STUDY OF SOME PARAMETERS AFFECTING THE MEASURED FLEXURAL TOUGHNESS OF FIBER REINFORCED CONCRETE

by

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ABSTRACT

Fiber reinforced concrete (FRC) exhibits better performance not only under static and quasi-statically applied loads, but also under fatigue, impact, and impulsive loading. This energy-absorption attribute of FRC is usually termed “Toughness”. Experimental characterization of the toughness of FRC remains an actively debated topic. In this thesis, concerns with various available techniques were studied and better ways of characterizing the effects of fibers on the toughness of concrete were sought.

For toughness characterization, beam tests which included standardized ASTM C1609 and C1399 tests were carried out both on lab-cast and site-cast specimens. In the first part of the study, the applicability of the initial loading rate described in ASTM C1609 was evaluated. Tests were conducted on specimens carrying two volume fractions of polypropylene fiber in two separate series with both the prescribed and proposed loading rates. A Comparison between ASTM C1609 and C1399 was carried out later in the study. The Canadian Highway Bridge Design Code (CHDBC) technique prescribed for FRC was also studied.

Based on the results of these tests, it can be concluded that the current loading rate specified in ASTM C1609-2010 is too high for normal strength concrete and it should be reduced to 0.001mm/min initially. It was also found that for calculating Residual Strength Index (R_i), ASTM C1609 procedure is more reliable than the ASTM C1399 as ASTM C1609 is performed in a feedback controlled mode (also called the closed-loop mode) which is very helpful for maintaining stability in specimens.

Since energy absorption is one of the most effective criteria for characterizing FRC, a new method called Flexural Toughness Strength Method (FTSM) was proposed. Tests are carried out on beam specimens according to ASTM C1609 and load- deflection curves are analyzed using the FTSM method. The results demonstrate that the proposed FTSM leads to FRC attributes that are not susceptible to user errors and hence more reliable. The characterization of flexural toughness based on the FTSM approach is independent of the type of deflection measuring technique and no sophisticated instrumentation is required. The Flexural Toughness Factor calculated using this approach has consistently lower coefficient of variation.

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1 INTRODUCTION

1.1 Introduction

It is well known that concrete is a quasi-brittle material with a low strain capacity. Randomly distributed fibers used as reinforcement can reduce concrete brittleness by improving its cracking resistance, toughness, and ductility (Bentur and Mindess 2006). Fiber reinforced concrete (FRC) exhibits better performance not only under static and quasi-statically applied loads, but also under fatigue, impact, and impulsive loading. This energy-absorption attribute of FRC is often termed “toughness.” Experimental characterization of FRC toughness remains an actively debated topic (Banthia and Trottier 1995; Banthia and Dubey 1999; Banthia and Dubey 2000; Gopalaratnam 1995).

There are a number of available techniques for measuring the toughness enhancement due to fiber reinforcement. Most of these techniques adopt the flexural beam specimen as the basis for quantifying toughness, although specimens loaded in other configurations such as compression, shear, tension, and bi-axial bending (plates) are also sometimes adopted. The available test methods for measuring the toughness of FRC include ASTM C1399/C1399M-10, ASTM C1609/C1609M- 10, ASTM C1550, JSCE-G 551, JSCE-G 552, and JSCE-G 553. Until a few years ago, this lists also included ASTM C1018-98. The suitability of these techniques, the concerns with their applicability, and the subjectivity they introduce has been discussed by Banthia and associates (Banthia and Trottier 1995; Banthia and Dubey 1999; Banthia and Dubey 2000). A number of the concerns originate from the fact that post-crack loads and deflections have to be measured in the specimen, which is something not done in most traditional concrete tests and these requirements result in issues arising from spurious specimen deflections, the inability to correctly locate the instant of first crack, and the large instability that occurs when a brittle material cracks. ASTM C1018-98, the first of the beam tests developed for FRC, suffered from a number of these drawbacks and has since been replaced by the ASTM C1609/C1609M-07 beam tests that successfully addressed these concerns (Banthia and Trottier 1995). ASTM C1550 is a round determinate panel test and is generally used only for fiber reinforced shotcrete. The material performance is characterized either in term of areas under the load-deflection curve, or by the load bearing capacity at a certain deflection or at a specific crack mouth opening displacement.

One major issue in FRC toughness measurement is the use of feedback control. Tests can be run in an open-loop arrangement or a closed-loop arrangement. In a closed loop system, there is feedback (via a sensor installed on the specimen) to the machine controls, which can then manipulate/ adjust its input based on a predetermined criterion. In an open-loop system, on the other hand, a feedback loop does not exist and the test cannot be run with a desired specimen response. The most common feedback control signal is in the form of specimen deformation. A closed-loop system can provide a stable deformation rate and produce a stable specimen response, thereby improving precision. Improved stability and precision are of particular interest in testing cementitious materials, as they are brittle and often display instability at the instant of cracking.

1.2 **Outline of the Thesis**

A framework of this thesis is presented here. The thesis comprises five chapters; 1) Introduction, 2) Loading Rate Concerns in ASTM C 1609, 3) Comparison between ASTM C1609 and ASTM C1399, 4) Characterization of Flexural Toughness of FRC, and 5) Concluding Remakes. The following is a summarized content of each chapter.

- Chapter 1 provides the literature review, assumed hypothesis and the background of this study. Literature is first reviewed to establish the effectiveness of fiber reinforced concrete over plain concrete and the importance of characterizing FRC performance. Discussed next are the available standard procedures for characterizing FRC along with their applicability and concerns. Stated next is the main focus of the study, which was how to improve the existing procedures and develop more convenient, reliable, and user friendly techniques.
- In Chapter 2 the applicability of ASTM C1609 for numerous specimens was studied in the contest of loading rate which is prescribed in the standard procedure. A test program was carried out to investigate the influence of loading rate in ASTM C1609-2010. Normal strength fiber reinforced concrete with 0.1% and 0.3% of polypropylene fiber was tested. Results indicate that while the prescribed loading rate is appropriate for the 0.3% fiber volume FRC, it is too high for FRC with 0.1% fiber volume. To obtain a stable load-deflection curve in FRC with 0.1% fiber by volume, a reduced loading rate was proposed.
- An alternative approach is proposed in chapter 3 for calculating Residual Strength Index (R_i) by conducting ASTM C1609 test and test results are compared with the existing method proposed in Canadian Highway Bridge Design Code (CHBDC)..

The results indicate that R_i values calculated using ASTM C1609 are very similar to those obtained in CHBDC.

- Chapter 4 proposes a novel characterization tool called "Flexural Toughness Strength Method (FTSM)". This method proposes applying the JSCE approach to ASTM C1609 curve in an improved manner to minimize the drawbacks such as variability. Results are also compared with existing ASTM C1399 and ASTM C1609 procedures and observed that the proposed method produces result with lower coefficient of variation.
- The last chapter presents the conclusion of the study. Recommendations for future work are also given here.

1.3 Objective and Scope

The main purpose of this thesis is to study various available techniques to characterize Fiber Reinforced concrete (FRC) and examine the existing debate regarding their use. ASTM C1609, ASTM C1399 and CHBDC for characterizing FRC are studied and outcomes are compared to achieve most reliable technique.

Numerous beam tests which included ASTM C1609 and C1399 tests were carried out both on lab-cast and site-cast specimens. The specimens tested were mostly of normal strength concrete with a low volume fraction (0.1% - 0.3%) of polypropylene fiber. In the first part of the study, the applicability of the initial loading rate prescribed in ASTM C1609 was evaluated. Tests were carried on different specimens of two volume fractions in two separate series which includes the prescribed and proposed loading rates.

Comparison between ASTM C1609 and C1399 was carried out later in the study. Tests were carried on beam specimens cast and cured at a real construction site to ensure real casting environment and fiber distribution.

Since energy adsorption is one of the effective techniques to characterize FRC a new method called Flexural Toughness Strength Method (FTSM) is proposed later in the study. Tests are carried on beam specimens according to ASTM C1609 and load-deflection curves are analyzed following the FTSM, advantage of the FTSM over the traditional methods are pointed out and results are compared with the existing techniques such as ASTM C1609 and C1399.

1.4 Fibers Reinforced Concrete versus Conventional Concrete

The term fiber-reinforced concrete (FRC) is defined by ACI 116R, Cement and Concrete Terminology, as concrete containing dispersed randomly oriented fibers. Over 50 years have passed since the initiation of the modern era of research and development on fiber-reinforced concrete.

Since ancient times, fibers have been used to reinforce brittle materials. Straw was used to reinforce sun-baked bricks, and horsehair was used to reinforce masonry mortar and plaster. A pueblo house built around 1540, believed to be the oldest house in the U.S., is constructed of sun-baked adobe reinforced with straw. In more recent times, large scale commercial use of asbestos fibers in a cement paste matrix began with the invention of the Hatschek process in 1898. Asbestos cement construction products are widely used throughout the world today. However, primarily due to health hazards associated with asbestos fibers, alternate fiber types were introduced throughout the 1960s and 1970s (Romualdi 1963; Romualdi 1964).

In modern times, a wide range of engineering materials (including ceramics, plastics, cement, and gypsum products) incorporate fibers to enhance their properties. The enhanced properties include tensile strength, compressive strength, elastic modulus, crack resistance, crack control, durability, fatigue life, resistance to impact and abrasion, shrinkage, expansion, thermal characteristics and fire resistance. Experimental trials and patents involving the use of discontinuous steel reinforcing elements such as nails, wire segments, and metal chips to improve the properties of concrete dates from 1910. During the early 1960s in the United States, the first major investigations were carried out made to evaluate the potential of steel fibers as reinforcement for concrete. Since then, a substantial amount of research, development, experimentation, and industrial applications of steel fiber reinforced concrete has occurred.

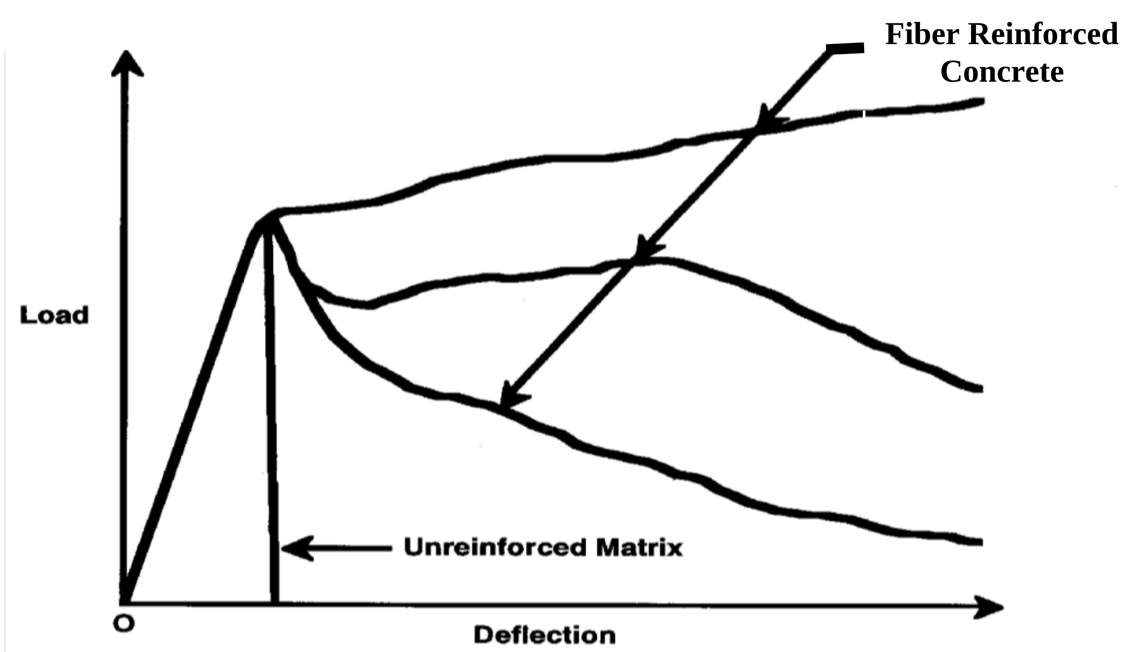

Figure 1-1 Fiber-Reinforced Concrete Versus Unreinforced Concrete.

Unreinforced concrete has a low tensile strength and a low strain capacity at fracture. These shortcomings are traditionally overcome by adding reinforcing bars or pre-stressing steel. Reinforcing steel is continuous and is strategically located in the structure to optimize performance. Fibers are discontinuous and are distributed randomly throughout the concrete matrix. Fibers are being used in structural applications with conventional reinforcement because of the flexibility in methods of fabrication; fiber reinforced concrete can be an economic and useful construction material. Fiber Reinforced concrete is also being used in slabs-on-grade; mining, tunnelling, and excavation support applications. Steel and synthetic fiber reinforced concrete and shotcrete have been used in lieu of welded wire mesh reinforcement in numerous applications.

1.5 **FRC Toughness**

In a load-deflection curve, for most Fiber-Reinforced Concrete (FRC), the pre-peak response is not expected to be much different from plain concrete. Rather it is actually the post-peak response that is of primary interest. The addition of fibers significantly improves many of the engineering properties of mortar and concrete, notably impact strength and toughness. The enhanced performance of fiber-reinforced concrete compared to its unreinforced counterpart comes from its improved capacity to absorb energy during fracture. While a plain unreinforced matrix fails in a brittle manner at the occurrence of cracking stresses (pre-peak response), the fibers in fiber-reinforced concrete continue to carry stresses successfully beyond matrix cracking, which helps maintain structural integrity and cohesiveness in the material (post-peak response). Further, if properly designed, fibers undergo pullout processes, and the mechanical and frictional work needed for pullout leads to a significantly improved energy absorption capability. This energy absorption attribute of SFRC is often termed “Toughness”.

1.6 Characterizing FRC Toughness

Adding sufficient amount of fiber not only improves the post crack characteristic of concrete but may also influence the pre-cracking behavior. Even though the principal role of fiber in concrete is to control the cracking of FRC and then to modify the behavior of the composite after matrix cracking, if it is used at high volumes fraction (i.e. >2% by volume), fiber may also enhance the pre-peak mechanical properties of concrete.

While qualitatively what the fibres do to modify the post-peak behavior of concrete has universal agreement, it has been very tricky to get universal agreement on a definite method to quantify this behavior. There are a number of uncertainties regarding the manner in which this flexural toughness should be measured, interpreted, or used. It is well known (Gopalaratnam et al. 1991; Kapserkiewicz 1990) that flexural toughness depends exactly upon how it is determined.

Factors that influence the measurement of flexural toughness of FRC include the following:

- Test configuration,
- Loading configuration,
- Loading rate,
- Type of loading control,
- Stiffness (type) of test machine,

- Type of deflection-measuring equipment,
- Location of deflection-measuring devices,
- Temperature at testing,
- Specimen size and geometry, and
- Method of manufacturing specimens (molding or sawing).

As FRC is most commonly used in flexural applications, the static mechanical tests most commonly used to characterize FRC are flexural tests. Several such tests have been proposed over the years, and several have been adopted as standards in various jurisdictions. According to Mindess et al. (Mindess et al. 2002), any toughness or residual strength parameter used for the specification or quality control of FRC should, ideally, satisfy the following criteria:

- It should have a physical meaning that is readily understandable.
- It should be largely independent of specimen size and geometry.
- The “end-point” used in the calculation of toughness parameters should represent the most severe serviceability conditions anticipated for any particular application.
- The variability inbuilt in any measurement of concrete properties should be acceptably low.
- It should be able to quantify some important aspect of FRC behavior (strength, toughness, crack resistance) and should reflect some characteristics of the load vs. deflection curve.

Unfortunately, none of the test methods that have so far been standardized is able to meet these criteria; in large part because neither strength nor the shape of the load vs. deflection curves are themselves fundamental concrete properties. However, it is important to understand the difficulties involved in using the methods described below, particularly since the tests often give conflicting results when compared with one other (ACI Committee 544 1988; ASTM 1018; ASTM C1609; ASTM C1399; JSCE- G552).

1.7 Available Testing Procedure for Characterizing FRC Toughness

1.7.1 ACI 544 Procedures

The AC1 544 toughness (ACI Committee 544 1988) index based on Henager's (Henager 1978) proposal was the first energy-based dimensionless index used to characterize the performance of FRC. It constituted the first major effort in recognizing that energy absorption, which is also related to ductility or brittleness, may be important besides strength of concrete, particularly true for FRC and high strength concretes. The AC1 544 toughness index (Fig. 1-2) is defined as the ratio of the area under the load-deflection curve up to a midpoint deflection of 1.9 mm (0.075 in) to the area under the same curve up to first-crack deflection, δ_f. In a later revision, AC1 544(ACI Committee 544 1988) additionally recommends an alternate toughness index, I_t, that is defined as the ratio of the energy absorption capacity of an FRC beam to that of its unreinforced counterpart (Figure 1-2). The definition provides a fundamental measure of the effectiveness of fiber incorporation until complete failure.

1.7.2 **Concern with ACI 544 Procedures**

Among the problems with this approach are that the first crack deflection is difficult to determine reliably, and that the choice of the fixed deflection limit of 1.9 mm is arbitrary. Realistically, deflection limits should be based on serviceability requirements and hence be ‘application-specific’. Afterwards, in the improved ACI 544 proposal, the need to test a companion plain concrete beam and the need to test the FRC beam up to complete fracture made the practical implementation of the index difficult.

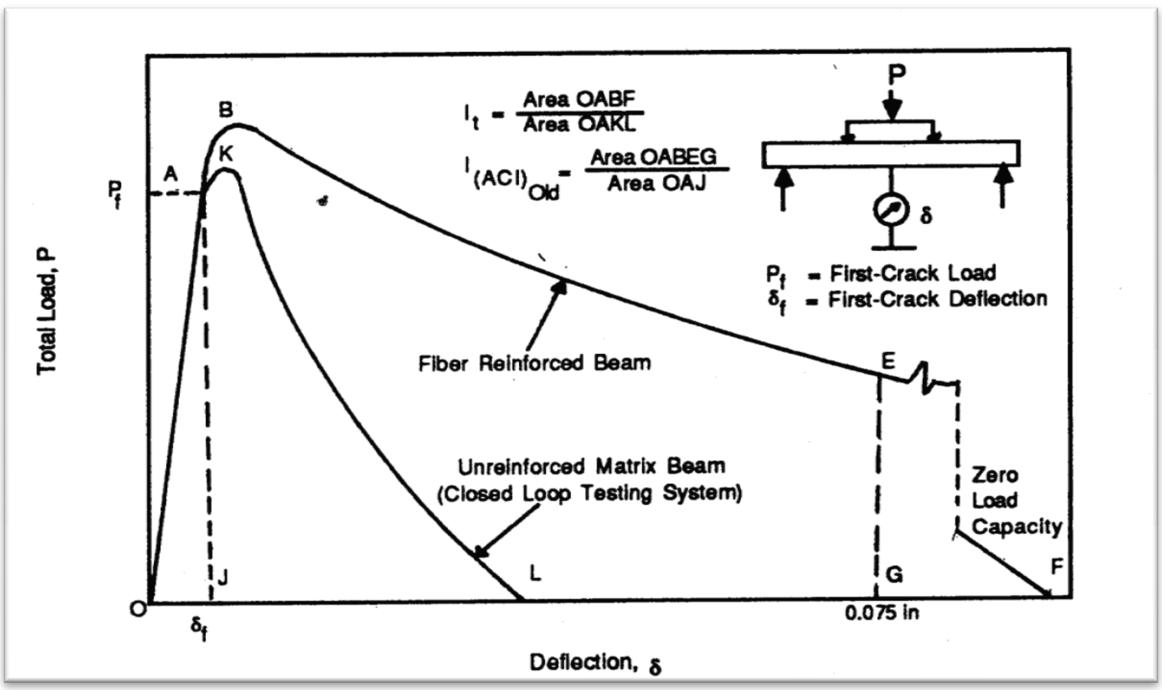

Figure 1-2 Toughness Definition From ACI Committee 544 Guidelines (Gopalaratnam et al. 1991).

1.7.3 **ASTM 1018**

For many years ASTM C1018: Standard Test Method for Flexural Toughness and First-Crack Strength of Fiber-Reinforced Concrete (Using Beam with Third-Point Loading) was the most common test for characterizing FRC toughness, at least in North America. Overall, in this test procedure a beam specimen (100mm x 100mm x 350mm) was tested in flexure under third-point loading, and "toughness indexes" were defined in terms of the ratio of the load under the load vs. deflection curve out to some specified deflection to the area under the curve out to the point of "first crack". In addition, "residual strengths" were usually calculated from the toughness indices; they represented the average post-cracking load that the specimen is expected to carry over a specific deflection interval. The ASTM C 1018 test method in essence was based on determining the amount of energy required first to deflect and crack an FRC beam loaded at its third points and then to selected multiples of the first-crack deflection (Fig. 1-3). Toughness indices I_5, I_{10}, I_{20}, I_{30}, etc., were then estimated by taking the ratios of the energy absorbed to a certain multiple of first-crack deflection and the energy consumed up to the occurrence of first crack expressed as follows:

$$I_N = \frac{\text{Energy absorbed up to a certain multiple of first Crack deflection}}{\text{Energy Absorbed up to first crack}}$$

The subscripts N in these indexes were based on the elasto-plastic correlation such that for a perfectly elasto-plastic material, the index I_N would have a value equal to N. The scheme, thus, compares a given FRC with a conceptual material that behaves in an ideally elasto-plastic manner. Implicitly, the method also used to assume that plain concrete is ideally brittle and, hence, the various toughness indexes in its case are equal to 1. The strength remaining in the material is characterized by the residual strength factors (R) calculated from the toughness indices (Fig. 1-3). Expressed in general terms $R_{M,N,}$ the residual strength factor between Indexes I_M and I_N ($N > M$) is expressed as

$$R_{M,N} = C\{I_N - I_M\} \qquad [1.1]$$

Where constant $C = 100/(N - M)$ were chosen such that for an ideally elasto-plastic material the residual strength factors assume a value equal to the stress at which the elastic-to-plastic transition takes place. Plain concrete, with its ideally brittle response, therefore, has residual strength factors equal to zero.

