widths of less than 0.008 in. (0.2 mm) is insignificant. Smaller crack widths were obtained in FRC mixtures.
Sappakittipakor and Banthia (2012) performed a set of experiments to study the effect of microfibers on the corrosion of steel reinforcing bars in concrete beams. Two fiber types, cellulose and polypropylene, at 0.1 and 0.3 percent volume fraction, were examined. Corrosion activity in reinforcing steel was monitored in loaded RC beams with and without fiber reinforcement for a year in a simulated marine environment. This is shown schematically in Fig. 5.2c. The bars were electrically connected to wires for corrosion measurements. Each beam was coated with epoxy on all sides except for the side at the bottom, which was stressed in tension and exposed to chloride solution. The tests indicated that fibers provide chloride binding. In beams with fibers, corrosion was delayed, and the delay was greater at higher fiber dosages provides the applied load did not exceed a certain threshold value.

Fig. 5.2b—Chloride penetration in concrete beams after 1450 marine cycles (Mangat and Ouruasamy 1987).
A schematic of the chamber simulating a corrosive environment (Sappakittipakor and Banthia 2012).

5.3—Testing FRC for permeability

Permeability, defined as the movement of fluid through a porous medium under an applied pressure head, is the most important property of concrete governing its long-term durability. Permeability of concrete is influenced by two primary factors: 1) interconnected porosity in the cement paste; and 2) microcracks in the concrete (Banthia and Bhargava 2007). Crack development in concrete is known to be profoundly altered by the presence of fiber reinforcement. The use of reinforcing fibers has been shown to produce a significant reduction in water permeability through a modification of crack topography. This has direct implications for improving durability because many deterioration mechanisms of cement-based materials require the ingress of water. The durability of concrete is intimately related to its permeability, that is, the rate at which water is able to penetrate the concrete. This is because concrete is susceptible to degradation through leeching, steel corrosion, sulfate attack, freezing-and-thawing damage, and other mechanisms that necessitate the ingress of water.
Because cracks significantly increase the permeability of dense materials such as concrete, their presence greatly accelerates the deterioration process. An alteration in the crack development mechanism resulting from fiber reinforcement can have a significant effect on durability by modifying the rate at which these deterioration processes can occur (Lawler et al. 2002). Use of the water permeability test has clearly demonstrated that permeability of damaged concrete is governed by the extent of cracking. In addition, it has been observed that beyond a certain crack width, approximately 100 microns, there is a substantial increase in water permeability (Wang et al. 1997). A number of studies have been conducted to investigate the relationship between FRC and water permeability under mechanical stress (Hoseini et al. 2009). There are many properties of FRC that suggest it should feature improved permeability, including crack resistance, increased tortuosity of individual cracks, and a greater likelihood for crack branching and multiple crack nucleation.

In a study, Rapoport et al. (2002) used different dosages of steel fibers, cast concrete cylinders, and precracked them under a closed-loop Brazilian splitting tensile test as shown in Fig. 5.3a. These cracked FRC specimens were then tested for water permeability. The water flow through the system was assumed to be continuous and laminar; therefore, Darcy’s law was applied. The higher steel fiber volume of 1 percent reduced the permeability more than the lower steel volume of 0.5 percent, which is still lower than the permeability of unreinforced concrete. For crack widths smaller than 100 microns, steel reinforcing macrofibers do not seem to affect the permeability of concrete. This research explores the relationship between permeability and crack width in cracked, steel fiber-reinforced concrete shown in Fig. 5.3b.
Banthia and Bhargava (2007) studied the water permeability of plain concrete and FRC with and without an applied compressive stress. For the measurement of permeability under stress, a novel test was used. In this technique, two hollow-core concrete cylinders were simultaneously tested—one with stress and the other without—using identical flow conditions. This is schematically shown in Fig. 5.3c. This special design of the permeability cell eliminates leakage and allows the specimen to achieve conditions of flow equilibrium early in the test. A collated cellulose fiber at volume fractions (F) of 0.1, 0.3, and 0.5 percent was used for this study. Results indicate that in the unstressed state, fiber reinforcement reduces the permeability of concrete as presented in Fig. 5.3d. For the stressed concrete on the other hand, an interesting phenomenon was observed. Initially, as the applied stress was increased, a reduction in the permeability for both plain concrete and FRC was observed. This reduction, however, occurred only to a certain threshold value of stress. Beyond this threshold, a rapid increase in the permeability occurred for plain concrete. For
FRC as well, an increase in the permeability was noticed beyond the threshold value of stress, but the permeability still remained below the unstressed level.

In a different study, Lawler et al. (2002) used steel fibers, polymeric fibers, and hybrid fibers (steel and polymeric combined) to investigate the effect of cracking on water permeability. They tested these specimens under closed-loop direct uniaxial tests and precracked are shown schematically in Fig. 5.3e. For the mixtures examined herein, when fibers were included, an improvement in resistance to permeation was seen for cracked mortars. While an improvement was seen with conventional macrofibers, microfiber reinforcement produced a significant reduction in cracked permeability, and the blending of these fibers with macrofibers further decreased the water flow through the cracked material. The improvement was the result of a modification in the crack topography, especially an increased propensity for multiple cracking. Blending fibers resulted in an additive and beneficial combination of the performance attributes characteristic of the
individual fibers in terms of mechanical performance and resistance to water permeability. The results for flow rate versus displacement are presented in Fig. 5.3f, showing much lower permeability rates for FRC at the same displacement levels when compared to plain mortar.