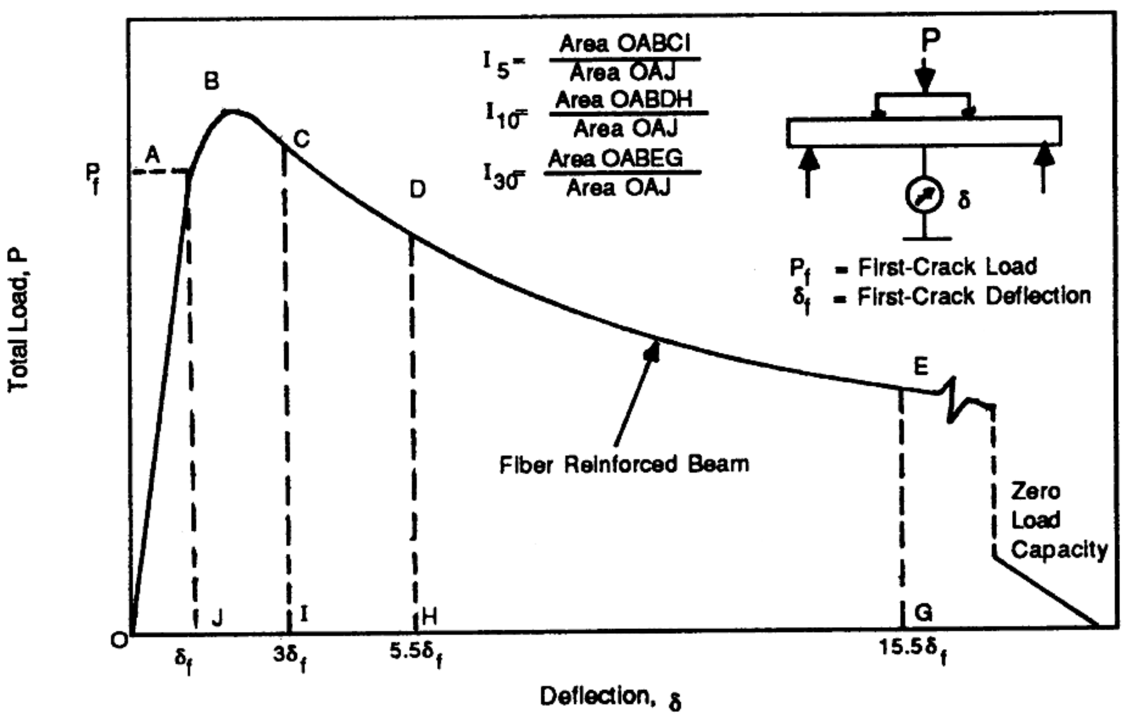

Figure 1-3 Definition of ASTM 1018 Toughness Parameters (Gopalaratnam et al. 1991).

Both toughness indexes and the residual strength factors provided information on the shape of the load-deflection plot and are assumed independent of the specimen size and other testing variables. Notice that an accurate assessment of the energy at first crack is of critical importance, since its use was made in the determination of all performance parameters. Equally important was an exact determination of the beam deflections both before and after the first crack.

1.7.4 **Concerns with ASTM 1018 Test Method**

Even though it is known that toughness characterization is based on energy computations, an exact measurement of specimen deflections and load is very important. From the language of the code, "...exercise care to ensure that the measured deflections are the net values exclusive of any extraneous affects due to seating or twisting of the specimen on its supports or deformation of the support system.. ." Given this precise requirement, a great deal of the data available in the literature based on inaccurate deflection measurements are worthless (Banthia and Trottier 1995). ASTM 1018 test suffered from a number of shortcomings such as:

- Since the deflections at first crack are very small, it is necessary to measure the first part of the load vs. deflection curve accurately, but this is often difficult, due to various "extraneous" deflections that may occur due to machine deformations and seating of the specimen on the supports. As was shown by Chen et al. (Chen et al. 1995), different laboratories correct for these affects differently, and thus may obtain quite different results. Different testing machines may also lead to different results. Chen et al. (Chen et al. 1995) found load vs. deflection curves, particularly in the post-peak region, that were obtained by different laboratories on identical specimens to be very different.

- The toughness parameters are not independent of specimen size.
- The calculated toughness parameters depend on precisely how the point of "first crack" is defined. However, since micro cracking begins almost as soon as the specimen is loaded, it's very difficult to locate the point in an unmistakable manner.

Instability often occurs in the measured load vs. deflection curves immediately after the first major crack, particularly for low fibre volume materials, as shown in Figure 1-4 (Chen et al. 1995). Again, different testing machines can lead to quite different toughness parameters.

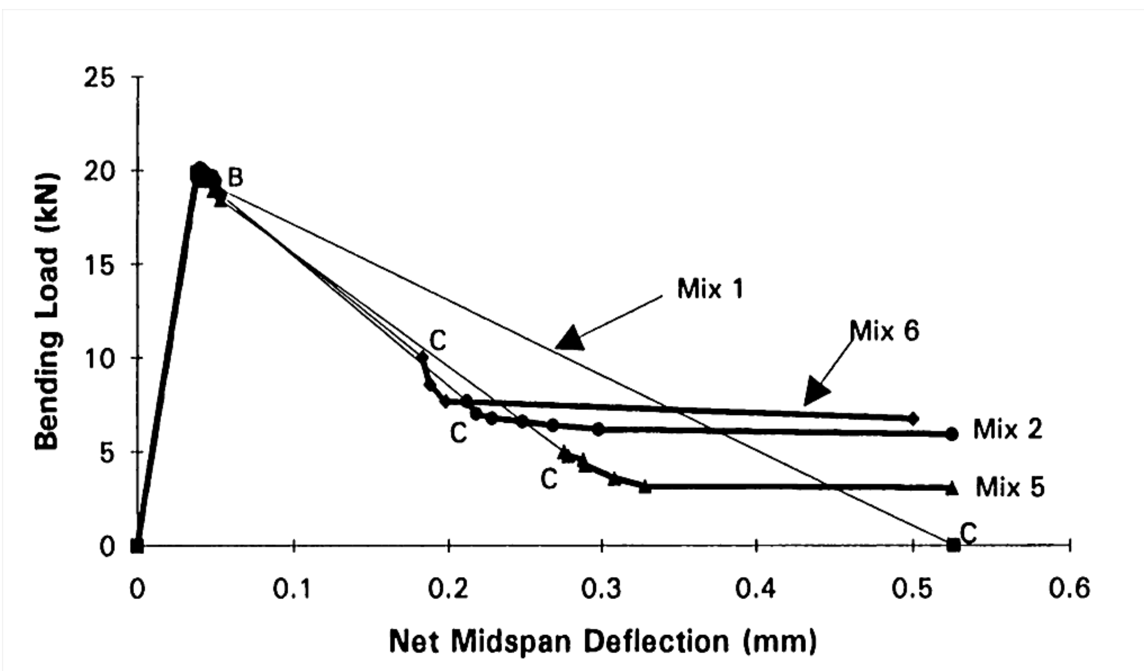

Figure 1-4 Region of Instability for Low Fiber Volume Beams; Mix 1 Represents Plain Concrete (Chen et al. 1995)

- Contrary to Chen et al (Chen et al 1995)'s findings, Gopalaratnam et al reported that (Gopalaratnam et al. 1991) the ASTM C1018 toughness indexes (I_5, I_{10} and I_{30}) are relatively insensitive to fiber type, fiber volume fraction and specimen size. Thus these was a lack of consensus.

As a result of these problems, ASTM C1018 was withdrawn in 2006 and has been replaced by ASTM C1609: Standard Test Method for Flexural Performance of Fibre-Reinforced Concrete (Using Beam with Third-Point Loading) which is described later in this chapter. This test uses the same procedures as ASTM C1018 for obtaining the load vs. deflection curve, but the resulting curve is analyzed in a totally different way. Instead of the Toughness Indexes of ASTM C1018, the residual strengths are determined directly from the load vs. deflection curve. In addition, a toughness parameter is calculated as the area under the load vs. deflection curve out to any specified deflection. This test appears to be more sensitive to different fibre types and volumes than was ASTM C1018.

1.7.5 **JSCE- G 552 (Former JSCE- SF4)**

The Japan Concrete Institute has published a method for determining the compressive toughness of FRC, JSCE–G 552: Test Method for Bending Strength and Bending Toughness of Steel Fiber Reinforced Concrete. In this technique (JSCE-G 552, 1999), the area under the load deflection plot up to a deflection of span/150 (Figure 1-5) is obtained. From this measure of flexural toughness, a flexural toughness factor (FT) is calculated. It may be noted that FT has the unit of stress such that its value indicates, in a way, the post-matrix cracking residual strength of the material when loaded to a deflection of span/150. The chosen deflection of span/150 for its calculation is purely arbitrary and is not based on serviceability considerations. (Zhang and Mindess 2006).

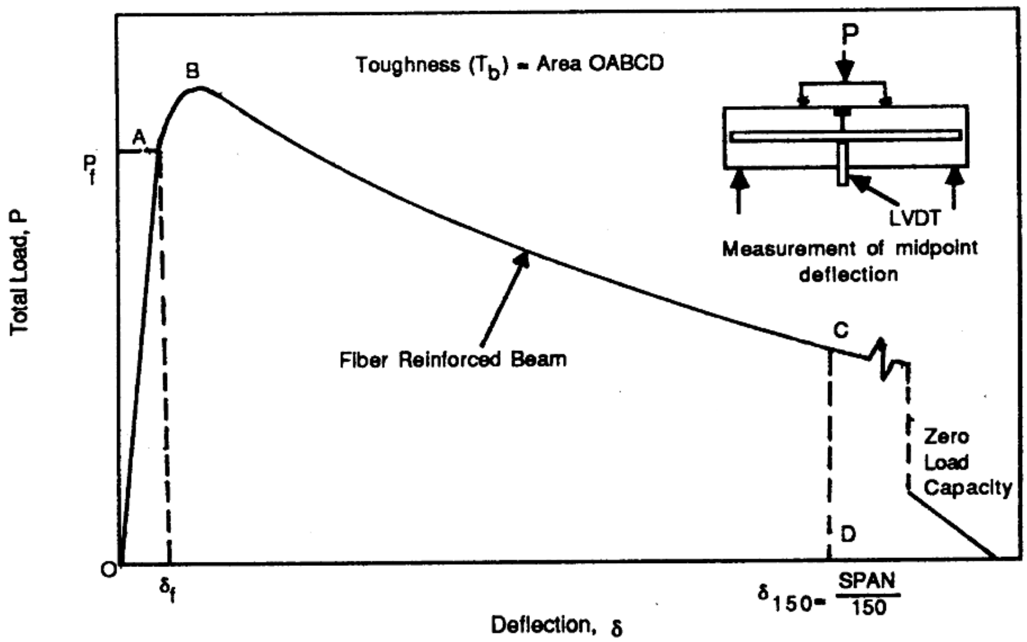

Figure 1-5 Definition of JSCE-G 552 Toughness Parameters (Gopalaratnam et al. 1991).

According to JSCE – G552, the test may be carried out in an open-loop testing machine. This method also relies on the measurement of energy absorbed by the specimen and, as such, an accurate measurement of specimen deflections is of importance. The FT values are found not to be dramatically changed when displacements are measured in different ways unlike ASTM C1018 (Banthia et al. 1992). Identifying the correct location of the first crack, which is crucial and one of the main problems with the ASTM method, is not a concern with the JSCE method. Furthermore, unlike the ASTM method, the instability in the load-deflection plot soon after the first crack is not of major concern in the JSCE method. The endpoint deflection of span/150 is too far out in the curve to be affected by the instability in the initial portion. Gopalaratnam et a1 (Gopalaratnam 1995;

Gopalaratnam et al. 1991) have observed that the energy absorption capacity (defined as energy absorbed per unit cross-sectional area of the beam specimen computed at any deflection limit) has in addition to the sensitivity desired for toughness characterization, the potential for correlation to more fundamental fracture parameters for the material.

1.7.6 **Concern with JSCE-G 552**

The JSCE technique, however, is also not without limitations and concerns.

- First of all the open loop system described in this technique has been found that this works only for compressive strengths below about 60 MPa (Zhang and Mindess 2006); for higher strengths, a catastrophic brittle failure occurs unless a closed loop machine is used.
- The behavior immediately following the first crack, which could be of importance in many applications, is not indicated in the flexural toughness factor in any way.
- The flexural toughness factors are specimen geometry-dependent, which makes an exact correlation with the field performance of FRC rather difficult (Gopalaratnam 1995; Gopalaratnam et al. 1991; Banthia and Trottier 1994).
- Also, the end-point chosen on the curve at a deflection of span/150 (Fig. 1-5) is often criticized for being much greater than the acceptable deflection/serviceability limits.
- Finally, the technique may be criticized for not considering distinguishing between the pre-peak and the post-peak behaviors and adopting a smeared

approach of using the combined area under the curve to calculate the flexural toughness factors.

1.7.7 **ASTM C1609**

Because of the disadvantages mentioned in Section 1.7.4, in the year 2005 the ASTM C 1018 standard was replaced with a new standard, i.e. ASTM C 1609. Thus, any matter contained in ASTM C 1018 with which the researchers often found faults were excluded from ASTM C 1609. According to ASTM C 1609, toughness tests are carried out on concrete beams. Flexural load is applied under constant rate of displacement at one-third of test specimen spans. The evaluation procedure for toughness test results is very similar to the evaluation procedure set down in JSCE- G 552. Specifically, in the evaluation of the test results, first-peak load, peak load, residual load, and the areas below the load–deflection curve are calculated.

In this technique, fiber-reinforced concrete beam specimens having a square cross-section are tested in flexure using a third-point loading arrangement similar to that specified in Test Method C78 but incorporating a closed-loop, servo-controlled testing system and roller supports that are free to rotate on their axes. Figure 1-6 shows the schematic diagram of ASTM C1609 test setup.

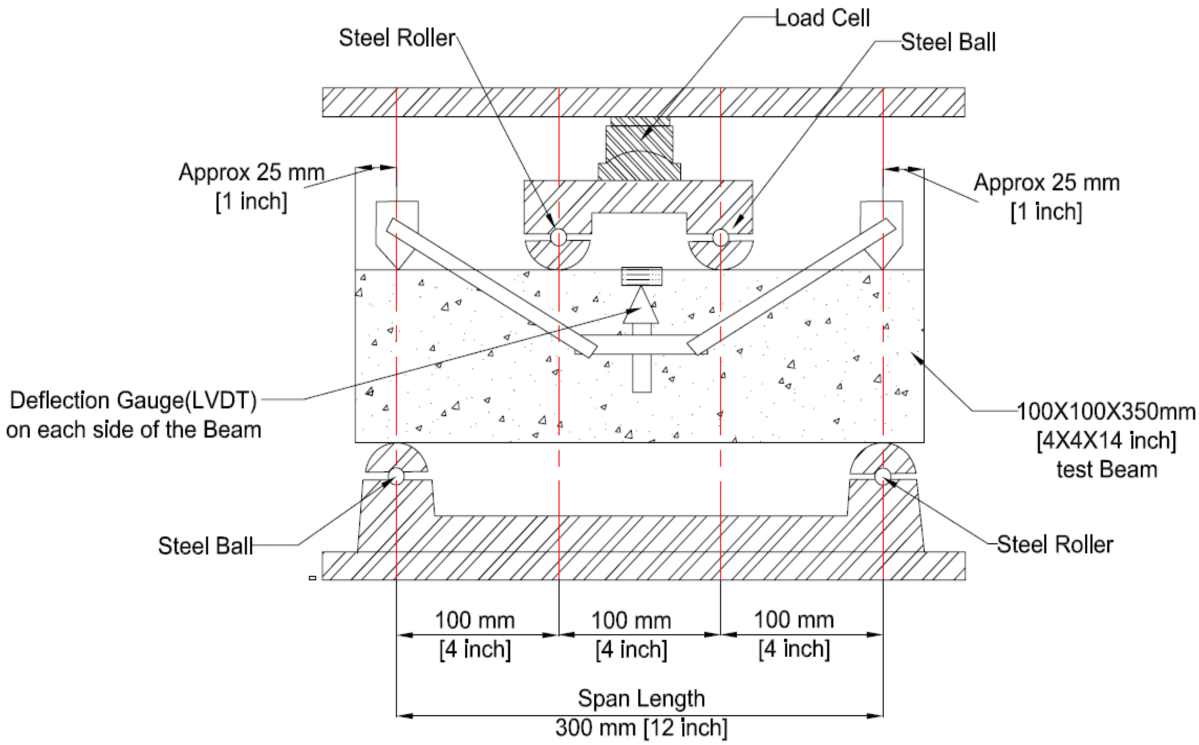

Figure 1-6 Schematic of ASTM C1609

Load and net deflection are monitored and recorded to an end-point deflection of at least 1⁄150 of the span. Data are recorded and plotted by means of an X-Y plotter, or they are recorded digitally and subsequently used to plot a load-deflection curve (Figure 1-7). On the curve first-peak, peak, and residual loads at specified deflections are identified and are used to calculate flexural performance parameters toughness (T^{D}_{150}) and Equivalent

Flexural Strength Ratio ($R^{D}_{T,\ 150}$), where D is nominal depth of the beam specimen in mm.

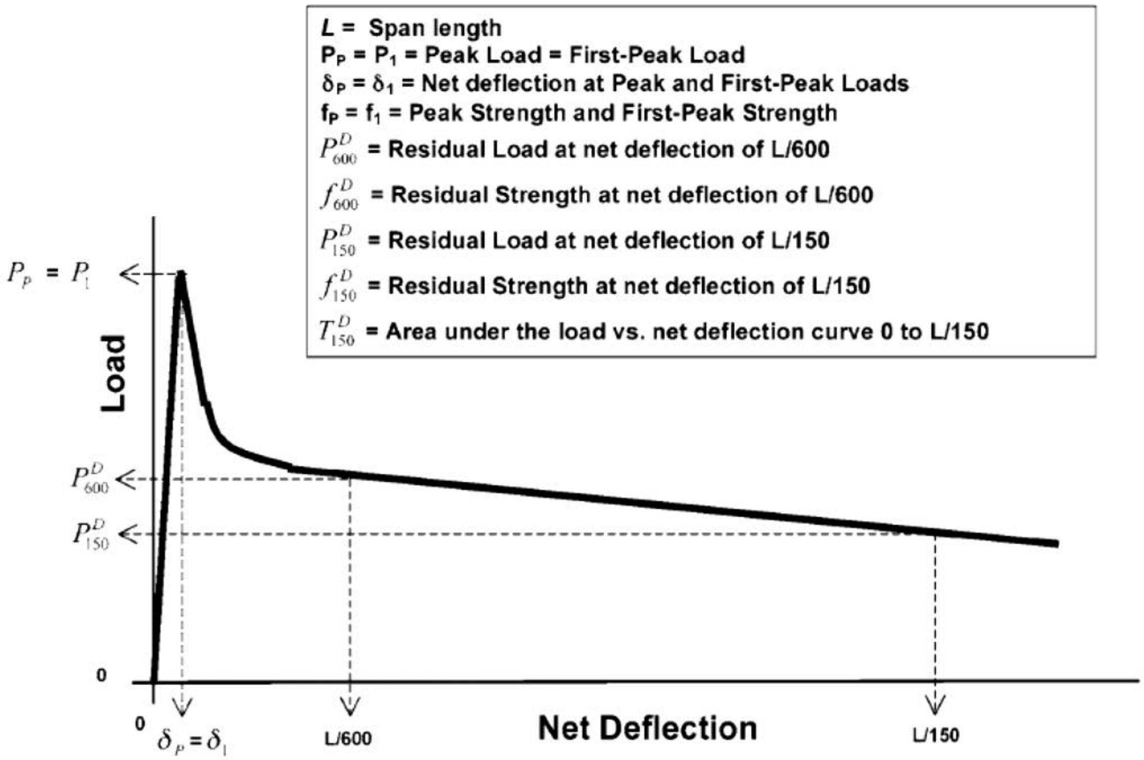

Figure 1-7 Parameters Calculated for ASTM C1609 (ASTM C1609 2011).

1.7.8 **Concern with ASTM C1609**

The ASTM Standard C 1609/C 1609M-10 replaces its predecessor ASTM Standard C1018-97. While the new standard is certainly an improvement over the older one in some respects, there are a number of difficulties that arise when the new standard is applied to ultra high performance fiber reinforced concrete containing very high volume fraction of fiber (2-5%) and exhibiting deflection- hardening behavior (Kim et al. 2008; Skazlić and Bjegović 2008).

- ASTM C1609 Standard recommends estimating toughness as the ‘‘energy equivalent to the area under load–deflection curve up to a net deflection of 1/150 of the span”. For materials that support large deformation in the deflection-hardening range (in excess of L/150), the situation becomes more complicated because the computed toughness may not then truly represent the energy absorption capacity of the material. It is therefore suggested that the computations of toughness be extended to L/100 and even L/50 if the case justifies it (Kim et al. 2008).
- Another difficulty with the ASTM C1609 Standard relates to the definition of Peak load, which is defined as the first point on the load–deflection curve where the slope is zero. Clearly, deflection-softening FRC will exhibit such a response. On the other hand, deflection hardening FRC may not show such a load drop and may not possess a point on their load–deflection curve where a zero slope is meaningful in the sense suggested by the ASTM C1609 Standard.
- First peak point cannot always be found in the initial portion of a load–deflection curve if the specimen shows an elasto-plastic or a stable deflection-hardening response, i.e. without a sudden load drop after peak load.

1.7.9 **ASTM C1399**

This test method covers the determination of residual strength of a fiber–reinforced concrete test beam and provides a quantitative measure useful in the evaluation of the performance of fiber–reinforced concrete. The Average Residual Strength (ARS) is computed using specified beam deflections that are obtained from a beam that has been pre-cracked in a standard manner. The test provides data needed to obtain that portion of the load–deflection curve beyond which a significant amount of cracking damage has occurred and it provides a measure of post–cracking strength, as the strength in question is affected by using fiber–reinforcement. It allows for comparative analysis among beams containing fibers of different types, dimension and shape, and contents. Results can be used to optimize the proportions of fiber–reinforced concrete mixtures, to determine compliance with construction specifications, to evaluate fiber–reinforced concrete which has been in service, and as a tool for research and development. Figure 1-8 shows the schematic of the test setup used for this experiment.

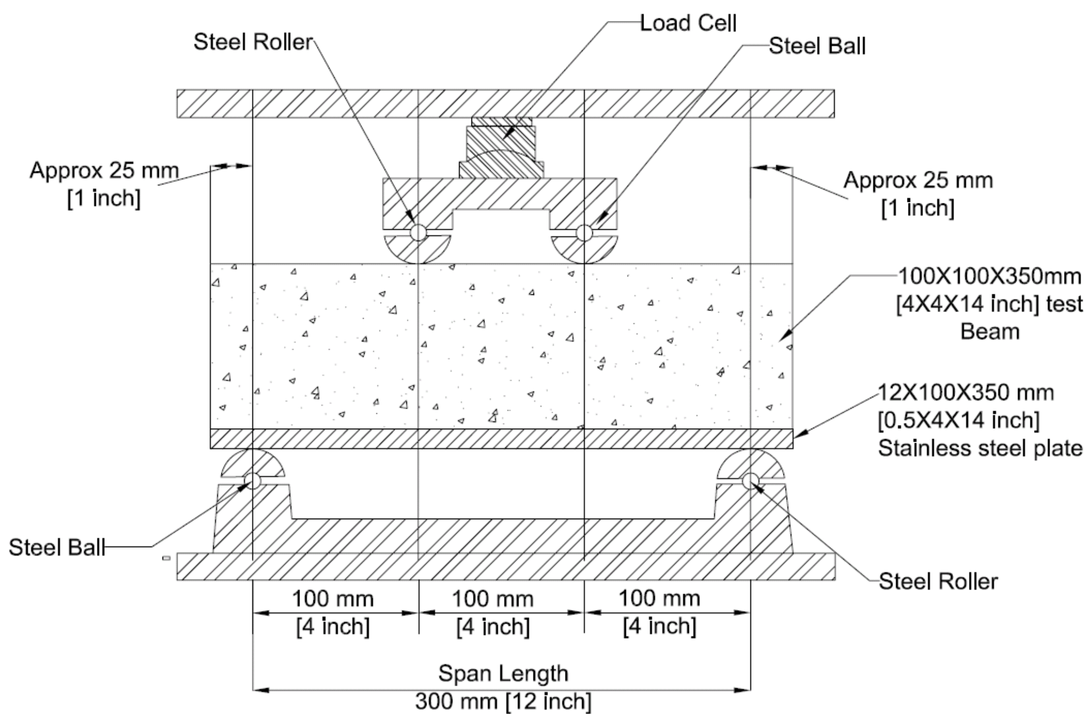

Figure 1-8 Schematic of ASTM C1399 Setup (ASTM C1399 2011).

This technique employs a small beam cracked in a standard manner by loading it in combination with a steel plate; the purpose of the plate is to prevent total failure when the beam starts to crack. The plate is then removed, and the cracked FRC specimen is reloaded in order to obtain a reload vs. deflection curve. The average residual strength of the FRC over the deflection range of 0.5 – 1.25 mm is then determined.

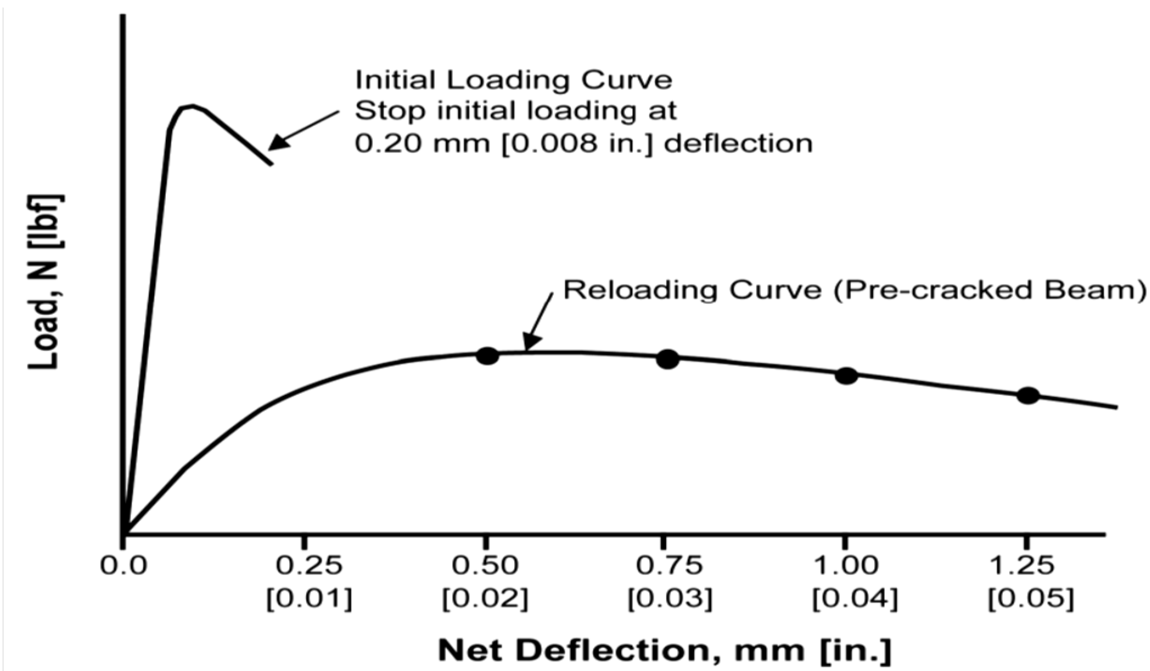

Figure 1-9 Load-Deflection Curve by ASTM C1399 (ASTM C1399 2011).

The Average Residual Strength (ARS) is calculated for each beam to the nearest 0.01 MPa [2 psi] using the loads determined at reloading curve (Figure 1-9) deflections of 0.50, 0.75, 1.00, and 1.25 mm [0.020, 0.030, 0.040, and 0.050 in.] as follows:

$$ARS = (\frac{P_A + P_B + P_C + P_D}{4}) * \frac{L}{bd^2} \qquad [1.2]$$

where:

ARS = Average Residual Strength, MPa [psi],

$P_A + P_B + P_C + P_D =$ Sum of recorded loads at specified deflections, N [lbf],

L = span length, mm [in.],

b = average width of beam, mm [in.], and

d = average depth of beam, mm [in.].

1.7.10 **Concern with ASTM C1399**

It was found by Bathia and co-workers (Banthia and Dubey 1999; Banthia and Dubey 2000) that the load vs. deflection curves obtained in this way were very similar to those obtained using a closed-loop testing machine with proper displacement control. This test thus appears to be most useful for relatively low fibre volumes. However, it too has some serious problems:

- Since the test procedure is divided into two parts the effect of the fibres on the behavior just after first cracking is lost.
- Simple beam theory (as required in this test method) cannot be used to calculate the “strength” of a cracked system, so it is far from clear what the calculated residual strengths actually represent.

- Another concern is that in an uncontrolled open-loop test, during initial loading, the deflection is very hard to control and the net deflection requirements are seldom met. This is of particular concern for very high strength matrices.
- Doubts are often raised as to the ability of the pre-cracking procedure (with steel plate) to effectively replace proper re-loading test setup.
- The length of the pre-crack obtained is not known, and is variable for different FRC systems. This makes comparison between different FRC beams difficult.
- The standard specifically notes that a closed-loop test control is not required. This may be completely valid for regular strength FRC, which only had a minor increase in its ARS values when a closed-loop environment was adopted. In the case of high strength FRC, on the other hand, the influence of load control on the apparent values of ARS is significant is found to be significant. Banthia et al (2011) reported an increase of nearly 40%in the ARS simply by changing the deflection control from open-loop to closed-loop. Thus for ensuring universal applicability for all range of matrix strength ASTM C1399-10 is suggested to be performed in a closed loop environment (Banthia et al. 2011).

1.8 Some Other Proposed Methods for Flexural Toughness Characterization

- ASTM C1550: Standard Test Method for Flexural Toughness of Fiber Reinforced Concrete (Using Centrally Loaded Round Panel) involves the centre point loading of a large circular plate, 800mm in diameter and 75mm thick, supported on three points. The specimen toughness is assessed in terms of the energy absorbed in loading the plate to some selected values of central deflection. This test has become popular for fibre reinforced shotcrete, and is often used in the mining industry. It provides similar results to other toughness test, though with lower variability. Its chief disadvantage is that the specimen itself is too large and heavy (~90 kg) to be handled easily, and does not fit into most common testing machines.

- Barr and Liu (Barr and Liu 1982) proposed a dimensionless toughness index based on the ratio between the area under the load-deflection curve up to $2\delta_f$ and four-times the area under the load-deflection curve up to δ_f .. This type of index provides an upper-bound value of 1 for post-cracking strain-hardening modulus approaching the initial elastic modulus and a lower-bound value of 0.25 for elastic ideally brittle materials. The index proposed by Barr and coworkers (Barr and Liu 1982; Barr et al. 1982; Barr and Hasso 1985) has been developed so as to be applicable for general notched and un-notched specimen geometries (e.g., eccentric compression, compact tension, four-point bending) although no direct correlation between toughness indices measured in the different geometries can be readily made. The ASTM C 1018 indexes and the indexes defined by Barr and Liu (Barr and Liu 1982) (similar to ASTM C1018) rely on the first-crack even more than the AC1 544 (ACI Committee 544 1988) toughness index does since the limiting deflections are multiples of the first-crack deflections.

- Wang and Backer have also proposed the use of an energy-based dimensionless index to characterize toughness (Wang and Backer 1989). The index has been defined as the ratio of the area under the load deflection curve up to a set compliance ($20C_0$ used as an example, where C_o is the initial compliance) to the first-crack. This definition is similar to indirectly limiting deflection but at varying values for materials exhibiting different post-cracking behavior. The limiting deflection is lower for materials with higher post-crack stiffness (relative to the pre-crack stiffness). Using several different types of post-cracking responses, they have demonstrated that their index provides a better representation of the energy absorption capacity than the ASTM Cl018 toughness indexes. The level of sensitivity reported for this type of index and the potential for practical implementation of compliance-based limits make this approach an attractive option deserving further investigation. It should, however, be noted that since compliance is a specimen size specific parameter, toughness for different FRC composites based on compliance limits should be compared only while using identical specimen sizes.

- The Spanish standard (AENOR 1989) requires computation of a dimensionless index equivalent to I_{30} of the ASTM C 1018 (computed at a deflection limit of 15.5 δ_f) perhaps recognizing that at the smaller limiting deflections such an index is not a sensitive toughness measure. It also requires reporting of the first-crack strength and the energy absorption capacity, as in the Japanese standard (JSCE-G552, 1999).

- Trottier and Banthia (Trottier and Banthia 1994) recommend using the equivalent flexural strength concept of the JSCE method with some modifications that are reported to characterize the post-peak toughening of FRC better. Use of different deflection limits makes the approach more general than the JSCE method in that it can also be applied at small deflection limits. The more significant different approach, however, involves the use of only the post-peak energy absorption in their strength computations. Equivalent mean post-cracking strength (PCS_m) is computed in their proposal as

$$PCS_m = \frac{(E_{post,m})L}{(\frac{L}{M} - \delta_{peak})bd^2} \qquad [1.3]$$

where $\mathbf{E}_{post,m}$ is the post-peak energy at a deflection of L/M, δ_{peak} is the deflection of the peak, *b* is the specimen thickness and *d* is the specimen depth. This method doesn't consider the pre-peak contribution to energy absorption and it might also be difficult to find the first crack accurately.

- Another test that is gaining popularity in the shotcrete industry is the South African Water Bed test (Trottier et al 2002). A large plate specimen 91600mm x 1600mm x 75mm is fastened in place over a water bladder, which is then filled with water to apply pressure over the entire specimen. The energy absorbed (i.e. the toughness) is the area under the load vs. deflection curve out to a series of given deflections ranging from 25mm to 150mm.

- EFNARC (European Federation of Producers and Contractors of Special Products for Structures) has proposed a plate test (EFNARC 1997) involving a 600mm square plate, 100mm thick, supported on all four sides and loaded at the centre. The toughness is determined from the load vs. deflection curve out to a deflection of 25mm. This test is sometimes used in Europe, but rarely in North America.

- RILEM TC162-TDF (RILEM TC 162-TDF 2002), as described in detail by Vandewalle (Vandewalle 2000; Vandewalle et al. 2003). In this procedure, a notched beam (150mm x 150mm x 550mm) is tested in centre-point loading, and the crack mouth opening across the mouth of the notch is measured, using a closed-loop testing machine. The energy absorption capacities out to particular deflections are determined as a function of the area under the load vs. deflection curve. This method is intended to provide values that can be used directly in the structural design of beams.

- Gopalaratnam et al. (Gopalaratnam et al. 1991) proposed the use of a notched beam tested under servo-controlled conditions to characterize toughness of FRC. The closed-loop test is controlled by the crack mouth opening displacement (CMOD) as in test procedures for determining fracture parameters of concrete Toughness was characterized in terms of net-deflections adopting the ASTM C 1018 procedure. It was observed that the toughness indices thus obtained were only as sensitive as the indices from un-notched beams but exhibited much less scatter.

As should be apparent from the description above, all of the proposed methods are empirical in nature, and are thus not directly comparable. They all violate one or more of the criteria outlined by Mindess et al (Mindess et al. 2000) above and thus are of limited usefulness in providing design values for FRC. Indeed, it is this lack of a commonly agreed upon method for characterizing the performance of FRC that has inhibited the truly structural uses of this composite material.

1.9 Equivalent Residual Strength (R_i) Calculation

According to Canadian Highway Bridge Design Code (CHBDC) FRC Residual Strength Index (R_i) is calculated using following equation:

$$R_i = \frac{ARS}{R} \qquad [1.4]$$

Where ARS is the mean value of the Average Residual Strength determined by using ASTM C 1399 and R is the mean value of modulus of rupture determined by performing ASTM C78.

The approach is based on the premise that the post-cracking load carrying capacity of concrete with fibers (ARS) requires to be normalized with respect to its modulus of rupture (R) in order to produce a non-dimensional parameter (R_i) which can then be specified based on a specific bridge application. The parameter therefore purports to characterize the ‘toughness’ of FRC over and beyond the stress carried by the concrete matrix at the moment of ‘first’ crack. It also unconditionally recognizes that very high strength concretes (higher R values) are essentially brittle and to provide a needed crack control a greater fiber dosage (higher ARS) may be required.

ASTM C78 test method is used to determine the flexural strength of specimens prepared and cured in accordance with Test Methods C42/C42M or Practices C31/C31M or C192/C192M. Results are reported as the modulus of rupture and calculated by using following equation:

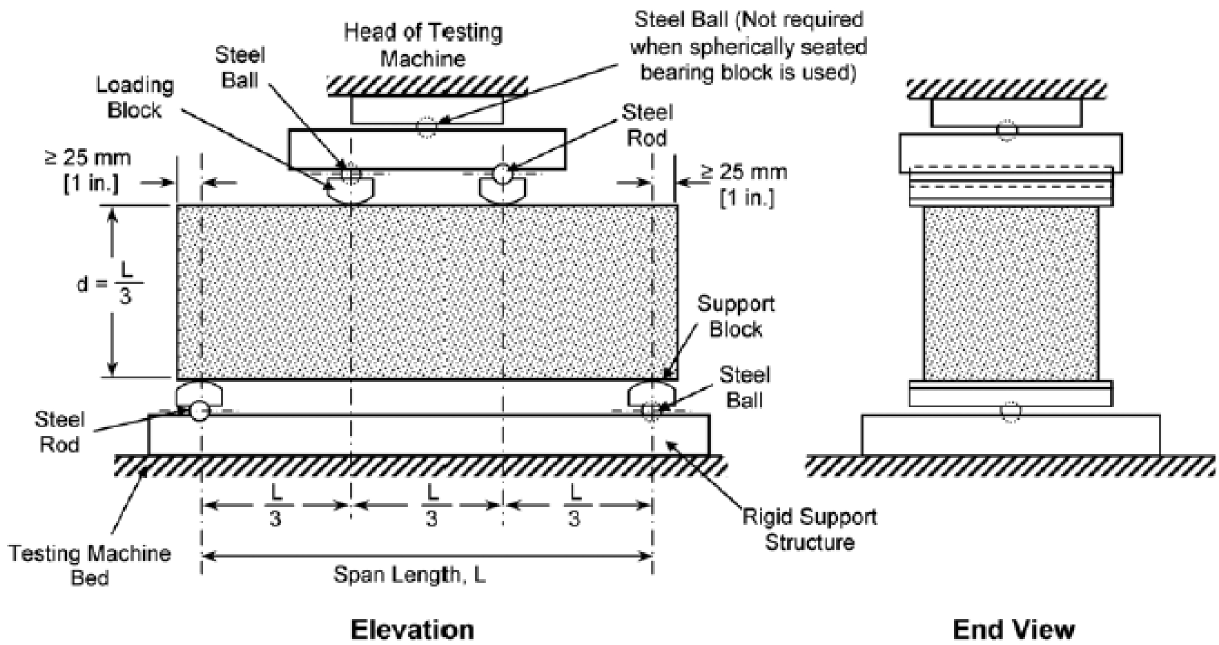

Figure 1-10 Schematic of ASTM C78 Setup (ASTM C78 2011).

(a) If the fracture initiates in the tension surface within the middle third of the span length, calculate the modulus of rupture as follows:

$$R = \frac{PL}{bd^2} \qquad [1.5]$$

Where:

R = modulus of rupture, MPa [psi],

P = maximum applied load indicated by the testing machine,

N [lbf],

L = span length, mm [in.],

b = average width of specimen, mm [in.], at the fracture, and

d = average depth of specimen, mm [in.], at the fracture.

Note that the weight of the beam is not included in the above calculation.

(b) On the other hand, if the fracture occurs in the tension surface outside of the middle third of the span length by not more than 5 % of the span length, the modulus of rupture is calculated as follows:

$$R = \frac{3Pa}{bd^2} \qquad [1.6]$$

where:

a = average distance between line of fracture and the nearest support measured on the tension surface of the beam, mm [in.].

Naturally, the strength determined will vary with specimen size, preparation, moisture condition, curing, or if the beam has been molded or sawed. The results of this test method may be used to determine compliance with specifications or as a basis for proportioning, mixing and placement operations. It is used in testing concrete for the construction of slabs and pavements.

1.10 **Open Loop versus Closed Loop Testing Procedure**

One of the major concerns in FRC toughness measurement is the application of feedback control. Tests can be run in an open-loop arrangement or a closed-loop arrangement. In a closed loop system, there is feedback (via a sensor installed on the specimen) to the machine controls, which can then manipulate/ adjust its inputs based on a predetermined criterion. In an open loop system, on the other hand, a feedback loop does not exist and the test cannot be run with a desired specimen response.

A system can be defined as an assembly of interacting elements, any of which can affect the response of the other elements. The inputs to the system are signals that are transferred from the environment to the system, and the system outputs are those that are received by its environment. Testing machines for concrete specimens and structural elements can be considered as systems, whose components are the actuator, test frame (including the loading fixtures), controller, transducers, and the specimen itself. The inputs are the loading functions, such as loading rates and waveforms imposed by the operator, while the outputs are transducer signals that can be converted to data. The capabilities of the testing system reflect its ability to respond accurately to an extensive range of inputs. This depends mainly on the controller and the manner in which the actuators are controlled. In general, the control can be classified as open loop or closed loop, where the loop signifies the use of the system output as feedback by the control process. In an open-loop control, the output is not used by the controller, and the process depends only on the system input (Figure l-11 (a)). In this kind of systems the controlled variables are usually the actuator (piston) displacement and applied load or pressure which is not significantly affected by the behaviour of the test specimen.

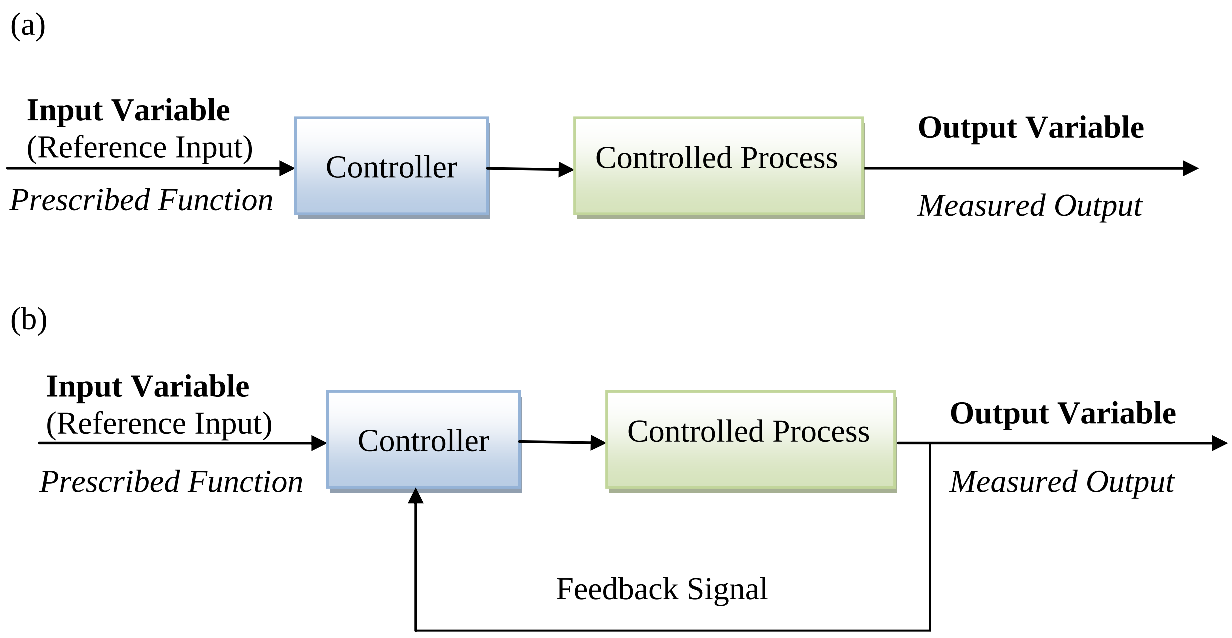

Figure 1-11 (a) Open Loop Control; (b) Closed Loop Control.