![Water permeability tensile testing setup](image)

Fig. 5.3e—Water permeability tensile testing setup (Lawler et al. 2002).

![Flow rates for plain mortar and various FRC systems](image)

Fig. 5.3f—Flow rates for plain mortar and various FRC systems (Lawler et al. 2002).

Abbas (2014) studied the effect of steel FRC (SFRC) on chloride ingress in precast tunnel lining segments. Cylindrical core specimens were taken from full-scale tunnel segments (illustrated in Fig. 5.3g) and tested in accordance with ASTM C1585, ASTM C642, AASHTO T 259, and ASTM...
C1202 test procedures. The results showed that the SFRC specimens exhibited lower initial and secondary sorptivity coefficients, reduced volume of permeable voids, lower chloride contents, and slightly lower coulomb values compared to that of the conventionally reinforced precast tunnel segment, respectively (Fig. 5.3g and h and Tables 5.3a and b).

![Coring process from full-scale SFRC PCTL segments.](image)

**Fig. 5.3g—Coring process from full-scale SFRC PCTL segments.**

**Table 5.3a—Sorptivity coefficients for reinforced concrete and SFRC specimens**

<table>
<thead>
<tr>
<th>Sorptivity</th>
<th>Surface</th>
<th>RC</th>
<th>SFRC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial</td>
<td>External</td>
<td>$4.12 \times 10^{-3}$</td>
<td>$3.90 \times 10^{-3}$</td>
</tr>
<tr>
<td></td>
<td>Internal</td>
<td>$4.15 \times 10^{-3}$</td>
<td>$3.92 \times 10^{-3}$</td>
</tr>
<tr>
<td>Secondary</td>
<td>External</td>
<td>$7.05 \times 10^{-4}$</td>
<td>$6.12 \times 10^{-4}$</td>
</tr>
<tr>
<td></td>
<td>Internal</td>
<td>$10.90 \times 10^{-4}$</td>
<td>$8.08 \times 10^{-4}$</td>
</tr>
</tbody>
</table>
Fig. 5.3h—Measured porosity of reinforced concrete and SFRC beams.

Table 5.3b—Diffusion coefficient and surface chloride for reinforced concrete and SFRC specimens

<table>
<thead>
<tr>
<th>Concrete type</th>
<th>Surface</th>
<th>Parameters</th>
<th>3% NaCl</th>
<th>3.5% NaCl</th>
<th>10% NaCl</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>90 days</td>
<td>180 days</td>
<td>90 days</td>
</tr>
<tr>
<td>RC</td>
<td>External</td>
<td>Diffusion ($10^{-12} m^2/s$)</td>
<td>4.06</td>
<td>3.55</td>
<td>4.38</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Surface chloride, %</td>
<td>0.23</td>
<td>0.32</td>
<td>0.24</td>
</tr>
<tr>
<td></td>
<td>Internal</td>
<td>Diffusion ($10^{-12} m^2/s$)</td>
<td>4.89</td>
<td>3.88</td>
<td>5.29</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Surface chloride, %</td>
<td>0.27</td>
<td>0.26</td>
<td>0.27</td>
</tr>
<tr>
<td>SFRC</td>
<td>External</td>
<td>Diffusion ($10^{-12} m^2/s$)</td>
<td>2.39</td>
<td>2.30</td>
<td>2.49</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Surface chloride, %</td>
<td>0.20</td>
<td>0.30</td>
<td>0.21</td>
</tr>
<tr>
<td></td>
<td>Internal</td>
<td>Diffusion ($10^{-12} m^2/s$)</td>
<td>3.22</td>
<td>2.70</td>
<td>3.54</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Surface chloride, %</td>
<td>0.21</td>
<td>0.30</td>
<td>0.21</td>
</tr>
</tbody>
</table>

Hubert et al. (2015) conducted a study to evaluate the influence of fiber content and steel reinforcement ratio on water permeability of reinforced cracked concrete prisms. The reinforced concrete prisms, with a length and cross section of 24.5 in. (610 mm) and 3.6 x 3.6 in.$^2$ (90 x 90 mm$^2$), respectively, were simultaneously subjected to uniaxial tensile loading and water pressure to characterize crack size and water permeability. The experimental results concluded that water permeability coefficients were reduced at serviceability by 31 and 92 percent for steel fiber...
volumes of 0.75 and 1.5 percent, respectively. Furthermore, the permeability of reinforced concrete with and without fibers globally follows a cubic trend with maximum crack width (Fig. 5.3i and 5.3j).

Fig. 5.3i—Permeability device and cell.

Fig. 5.3j—Permeability in function of fiber dosage.
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