This is similar to other automated systems such as programmable washing machines, toasters, coffee machine etc. In a closed-loop system the output of the controlled variable is directly monitored by the controller (Fig. l-11 (b)). This can, therefore, be any capacity that is accessible to the controller, such as specimen displacement, strain, and crack opening. The actual and desired (reference input) values are equalized indirectly by the controller by manipulating the movement of the actuator. In closed-loop controlled systems, as shown simply in Figure l-11(b), the current value of the controlled variable is fed back to the controller and compared with the reference input signal. The difference between the two signals (i.e., the error) is used to manipulate the actuator, and, therefore, the process is also known as negative feedback control. Obviously, the scope of closed loop control is greater than open loop control, because the range of controlled variables is much wider. Even for the same controlled variable, say, piston displacement, the closed-loop system produces a more accurate output than the open-loop system.

However, closed-loop-control also has a few drawbacks;

- This kind of sophisticated system requires complex instrumentation and data acquisition system which results in higher initial cost which may not be always possible for every lab to provide that.
- The system requires more operator skills. Improper use could make the system unstable and oscillatory. This may produce misleading data too.
- There is always a lag between the actual response and the corrective action of the controller, which may result in the loss of control, overcorrection, or under correction.
- These kinds of test are difficult to run and time consuming.

Due to these considerations, closed-loop controllers have to be properly designed through modeling and analysis. CLC is most useful when there is a rapid and unpredictable change in system input or in the specimen behaviour. Therefore, the transient response of the system in the time domain is important. This is normally evaluated by imposing a step input and the response parameters.

Concrete specimen with low fibre volume fraction behave very unpredictably specially when are tested at their early ages i.e. at 7 days of casting. Precaution must be taken while testing this kind of specimens. Since they are very unpredictable CLC will be one of the best solutions. The most common feedback control signal is in the form of specimen deformation. A closed-loop system can provide a stable deformation rate and produce a stable specimen response, thereby improving precision. Improved stability and precision are of particular interest in testing cementitious materials, as they are brittle and often display instability at the instant of cracking. Banthia et al suggested performing the ASTM C1399/C 1399-10 only in closed loop environment especially for the high strength fiber reinforced concrete because of the control on the displacement which provide a more stable specimen and produce the data that can be relied on (Banthia et al. 2011).

1.11 Data Analysis Technique Associated With FRC Characterization (Strength versus Toughness)

Strength and toughness are generic terms useful only when they are precisely defined and determined. As regards FRC, no single definition is universally accepted. Furthermore, consent on a definition does not appear to be forthcoming and in fact may not even be necessary. Strength is considered a stress capacity, and toughness an energy capacity. Up to today, most of the techniques for characterizing FRC toughness such as standards, guidelines from standard institutions, various professional agencies and published literature can be broadly divided into two categories;

- A major part of the proposed techniques are energy based. Many tests have been developed to directly characterize the energy absorption capacity of cementitious composites in simple loading configurations such as compression, flexure and tension. Among them the flexural test is the most popular because it represents more realistically the conditions in many practical situations and is simpler to conduct than other tests. The results allow toughness characterization through one or more of the following: absolute energy absorption, dimensionless indices related to energy absorption capacity, equivalent flexural strengths at prescribed post-cracking deflections or other parameters that describe the post-cracking response of the composite. Results from these tests are usually affected by the specimen size and geometry although they are intended to characterize the material behavior only. Nevertheless, these tests have potential engineering uses as evident by the sudden increase, in recent years, of standards and recommended procedures (ACI Committee 544 1988; AENOR 1989; ASTM C1018; ASTM C1609; Banthia and Trottier 1995; EFNARC 1997; JSCE-G 552 1999; RILEM TC 162-TDF 2002) for the flexural toughness testing of FRC. Critical review of these standards and other significant proposals (Barr and Liu 1982; Barr et al. 1982; Barr and Hasso 1985; Gopalaratnam et al. 1991; Gopalaratnam et al. 1991; Henager 1978; Banthia and Trottier 1994; Wang 1989) available in the published literature is undertaken to identify possible improvements in toughness characterization procedures and use. These test procedures are particularly worrisome when concrete reinforced with low volume fraction of synthetic or other fibers tested in an open loop system (Banthia et al 2011). Despite their drawbacks the tests are useful to some extent in comparing the relative performance of different mix proportions and in providing information on strength as well as toughness for the particular parameters defined in the test.

- Another technique to characterize FRC is calculating Average Residual Strength, described in ASTM C1399-10 (ASTM C1399), which is basically a four point bending test divided into two part as describe in Section 1.7.9. The procedure is based on assessing the fiber's capability to improve toughness by the ability to transmit stresses across matrix cracks. Proponents of C1399 argued that, rather than define and measure the energy absorption capability which is a doubtful concept in itself, it makes more sense to underscore toughness by quantifying the magnitude of stresses fibers can transmit beyond matrix cracking. This property, often called post-cracking strength (or simply the residual strength), is measured in stress units and is thus a derived measure of toughness. This way of quantifying the contribution of fibers to concrete is easy to comprehend, unambiguous, and also highly relevant in designs. Residual Toughness Strength Method (RSTM) is proposed by Banthia et al (Banthia and Dubey 1999; Banthia and Dubey 2000) which is approved by ASTM (as C1399) to provide controlled cracking in the specimen to eliminate the problem of instability.

Strength and toughness measurements are affected by the particular testing machinery and measurement devices employed, and by the size and shape of test specimens. As it is generally accepted that the principal benefit of fiber reinforcement relates to tensile stress and strain capacities of the composite, cracking and crack propagation are the failure events most often used in strength and toughness definitions. But these events cannot themselves be precisely determined, which makes comparison of test results among testing laboratories problematic. Nevertheless, strength and toughness, both in pre- and post-cracking regimes of performance, are the parameters best suited for establishing design criteria for FRC. Precise determination of strength and toughness, however specified, generally requires sophisticated and costly testing procedures. Such procedures are thought to be more applicable to R&D efforts than they are to production and quality control testing. Measurements which are generally required for engineering design and specification or for quality control should be obtained with less effort. Test methods which integrate over the imprecisely defined events that make testing problematic, such as cracking, are under development and are discussed later. Test procedures which are mainly based on converting load values to stress values provide a discontinuous measure of toughness particularly in the cracked zone as in the calculation they only consider loads at some particular points. For a specific beam specimen it might have either higher or lower load values at those points and misleading results may be produced.

2 Loading Rate Concerns in ASTM C 1609

2.1 Outline

ASTM C1609 remains one of the most prescribed tests for characterizing the performance of fiber reinforced concrete (FRC). Although significant progress has been made over the years in addressing the deficiencies that existed, concerns persist with the loading rate prescribed in the current version of test.

A test program was carried out to investigate the influence of loading rate in ASTM C1609-2010. Normal strength fiber reinforced concrete with 0.1% and 0.3% of polypropylene fiber was tested. Results indicate that while the prescribed loading rate is appropriate for the 0.3% fiber volume FRC, it is too high for FRC with 0.1% fiber volume. To obtain a stable load-deflection curve in FRC with 0.1% fiber by volume, a reduced loading rate was required. In the context of these findings, a reduced loading rate is proposed in this chapter for performing C1609 which is being consistent for rest of experiments in chapter 3 and 4.

2.2 Introduction

Among the flexural tests, ASTM C1609 remains one of the most performed. The test requires a closed-loop machine that operates under feed-back control from a transducer placed on the specimen, and the resulting load-deflection curve is analyzed to obtain the specified toughness parameters. In response to the criticism from the laboratories, the specified loading rate was reduced in the current version of the test (ASTM C1609) for high strength concrete. In the latest version of ASTM C1609 in Section 9.4, it is permitted to reduce the initial net deflection rate by 50% for high strength concrete. The rationale is that high strength concrete would generally depict greater brittleness and hence a slower rate of loading will be necessary to avoid a sudden failure and obtain a stable curve. The current specified rates are given in Table 2-1.

Table 2-1 Rate of Increase of Net Deflection as Per ASTM C1609-2010

	Beam Size	**Up to the net deflection of L/900**	**Beyond net deflection of L/900**
Regular Strength Concrete	100 X 100 X 350 mm	0.025 to 0.075 mm/min	0.05 to 0.20 mm/min
	150 X 150 X 500 mm	0.035 to 0.10 mm/min	0.05 to 0.30 mm/min
High Strength Concrete	100 X 100 X 350 mm	0.0125 to 0.0375 mm/min	0.05 to 0.20 mm/min
	150 X 150 X 500 mm	0.0175 to 0.05 mm/min	0.05 to 0.30 mm/min

Where “L” is span of the beam specimen. A test program was undertaken to investigate if the rates specified in the current standard for normal strength concrete were appropriate.

2.3 Experimental Program and Results

2.3.1 Fiber Specification

A polymeric fiber (Figure 2-1) produced by Wildfibre LLC was used. The 24 mm fiber has a twisted cross-section and enlarged ends.

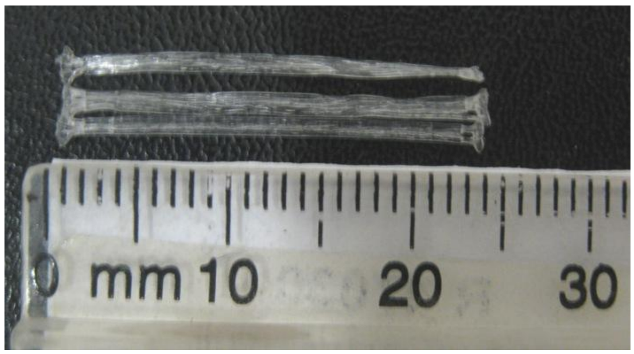

Figure 2-1 Polypropylene (PP) Fiber Investigated

2.3.2 Materials and Mixes

ASTM Type GU Portland cement was used in all concrete mixtures. Locally available natural river sand and gravel were used as aggregates. The mixture proportions of the concrete are given in Table 2-2. Two volume fractions of the PP fiber were investigated: 1.0 kg/m^3 and 3.0 kg/m^3. For each mix, five 100 mm x 200 mm cylinders for compressive strength determination as per ASTM C39/C39M–09 were cast along with ten 100 mm x 100 mm x 350 mm beams for performing toughness tests as per ASTM C1609/C1609 M-10. Specimen were cast in reusable plastic moulds, consolidated on a vibration table, demolded a day later and then cured in lime saturated until tested up to 7days.

Table 2-2 Concrete Matrix Mix Proportion

Materials		kg/m^3
CSA Type 10(ASTM type I) Portland Cement		400
Water		180
Fine Aggregate (Sand)		560
Coarse Aggregate(Gravel 3/8")		1110
w/c ratio		0.45
f'_c		40 MPa
PP Fiber	Mix M1	3.0
	Mix M2	1.0

2.3.3 **Experimental Procedure**

The C1609 test setup used is shown in Figures 2-2(a) & 2-2(b). A closed-loop, fatigue-rated Instron 8802 test machine was used. A 'yoke' was installed around the specimen to eliminate spurious deformation arising from crushing and support settlement and to record only the 'net' deformation of the neutral axis. Two Linear Variable Displacement Transducers (LVDTs) were mounted on opposite sides of the specimen to measure the average deflection. The LVDTs also provided feedback to the servo-valve for closed-loop control in the test.

(a) (b)

Figure 2-2 (a) – ASTM 1609 Tests Setup; **(b) –** Close-up View of the Yoke Assembly and Instrumentation

Compressive strength tests yielded an average compressive strength of 40 MPa (Table 2-2) for the concrete cast. Therefore, concrete used in these tests was claimed to be of 'Normal' strength.

On the 7th day of curing age, the ten beams were divided in to two groups of five beams each. The first five beams (termed Series I) were tested at the loading rate specified in C1609. The remaining five beams (termed Series II) were tested at a significantly lower rate of loading.

2.4 Results and Discussion

2.4.1 Series I Tests

For Series I tests, the exact rate of loading prescribed in ASTM C1609-2010 for normal strength concrete was used. The load deflection curves are shown in Figure 2-3 for M1 (3.0 kg/m^3) and in Figure 2-4 for M2 (1.0 kg/m^3). The analysis is given in Table 2-3.

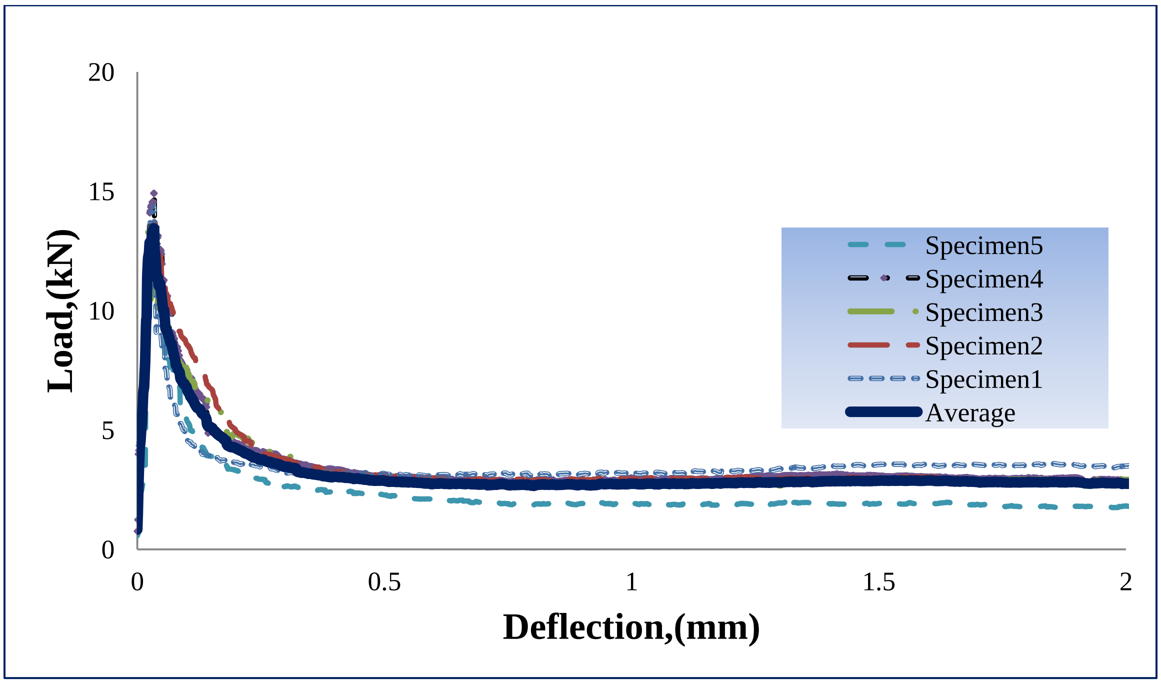

Figure 2-3 Series I Load-Deflection Curves for M1 with 3.0 kg/m^3 of Fiber with the Prescribed Loading Rate

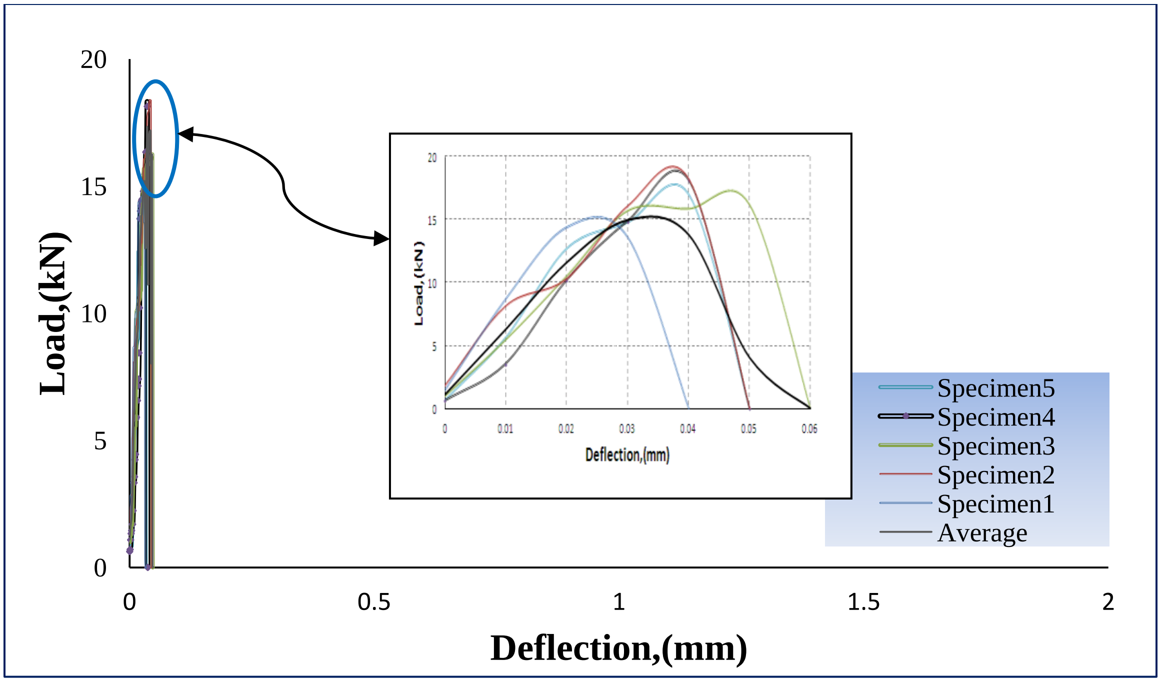

Figure 2-4 Series I Load-Deflection Curves for M2 with 1.0 kg/m3 of Fiber with the Prescribed Loading Rate

Table 2-3 Toughness Parameters Derived Using the Specified Loading Rate (Series I Tests)

ASTM 1609 Parameter	Mix	
	M1	M2
P1 (kN)	13.52	16.12
(MPa)	4.05	4.836
δ1 (mm)	0.03	0.036
$P_{100,0.5}$ (kN)	2.78	0.0
$f_{100,0.5}$ (MPa)	0.86	0.0
$P_{100,2.0}$ (kN)	2.79	0.0
$f_{100,2.0}$ (MPa)	0.84	0.0
$\text{Toughness}_{100,2.0}$ (J)	6.60	0.0
R_{600}^{100}(%)	21.0%	0.0
Co-efficient of Variation	9.97%	N/A

Notice in Figures 2-3 and 2-4 and Table 2-3 that the use of the prescribed rate of loading resulted in a stable curve in the case of M1 (3.0 kg/m^3 fiber content) but not in the case of M2 (1.0 kg/m^3 fiber content). Thus the current rate of loading specified in the standard is too high even for normal strength concrete tested here.

2.4.2 **Series II Tests**

In the second series of tests, the rate of increase of net deflection was reduced significantly over the one specified (see Table 2-4). It can be noted that, the loading rates are divided into two parts. The first part is up to deflection L/900 and the second part is beyond L/900, where L is the span of the beam. Even the initial loading rate started from a very low as 0.001mm/min, it can be increased after the beam reaches its peak load and pass another 0.05 mm of deflection. During the test, when the beam shows its stability, the loading rate can be further increased to finish the test within a reasonable time limit around 60 to 75 minutes.

Table 2-4 Rate of Increase of Net Deflection Series I and Series II

	Rate of Increase of Net Deflection	Up to the net deflection of L/900	Beyond net deflection of L/900
Series I	Prescribed	0.025-0.075mm/min	0.05-0.20mm/min
Series II	Reduced	0.001-0.015mm/min	0.02-0.15mm/min

The load deflection curves for Series II are shown in Figure 2-5 for M1 (3.0 kg/m3) and in Figure 2-6 for M2 (1.0 kg/m3). The analysis is given in Table 2-4. Notice in Figures 2-5 and 2-6 and Table 2-5 that a reduction in the rate of loading resulted in stable curves for both M1 and M2. Mix M2 which had failed to show any post peak toughness (Table 2-3) was now able to depict a non-zero R_{600}^{100} value of 10%. Furthermore, the toughness parameters for M1 were comparable at the two rates indicating that a slower rate did not unduly affect the measured toughness characteristics.

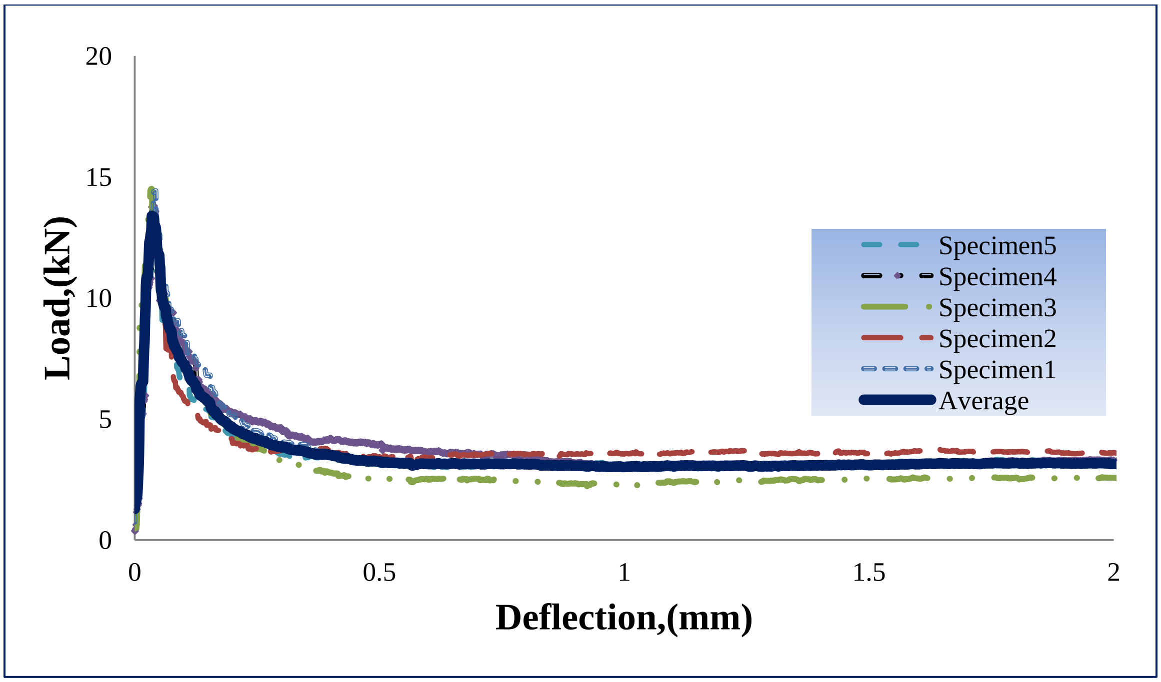

Figure 2-5 Series II Load-Deflection Curves for M1 with 3.0 kg/m^3 of Fiber with the Reduced Rate of Loading

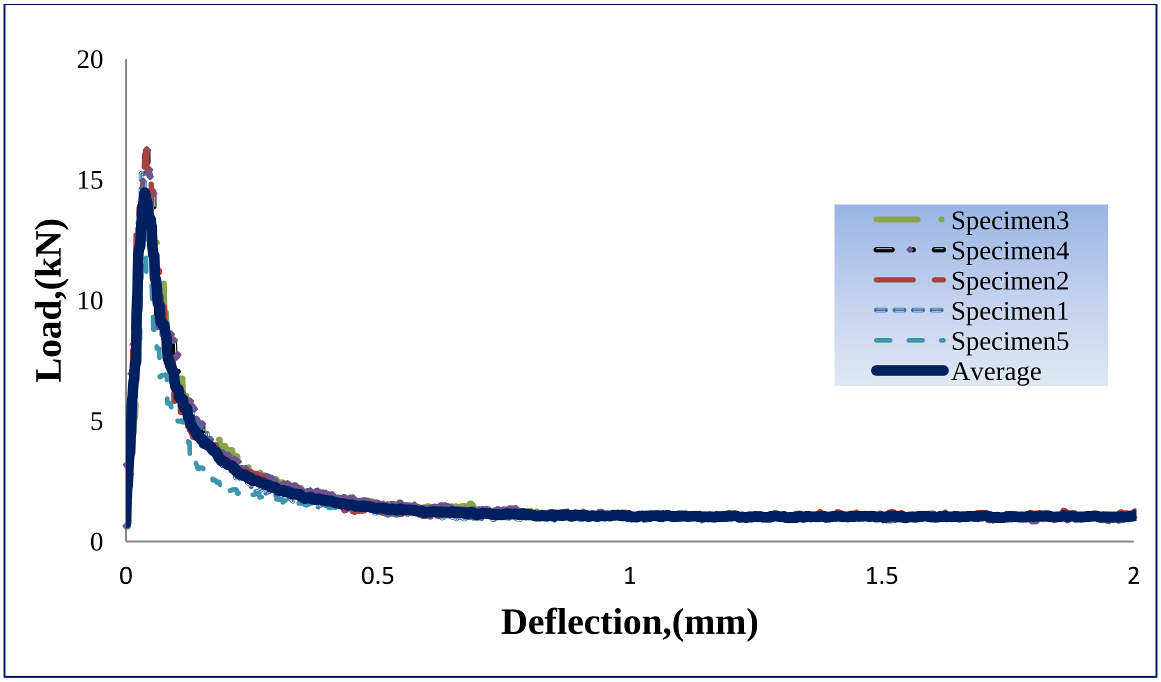

Figure 2-6 Series II Load-Deflection Curves for M2 with 1.0 kg/m^3 of Fiber with the Reduced Rate of Loading

Table 2-5 Toughness Parameters Derived Using the Reduced Rate of Loading (Series II Tests)

ASTM 1609 Parameter	Mix	
	M1	M2
P1 (kN)	13.58	15.39
(MPa)	4.07	4.61
δ1 (mm)	0.039	0.04
$P_{100,0.5}$ (kN)	3.23	1.56
$f_{100,0.5}$ (MPa)	0.971	0.469
$P_{100,2.0}$ (kN)	3.17	1.52
$f_{100,2.0}$ (MPa)	0.952	0.460
$Toughness_{100,2.0}$ (J)	7.23	3.67
R_{600}^{100}(%)	24.0%	10%
Co-efficient of Variation Of R_{600}^{100} (%)	6.61%	6.40%

2.5 Conclusion and Recommendation

Based on the results of these tests, it can be stated that the standard loading rate is described in ASTM C1609 is not effectively applicable for all types of specimens and may applicable only for specimens with higher fiber volume fraction. So it can be assumed that while the prescribed loading rate is appropriate for the 0.3% fiber volume FRC, it is too high for FRC with 0.1% fiber volume. In order to obtain a stable load-deflection curve in FRC with 0.1% fiber by volume, a reduced loading rate was required. In the context of these findings, it can be concluded that:

1. For FRC Toughness Characterization initial loading rate described in ASTM C1609 is high enough to fail the specimen. These tests might only be efficiently performed for all kind specimens with different fiber volume fraction with the proposed new loading rate described in Table 4 Series II which is a reduced value of 0.001-0.015 mm/min up to a net deflection of L/900 and 0.02-0.15 mm/min after a net deflection of L/900. Even though initially it's going to take longer time but sudden failure of specimen can be avoided which occurs due to high initial loading rate and also consistent result will be produced eventually.

2. So in performing ASTM C1609-10 either the test method should clearly indicate it's limitation for brittle materials or the lower initial loading rate proposed at Table 2-4 Series II should be applied for all kind of concrete specimens.

3. In this study only one set of reduced loading rate is evaluated. There is still room for further investigating the optimum loading rate for different specimen based on various volume fractions, age, and matrix strength as concrete will behave differently under different conditions.

3 Comparison between ASTM C1609 and ASTM C1399

3.1 Outline

Characterization of Fiber Reinforced Concrete (FRC) remains a challenge. The Canadian Highway Bridge Design Code (CHBDC-S06-16) describes a method to calculate Residual Strength Index (R_i) for Fiber Reinforced Concrete (FRC) which requires running two separate sets of tests under both ASTM C1399 and ASTM C78. This method becomes very time consuming and tricky to maintain consistency since 10 specimens have to be tested after the 7th day of casting. In reality it is often impossible to test a batch of 10 specimens in one day. Towards this end, an alternative approach is proposed by conducting ASTM C1609 tests alone and test results are compared with the existing CHBDC procedure. Tests were performed on specimens of same dosage of a polymeric fiber under both ASTM C1399 and ASTM C1609. *Ri* values were calculated for both methods and compared. The results indicate that R_i values are very similar to one other for both methods while the proposed ASTM C1609 can be performed more effectively and consistently using only half the specimens. In the context of these findings, it is recommended that CHBDC method can be conveniently replaced by proposed ASTM C 1609 Method.

3.2 Introduction

Considerable drawbacks of Canadian Highway Bridge Design Code CHBDC-S06-10 procedure can be described as follows:

- It requires at least 10 good specimens for testing, 5 of each under ASTM C 1399 and ASTM C78 cast on the same day around same time which might not be practical for the construction site from where the specimen will be produced.
- The tests itself requires a long time. For example one ASTM C78 takes around 15-20 minutes and for ASTM C 1399 it is 75-90 minutes. Overall for doing one full batch of 10 specimens the total minimum time required is 5*20+5*90=550 minutes which is about 9.17 hrs! Typically it is impossible to perform a batch of 10 specimens on the same day.
- Another concern would be the consistency of the data produced. For calculation purpose Modulus of Rupture, R vales are taken as average of 5 specimens and ARS values come from 5 different specimens. Although all the specimens are from the same batch but uniformity in the specimens is hard to maintain.
- Doubts are often raised for C1399 to be able to capture a true closed loop response. In an uncontrolled open-loop test, during initial loading, the deflection is very hard to control and the net deflection requirements are seldom met. This is of particular concern in the case of very high strength matrices. (Banthia et al. 2011)
- ASTM C1609 other hand is a feedback controlled with stability and reliability.

Considering all these matters with CHBDC and ASTM C1399, a new method is proposed in this chapter where the specimens are tested only under ASTM C1609 following the loading rate suggested in Chapter 2 and load deflection curves are analyzed and results compared with traditional CHBDC procedure that require 10 specimens.

3.3 Experimental Setup

3.3.1 Fiber Specification

For this study MasterFiber MAC100 product, a macro synthetic reinforcing fiber manufactured from a proprietary blend of polypropylene resins was used. Fiber properties are summarized in Table 3-1 and shown in Figure 3-1.

Table 3-1 Properties of Fiber Investigated (Source: http://www.basf-admixtures.com)

Property	MasterFiber® MAC100
Specific Gravity	0.91
Configuration	Highly modified collated fibrillated
Tensile Strength	60,000psi (415 MPa)
Available Length	1.5inch (38mm)
Water Absorption	Nil
Chemical Resistance	Excellent
Alkali Resistance	Excellent
Melting Point	320 ^{o}F (160 ^{o}C)
Ignition Point	1094 ^{o}F (590 ^{o}C)

Figure 3-1 MAC100 Fiber

3.3.2 **Materials and Mixes**

Normal Portland cement (Lafarge Type GU), river sand crushed stone and portable tap water was used in all concrete mixes. The mix proportions of the concrete are given in Table 3-2.The design strength for all the batches were 35 MPa at 28 days.

For each mix-design beam specimens with dimensions of 100 mm x 100 mm x 350 mm were cast at a construction site. All the specimens were cast in reusable plastic moulds, consolidated by vibration, allowed to set in a sealed environment, then demolded after 24 hours later, and then cured in lime saturated water. They were then transported from the construction site to the laboratory for testing purpose. The flexural toughness tests were performed at an age of 7 days as per ASTM C1609/C1609 M-10 and ASTM C1399/C1399-10.

Table 3-2 Concrete Matrix Mix Proportion

Materials	**kg/m^3**
CSA Type 10(ASTM type I)	450
Water	166
Fine Aggregate (Sand)	760
Coarse Aggregate(Gravel 3/8")	910
w/c ratio	0.37
MAC100 Fiber	2.294
Air Entrainment	85ml/100kg
Water Reducer	550ml/100kg

3.3.3 **Experimental Procedure**

For this study 6*10=60 regular strength FRC specimens from 6 different construction site having same fiber volume fractions were tested. To calculate the Modulus of Rupture (MOR) instead of performing ASTM C78, ASTM C 1609 was carried out on 30 specimens. Table 3-3 described the experimental program in details. MOR was estimated according to ASTM C1609 using following equation.

$$MOR = f_p = \frac{Pl}{bd^2} \qquad [3.3]$$

Where,

MOR= Modulus of Rupture.

f_p= Peak Strength.

P = Maximum applied load indicated by the testing machine, N [lbf],

L = Span length, mm [in.],

b = Average width of specimen, mm [in.], and

d = Average depth of specimen, mm [in.].

Table 3-3 Experimental Program

	No. of Specimen	
	ASTM C1399	ASTM C1609
Batch 1	5	5
Batch 2	5	5
Batch 3	5	5
Batch 4	5	5
Batch 5	5	5
Batch 6	5	5

3.4 **Result and Discussion**

Load-deflection curves were produced and results were analyzed for the calculation of Equivalent Flexural Strength Ratio R_i value according to CHBDC following the equation 3-2. R_i values were also estimated from ASTM C 1609 curve and compared with ASTM C 1399 outcomes. Figure 3-2 to 3-7 represents the load deflection curve for ASTM C1399 and 3-8 to 3-13 represents same for ASTM C1609. Comparison between ASTM C1399 and ASTM C1609 is described in Figures 3-14 to 3-19 which shows that after around 0.25mm deflection, both of these curves overlap each other.

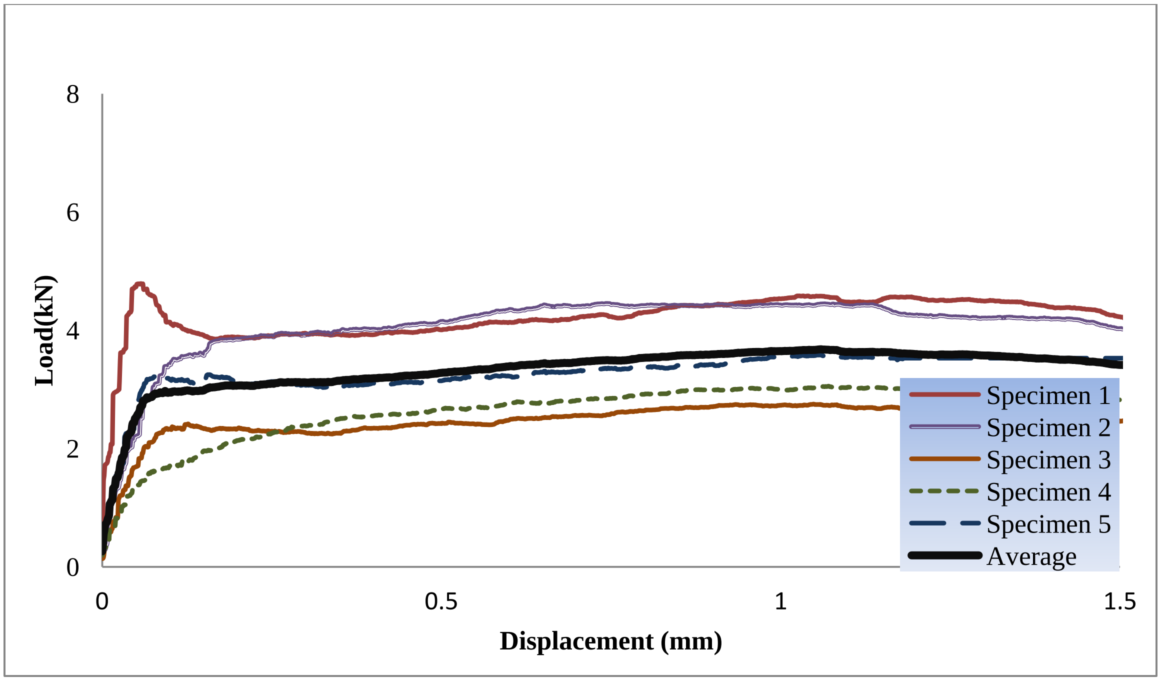

Figure 3-2 Re-Loading Curve According to ASTM C1399 (Useful for ARS Value) for Batch 1

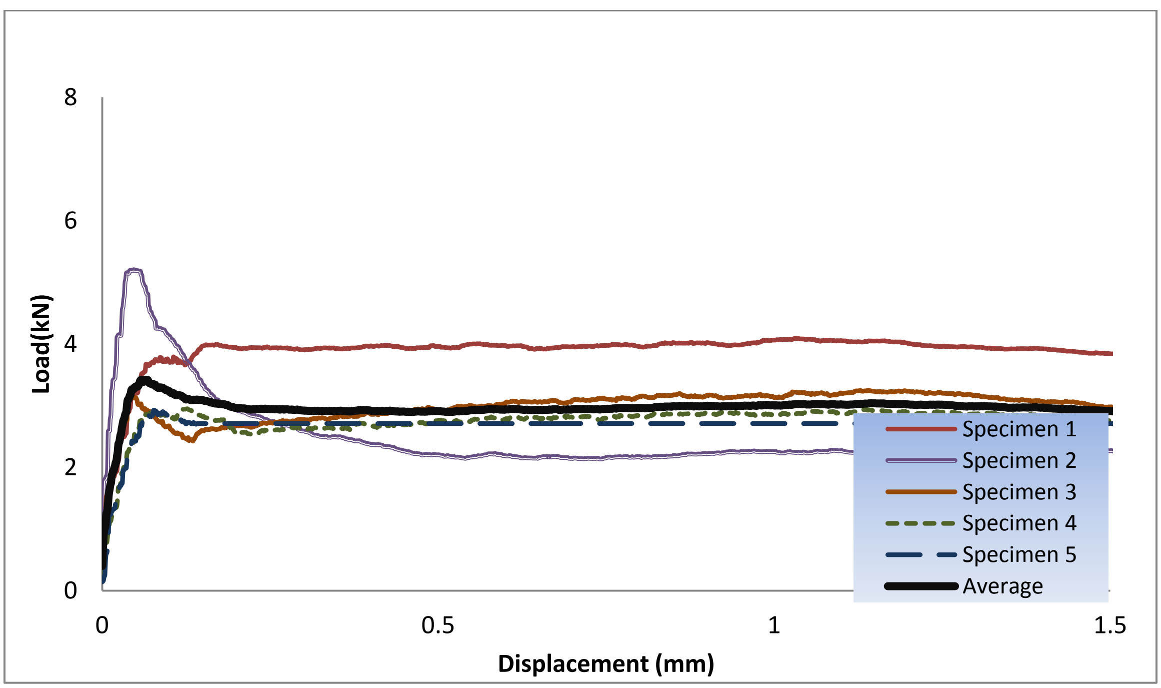

Figure 3-3 Re-Loading Curve According to ASTM C1399 for Batch 2

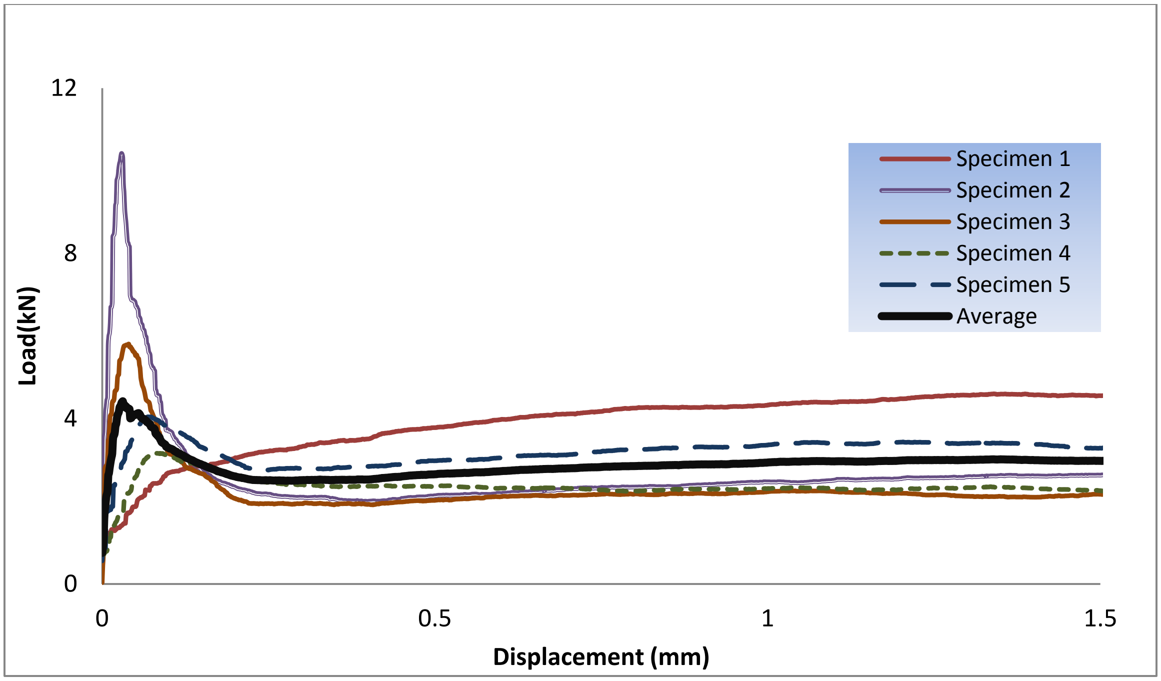

Figure 3-4 Re-Loading Curve According to ASTM C1399 for Batch 3

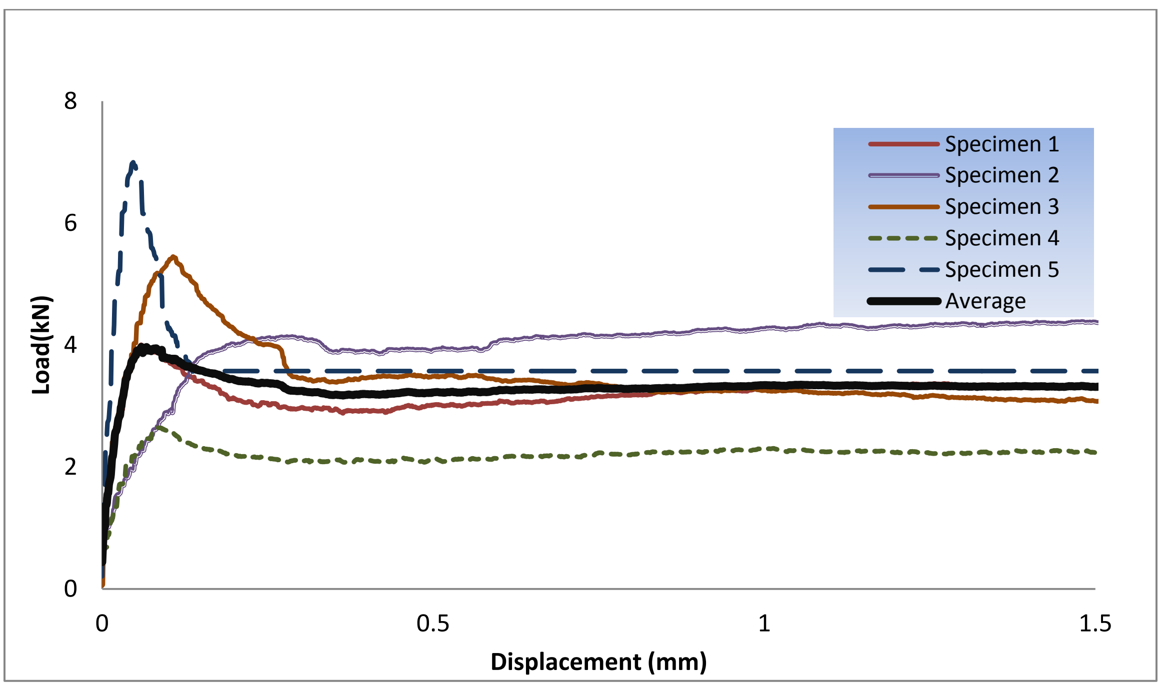

Figure 3-5 Re-Loading Curve According to ASTM C1399 for Batch 4

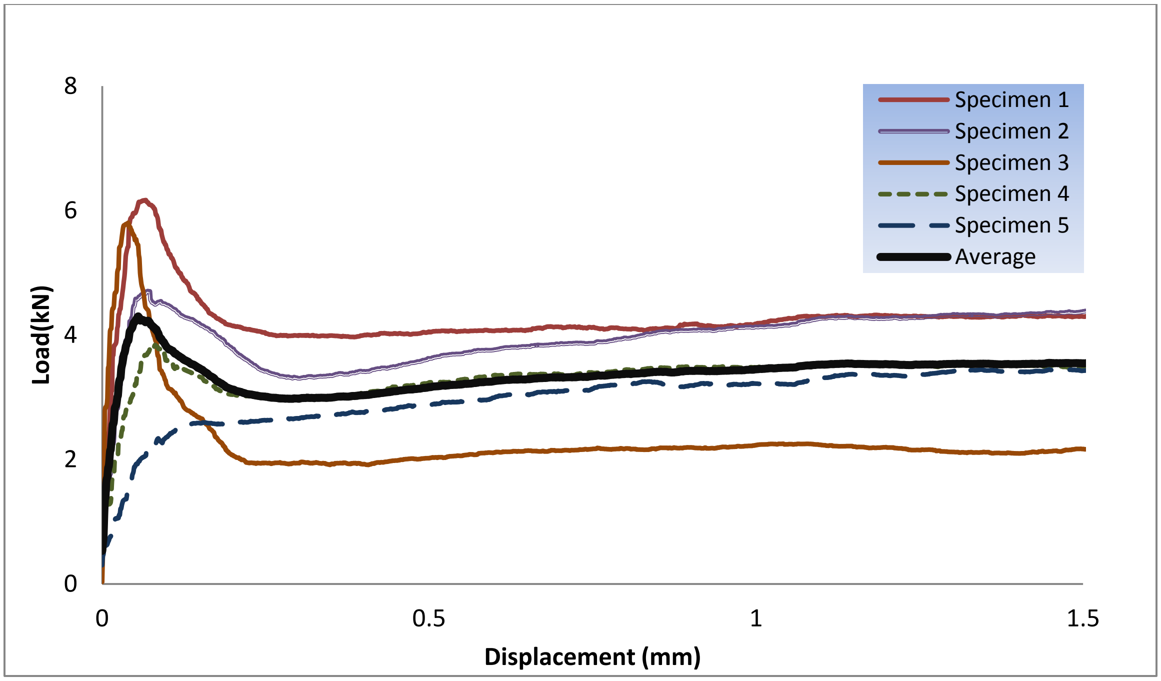

Figure 3-6 Re-Loading Curve According to ASTM C1399 for Batch 5

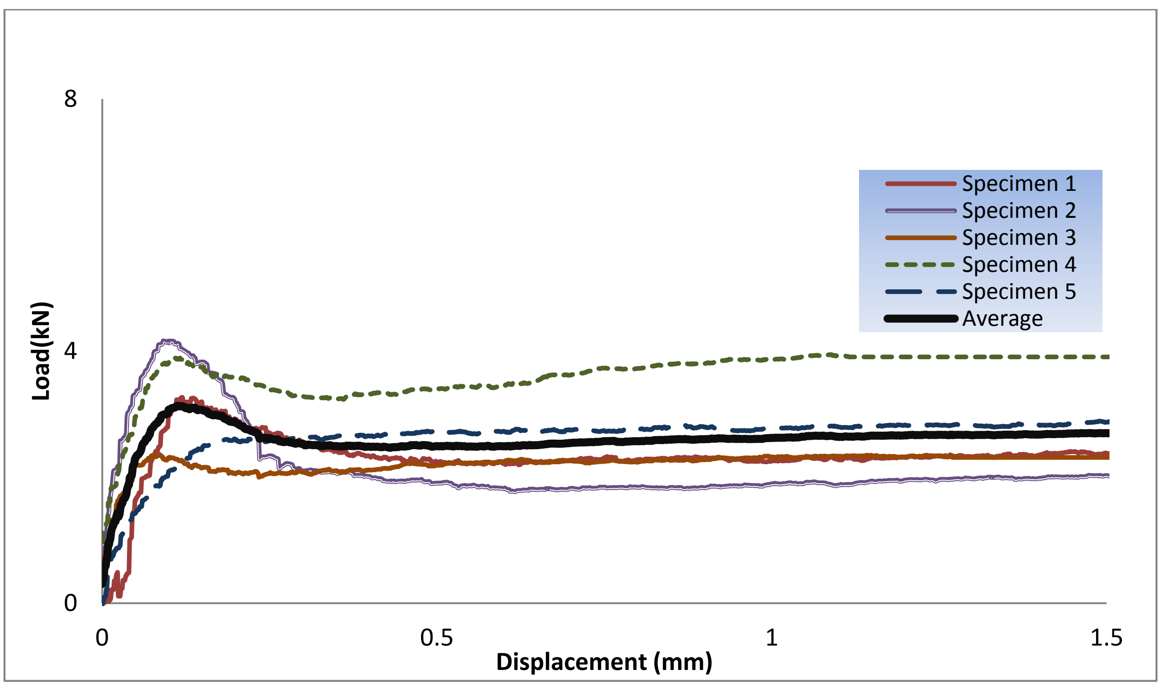

Figure 3-7 Re-Loading Curve According to ASTM C1399 for Batch 6

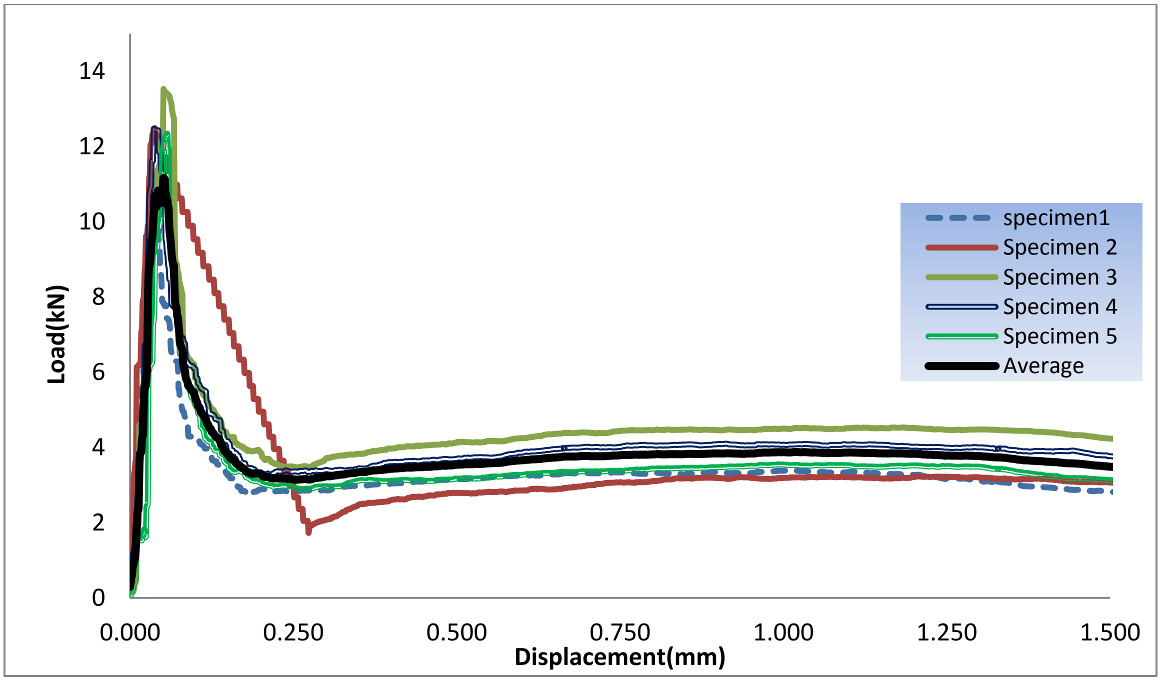

Figure 3-8 Load Deflection Curve using ASTM C 1609 for Batch 1

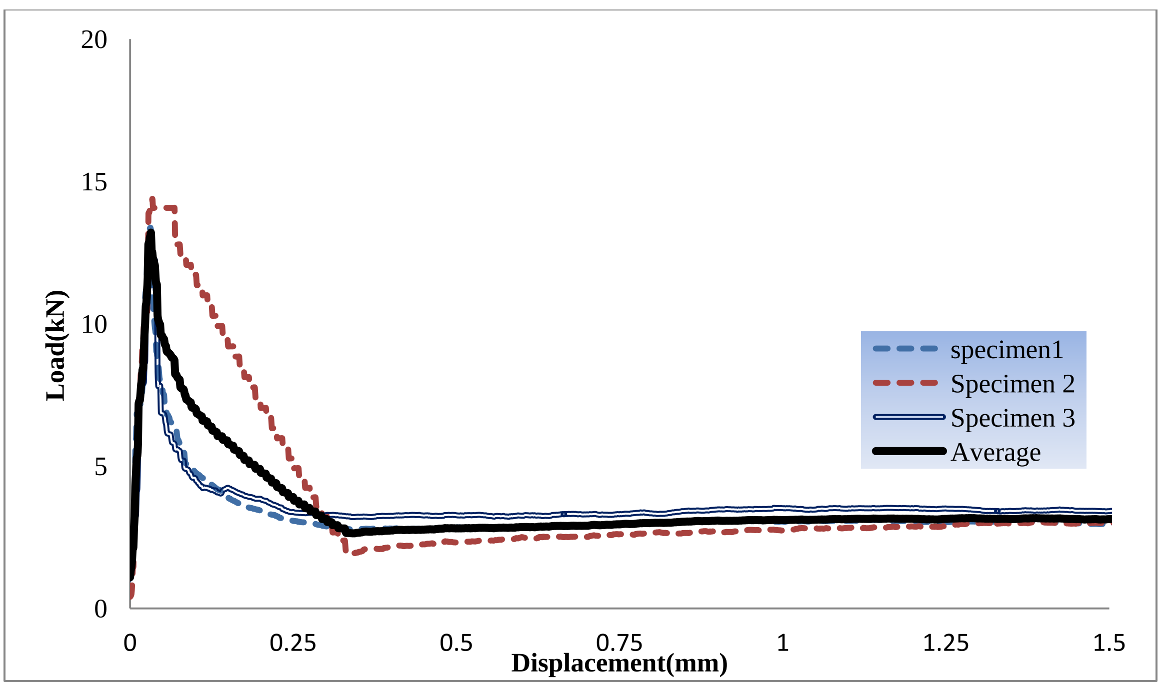

Figure 3-9 Load Deflection Curve using ASTM C 1609 for Batch 2

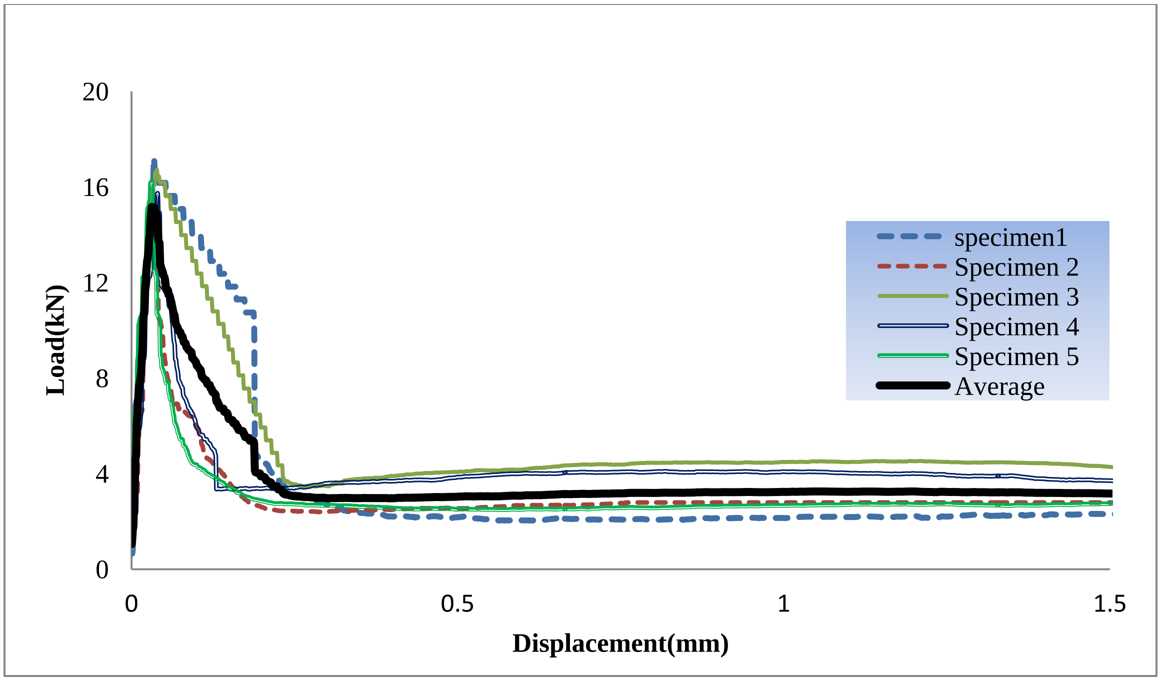

Figure 3-10 Load Deflection Curve using ASTM C 1609 for Batch 3

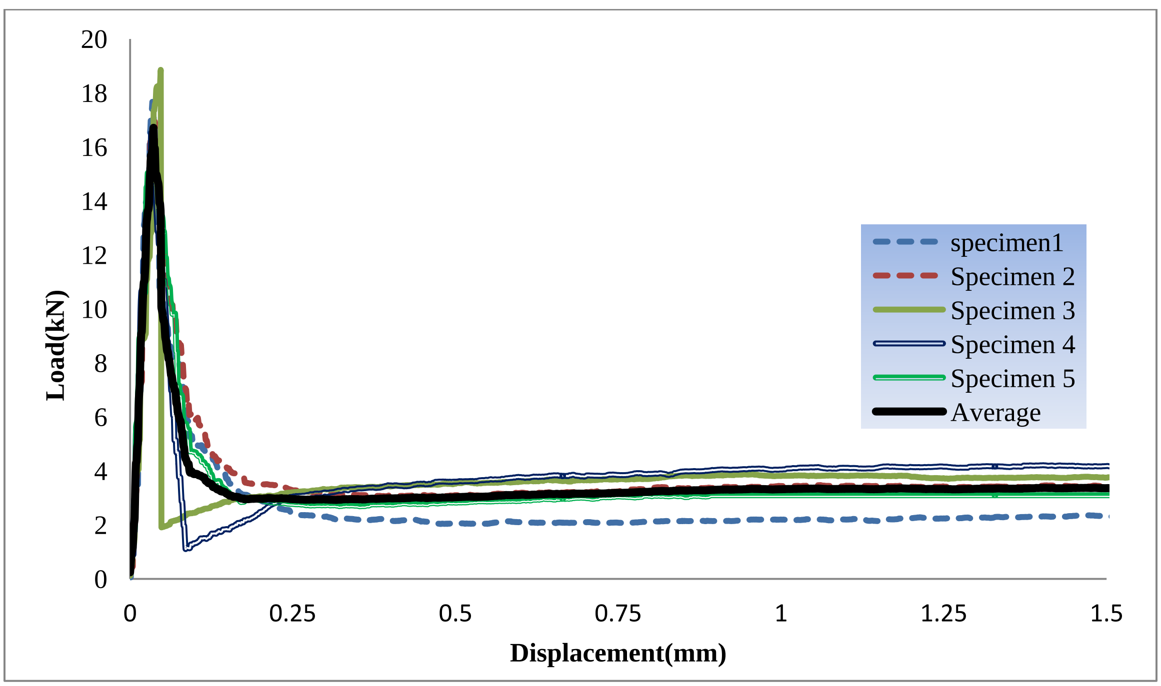

Figure 3-11 Load Deflection Curve using ASTM C 1609 for Batch 4

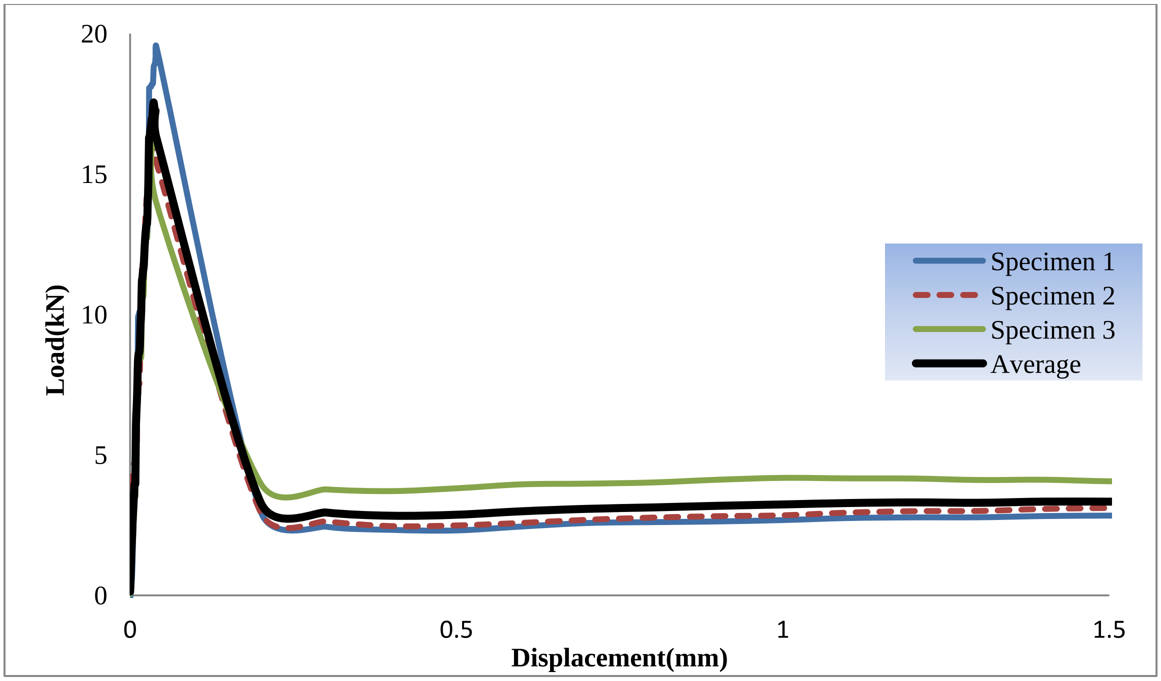

Figure 3-12 Load Deflection Curve using ASTM C 1609 for Batch 5

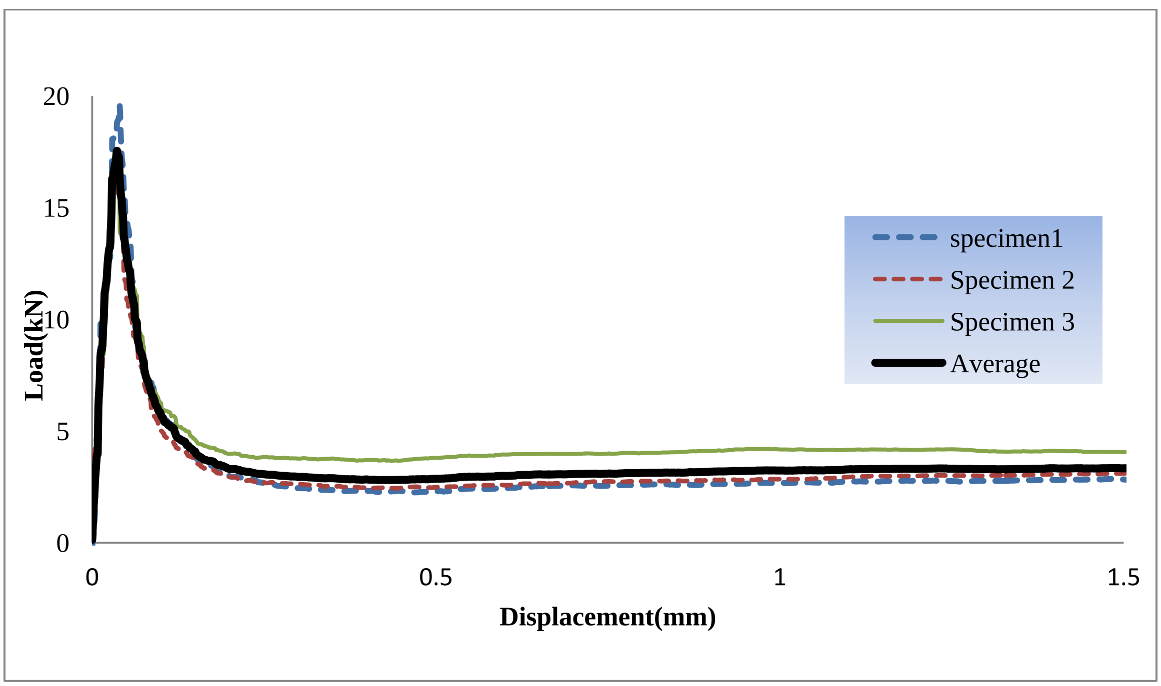

Figure 3-13 Load Deflection Curve using ASTM C 1609 for Batch 6

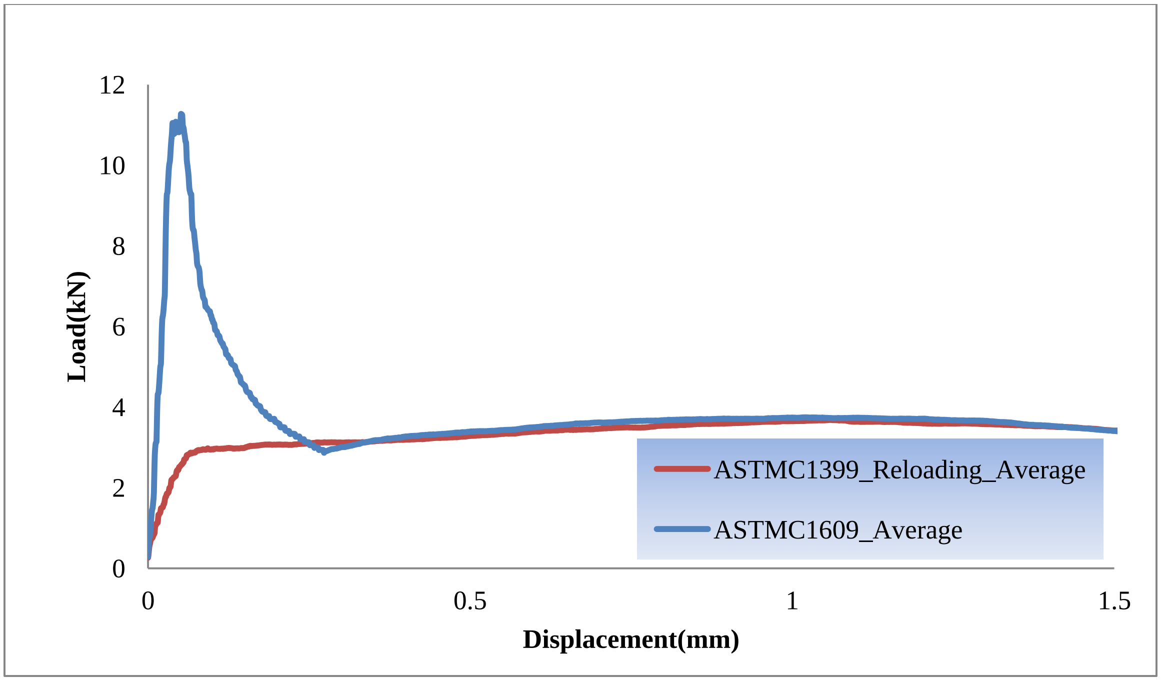

Figure 3-14 Comparison between Load Deflection Curve for ASTM C1399 and ASTM C1609 for Batch 1

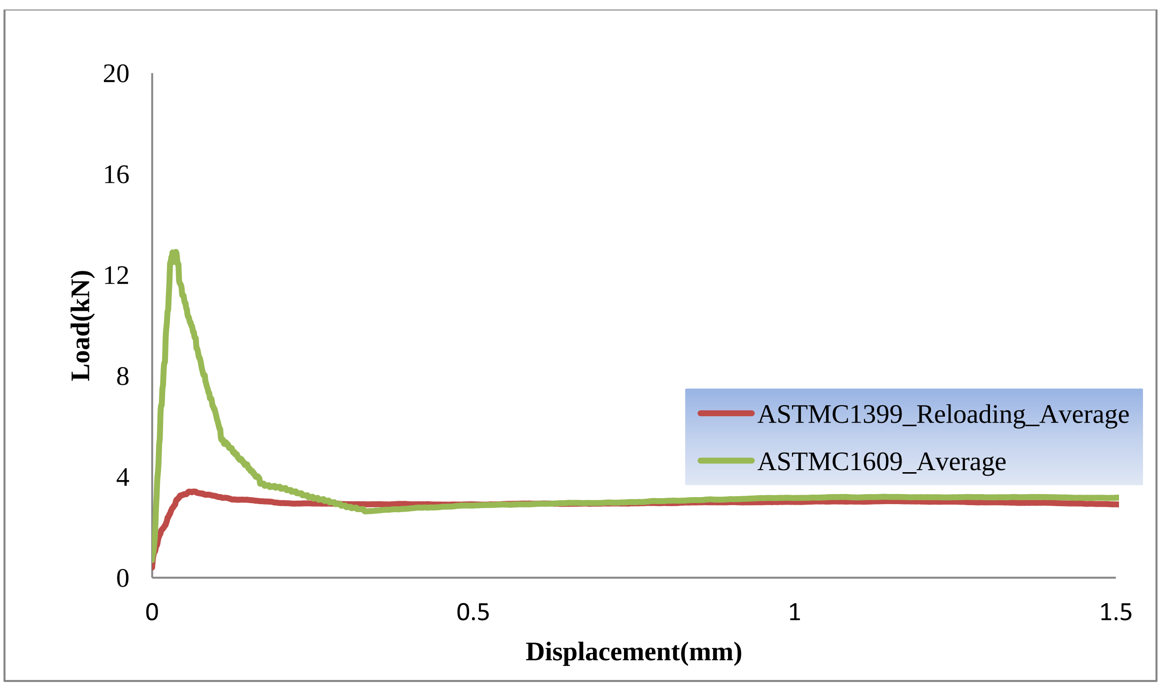

Figure 3-15 Comparison between Load Deflection Curve for ASTM C1399 and ASTM C1609 for Batch 2

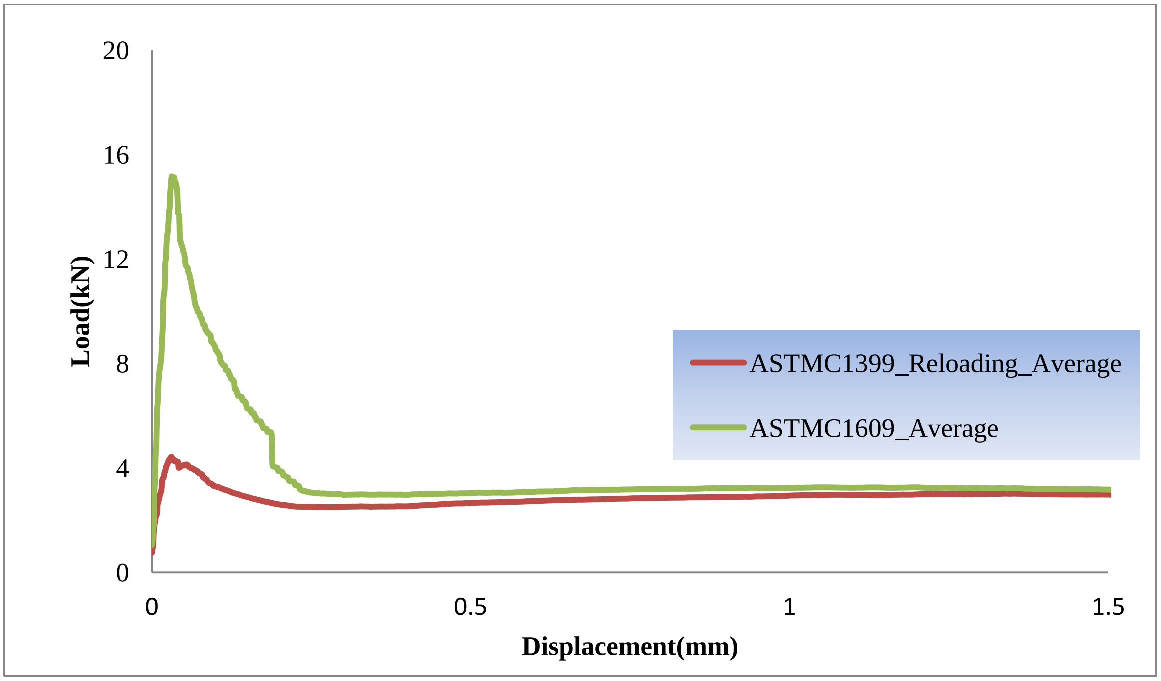

Figure 3-16 Comparison between Load Deflection Curve for ASTM C1399 and ASTM C1609 for Batch 3

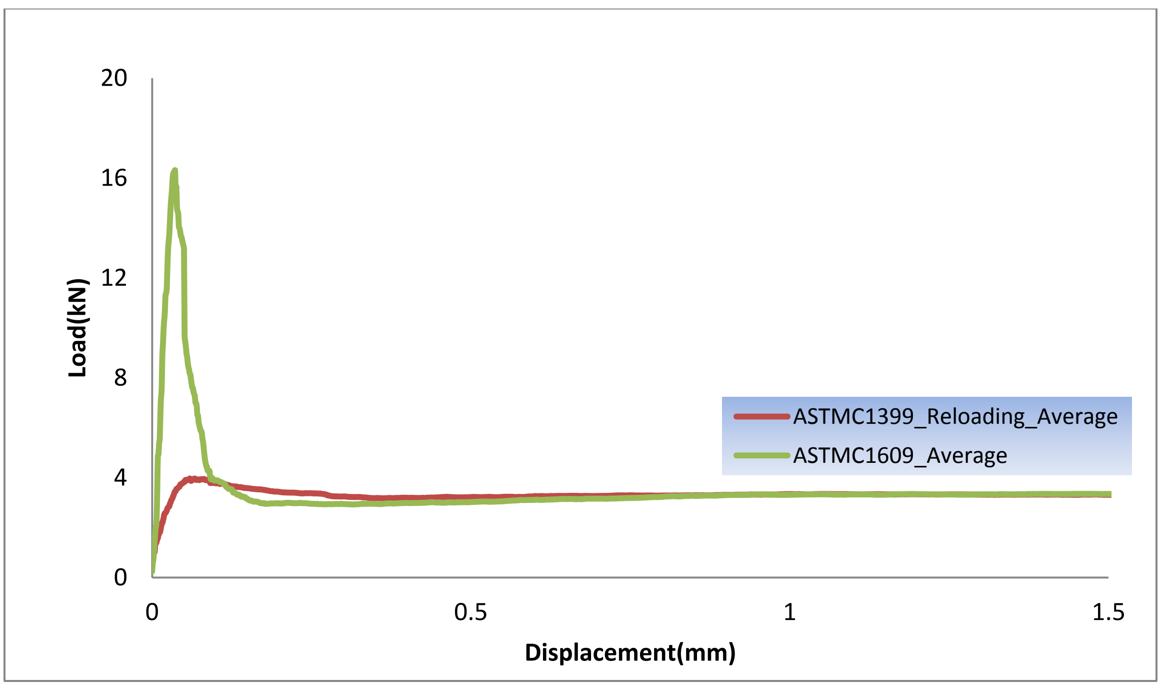

Figure 3-17 Comparison between Load Deflection Curve for ASTM C1399 and ASTM C1609 for Batch 4

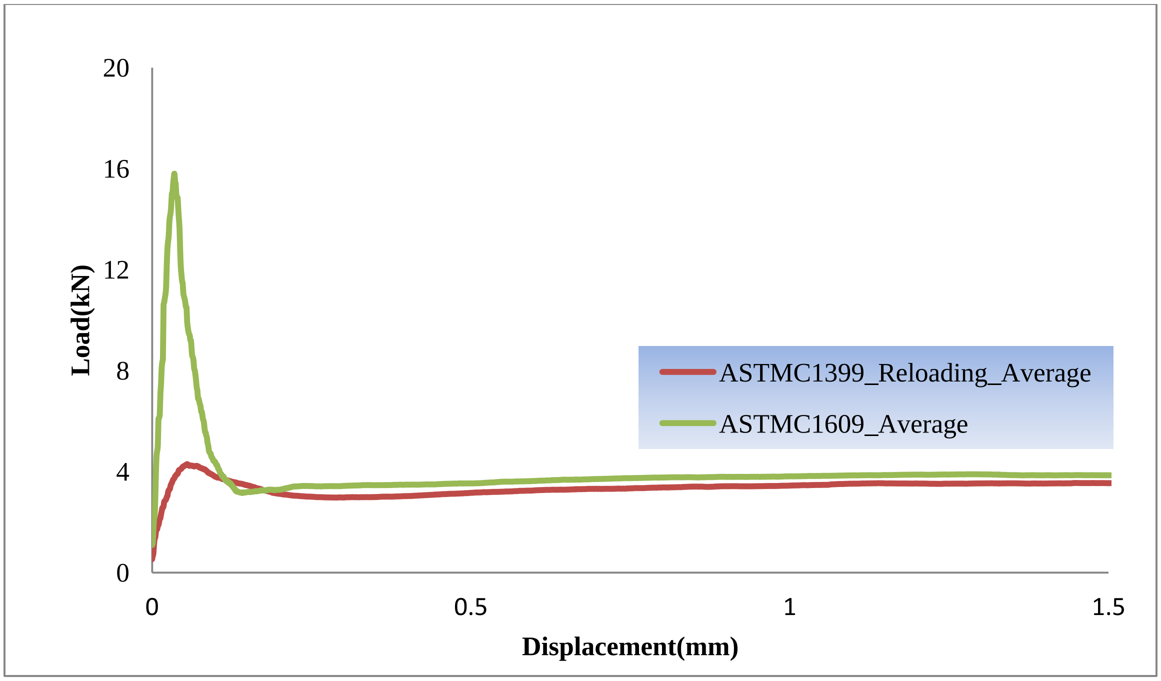

Figure 3-18 Comparison between Load Deflection Curve for ASTM C1399 and ASTM C1609 for Batch 5

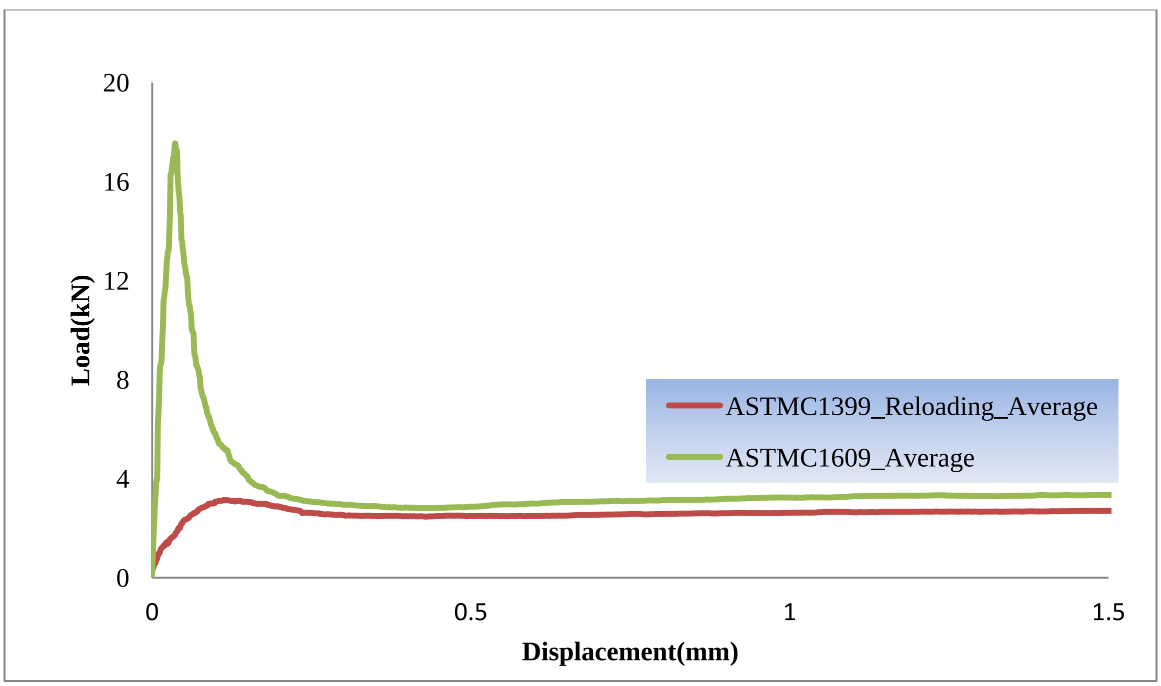

Figure 3-19 Comparison between Load Deflection Curve for ASTM C1399 and ASTM C1609 for Batch 6

ARS and Ri values calculated using ASTM C1399 and ASTM C1609 are summarized in Table 3-4. It is observed that R_i value calculated from the data produced by both of the testing procedure are similar to each other and they are following the same trend. The standard deviation of R_i values are compared and summarized in Figure 3-20 and it is observed that 5 out of 6 batches has lower variability for the proposed method by ASTM C1609. So it can be proposed that for calculating R_i value the existing procedure can be replaced by the following:

$$R_i = \frac{ARS}{R} \qquad [3.4]$$

Where,

Average Residual strength (ARS) is to be calculated from ASTM C1609 procedure at the deflections of 0.50, 0.75, 1.00, and 1.25 mm [0.020, 0.030, 0.040, and 0.050 in.] instead of ASTM C 1399.

$$ARS = \frac{(P_A + P_B + P_C + P_D)L}{4bd^2} \qquad [3.5]$$

$P_A + P_B + P_C + P_D =$ Sum of recorded loads at specified deflections, N [lbf] .

Modulus of rupture,

$$R = f = \frac{PL}{bd^2} \qquad [3.6]$$

P= Peak load calculated from ASTM C1609 curve..

L = span length, mm [in.],

b = average width of beam, mm [in.], and

d = average depth of beam, mm [in.].

Table 3-4 Comparison of Ri Values Calculated using ASTM C1399 and ASTM C1609

		ASTM C1399/CHBDC		**ASTM C1609/CHBDC**		
Batch & Specimen No. **B#-S#**	Average Residual Strength, **ARS** (MPa)	Residual Strength Index, $\mathbf{R_i}$ (%)	Average $\mathbf{R_i}$ (%)	Average Residual Strength, **ARS** (MPa)	Residual Strength Index, $\mathbf{R_i}$ (%)	Average $\mathbf{R_i}$ (%)
B1-S1	1.29	34.74%	28.20%	1.32	35.33%	29.10%
B1-S2	1.29	34.72%		1.18	31.78%	
B1-S3	0.78	20.99%		0.92	24.72%	
B1-S4	0.86	23.21%		0.98	26.29%	
B1-S5	1.02	27.33%		1.02	27.38%	
B2-S1	1.20	29.55%	22.00%	0.89	22.28%	22.54%
B2-S2	0.67	16.55%		0.79	19.82%	
B2-S3	0.93	22.92%		1.02	25.53%	
B2-S4	0.85	20.94%				
B2-S5	0.81	20.06%				
B3-S1	1.26	25.89%	17.58%	0.65	13.27%	18.49%
B3-S2	0.71	14.64%		0.82	16.77%	
B3-S3	0.64	13.19%		1.12	21.65%	
B3-S4	0.69	14.25%		1.20	24.59%	
B3-S5	0.97	19.95%		0.79	16.15%	
B4-S1	0.96	18.70%	19.21%	0.95	18.50%	19.95%
B4-S2	1.25	24.26%		0.99	19.24%	
B4-S3	1.00	19.35%		1.11	21.52%	
B4-S4	0.66	12.91%		1.18	22.85%	
B4-S5	1.07	20.81%		0.91	17.64%	
B5-S1	1.25	25.64%	20.73%	1.19	24.80%	20.96%
B5-S2	1.19	24.48%		0.68	14.19%	
B5-S3	0.64	13.19%		0.89	18.70%	
B5-S4	1.02	20.97%		1.22	25.60%	
B5-S5	0.95	19.40%		1.10	21.50%	
B6-S1	0.77	16.27%	18.09%	0.78	16.44%	18.80%
B6-S2	0.83	17.51%		1.12	22.94%	
B6-S3	1.01	20.50%		0.83	17.01%	

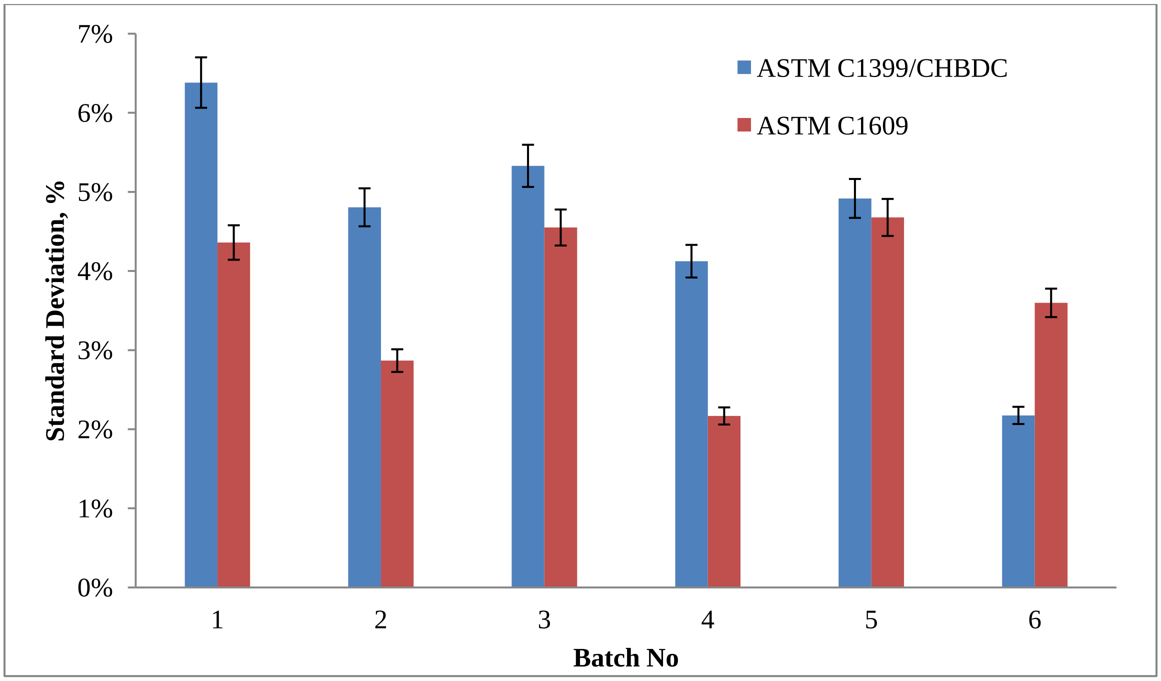

Figure 3-20 Comparison of Standard Deviation of R_i values for CHBDC and Proposed ASTM C1609 Methods.

3.5 Conclusions

A new method to calculate Residual Strength Index (R_i) is proposed by conducting ASTM C1609/C1609M-10 tests (instead of C1399 and C78 as prescribed in CHBDC). Conclusion can be summarized as follows:

1. ASTM C1609 can be effectively used to calculate Residual Strength Index since the average result is only 0.63% higher than the prescribed method.
2. By using ASTM C1609 a significant amount of time can be saved as the method require only half of the number of specimens as of the CHBDC method.

3. The data produced by the proposed method is more consistent than that of CHBDC since the Modulus of Rupture (MOR) is calculated along with the ARS values from same load deflection curve. Specimen to Specimen variability thus minimized.
4. ASTM C1609 is run in a closed-loop arrangement whereas C1399 is performed in an open-loop configuration. The C1609 generated R_i values are therefore reliable.

4 Characterization of Flexural Toughness of FRC

4.1 Outline

It is now widely accepted that the measure of first crack deflection and post-peak response of fiber-reinforced cement-based composite carrying low fiber volume fraction of steel or synthetic fibers are greatly affected by the machine configuration, the test details, human error, etc. For such composites, the post-peak load response obtained from open loop test machine tends to be very unreliable given the sudden release of energy in these machines at the occurrence of the peak load. Only a properly run closed loop displacement control test can capture the true post-peak response.

Although the JSCE method raises many concerns such as test being run in open-loop, choice of excessive deflection of L/150 of span as their end point this method should not be totally ruled out. One of the main advantages of this method is that it counts every single point when absorbed energy is being calculated. A new method is proposed in this study following the JSCE approach which is abbreviated as Flexural Toughness Strength Method (FTSM). The drawbacks of JSCE are eliminated by performing the tests in a closed-loop environment and taking the end deflection to only L/600 span (for a 300mm span specimen). This method is very similar to the Post-Crack Strength Method (PCSM) developed by Banthia and Trottier (Banthia and Trottier 1995) which based on converting the total post-crack energy to strength to characterize FRC. PCS method has some drawbacks while it's considering the first crack in the calculation whereas the proposed method eliminates that part. The outcomes are compared with that of CHBDC and ASTM C1609 and the results shows the FTSM produces lower coefficient of variation.

4.2 Introduction

Although the JSCE method raises many concerns, given the various advantages of this method, it should not be summarily ruled out. The method is simple and FT can be determined easily by using any deflection measuring technique and the determination of first crack is not needed. Moreover it counts every single point during loading and represents the FRC material more reliably. The technique is sometimes criticized for the chosen deflection of span/150, as it is considered excessive for many applications. Since this deflection is purely arbitrary and not based on serviceability considerations, any other suitable limit can be used based on the serviceability requirements and the method can still be used. In this study a better characterization tool “Flexural Toughness Strength Method (FTSM)” is proposed by applying the JSCE approach which produces result with lower Coefficient of Variation compared with the currently available techniques.

4.3 Flexural Toughness Strength Method (FTSM)

Since most of the available test procedures which are primarily based on converting load values to stress values provide a discontinuous measure of toughness particularly in the cracked zones as they only consider loads at some particular points. In procedure followed by ASTM C1399 residual strength is the average of four points load corresponds to deflections at 0.5, 0.75, 1.0, and 1.25 mm. In ASTM C1609 residual strength index, R_i being calculated from strength at L/600 (0.5mm for 300mm span) deflection normalize by peak strength. In practice for specific beam specimen it might have either higher or lower load values at those points and overall misleading results will be produced.

An alternative method called the Flexural Toughness Strength (FTS_m) method is proposed in this study. In this method, the total energy which is the area under load deflection curve up to certain deflection L/m is converted to strength by using equation 4-1.

$$FTS_m = \frac{(E_{total,m})L}{(\frac{L}{m})bh^2} \qquad [4.1]$$

Where $E_{total,m}$ is the total energy value up to a deflection of L/m; L is the span length of specimen; and b and h are the width and height of the specimen, respectively. Also, note that m is a specified divisor of the span length used to calculate a deflection value of interest. FTS_m values are calculated from the total energy illustrated in Figure 4-1.

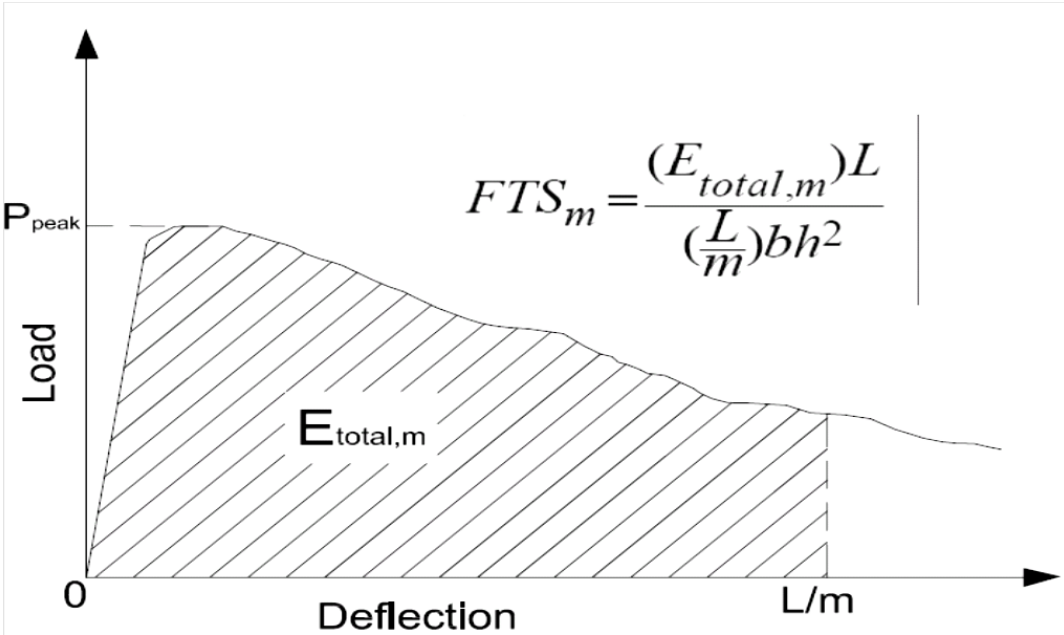

Figure 4-1 Flexural Toughness Strength Analysis

In addition to the FTS_m, another property called the Flexural Strength Index (FSI_m) is also defined as normalized by the modulus of rupture (MOR) values as follows:

$$FSI_m = \frac{FTS_m}{MOR} \times 100 \qquad [4.2]$$

Where FSI_m= Flexural Strength Index at L/m deflection.

FTS_m= Flexural Toughness Strength.

MOR= Modulus of Rupture.

In order to validate the proposed analysis technique based on FTS_m and FSI_m , results from the previously performed ASTM C1609 tests were used in Chapter 3.

4.4 Result and Discussion

The flexural toughness results are represented graphically in Figures 4-2 to 4-6 for batch 1 to 5 respectively according to ASTM C1609 (See Chapter 3 for materials and other details). The average load-deflection curves are compared in Figure 4-7 for Both ASTM C1609 and ASTM C 1399.

These curves are analyzed for the calculation of Flexural Toughness Strength (FTS_m) by following Equation 4-1. Equivalent Flexural Strength Index FSI_m values are calculated using Equation 4-2. Results are compared with ASTM C 1399/CHBDC and ASTM C1609 outcomes and summarized in Table 4-1. For the purpose of the calculation the value of "m" is chosen as 600 which mean the area under load-deflection curve is taken in consideration up to only 0.5mm since L/m=300/600=0.5.

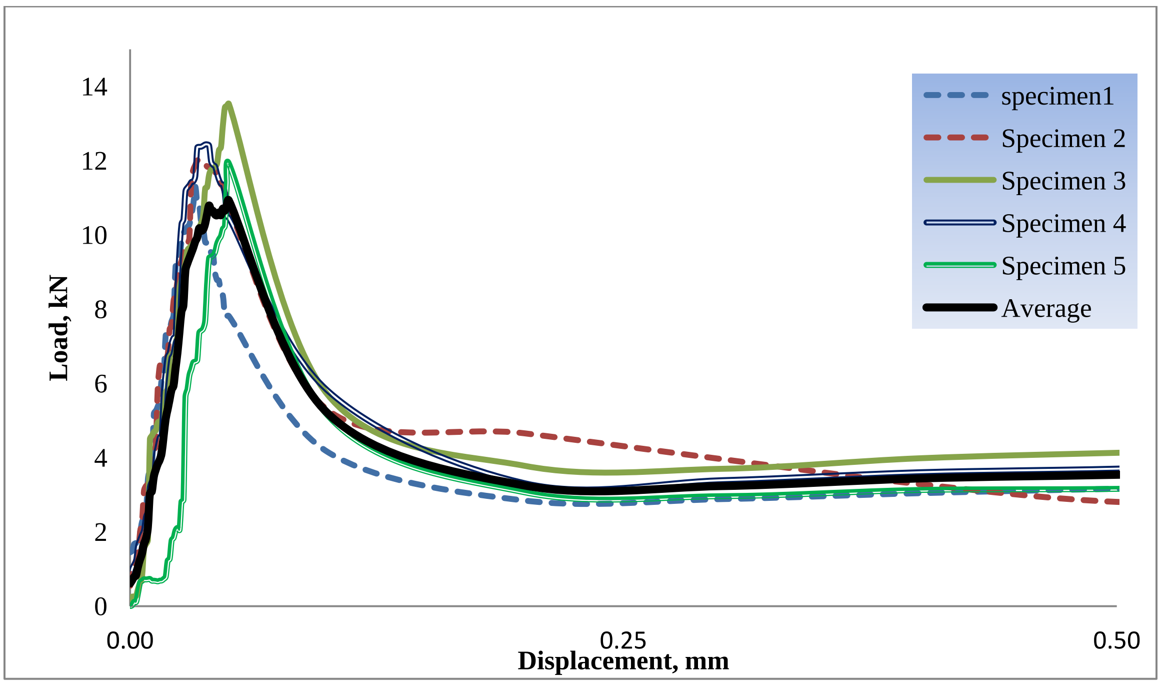

Figure 4-2 Load Deflection Curve for Batch 1

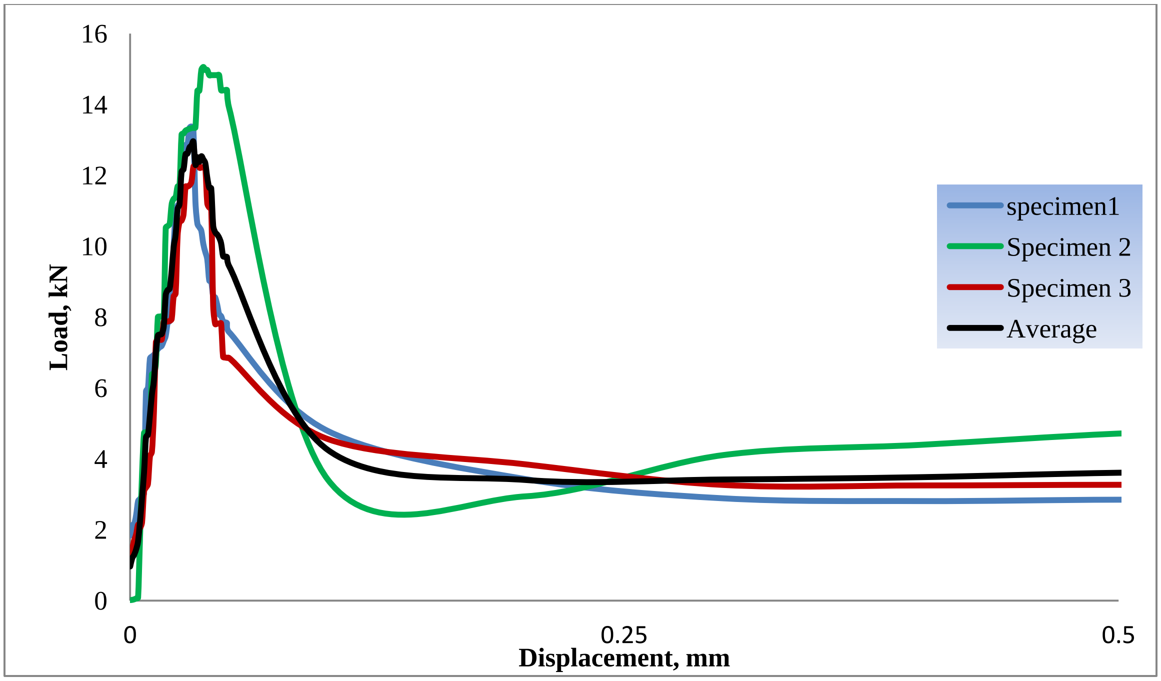

Figure 4-3 Load Deflection Curve for Batch 2

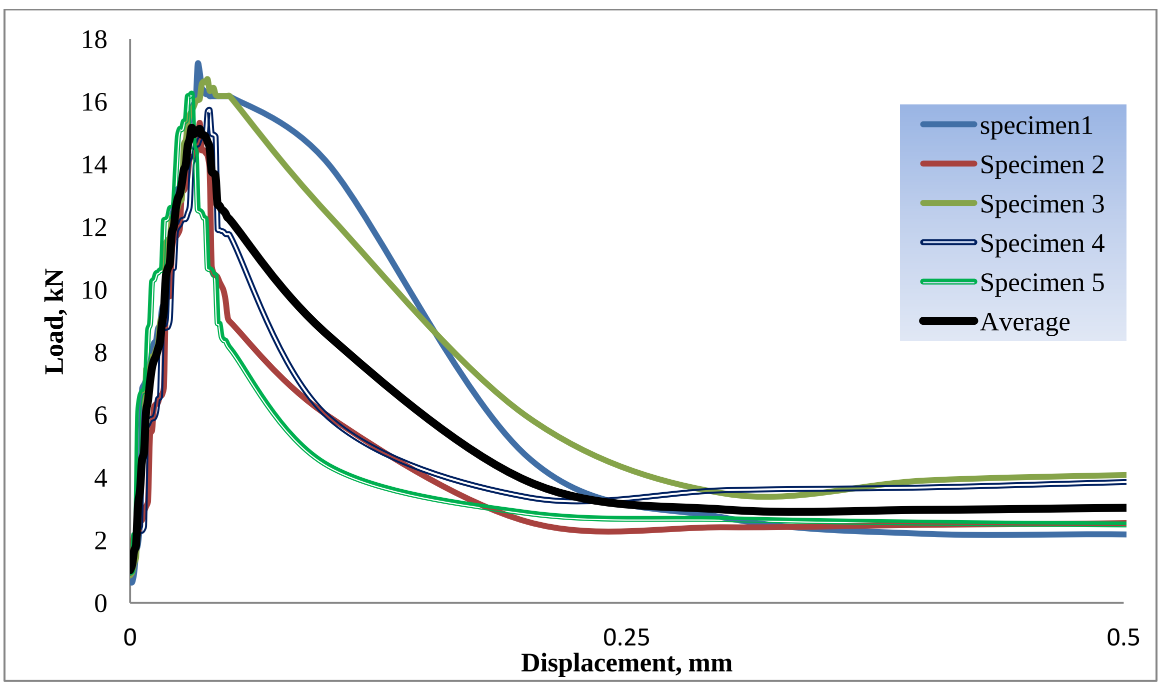

Figure 4-4 Load Deflection Curve for Batch 3

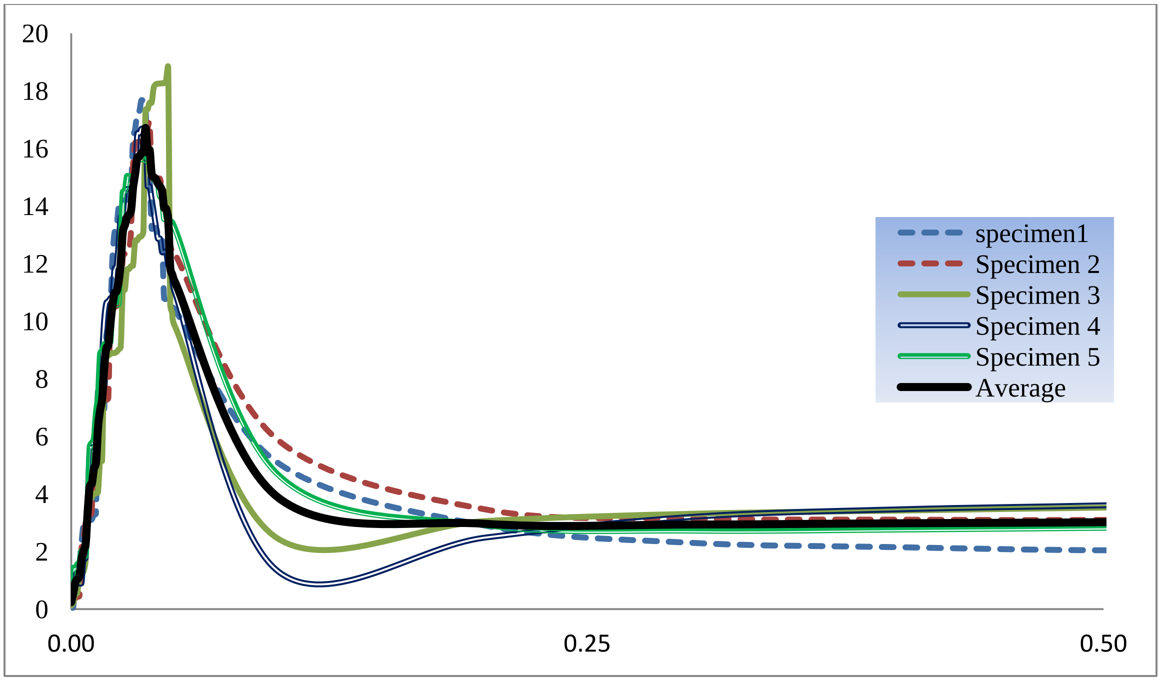

Figure 4-5 Load Deflection Curve for Batch 4

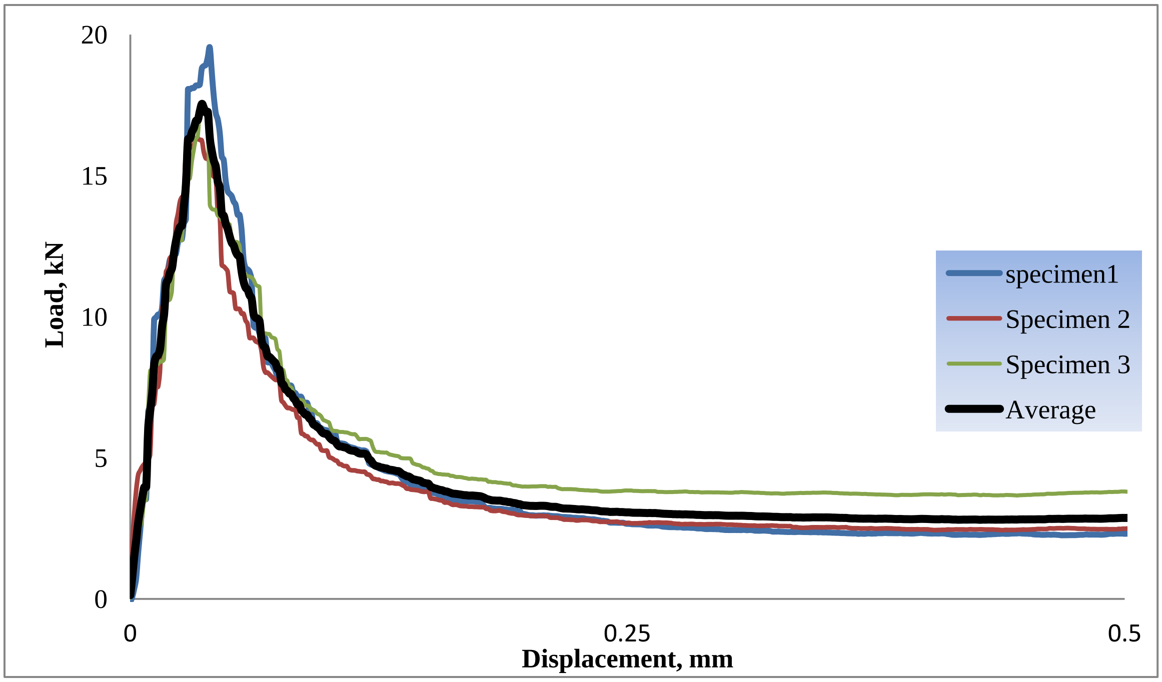

Figure 4-6 Load Deflection Curve for Batch 5

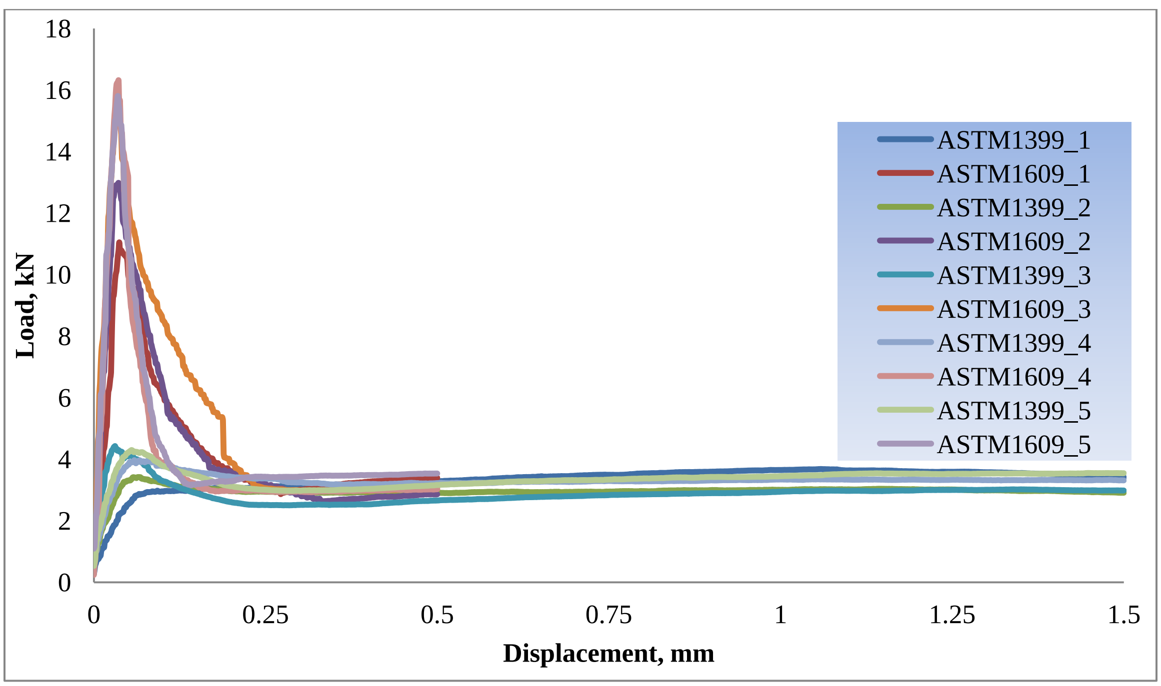

Figure 4-7 Average Load Deflection Curve Tested by ASTM C1399 and ASTM C1609

Table 4-1 Comparison of Strength Index Values Among ASTM C1399, ASTM C1609 and FTSM Method.

	ASTM C1399/CHBDC				ASTM C1609		Flexural Toughness Method (FTM)		
Batch & Specimen No. **B#-S#**	Average Residual Strength, **ARS** (MPa)	Residual Strength Index, **Ri** (%)	Coefficient of Variation of $\mathbf{R_i}$ (%)	Residual Strength $\mathbf{f_{100,0.5}}$ (MPa)	Residual Strength Index, $\mathbf{R_{600}}$ (%)	Coefficient of Variation of $\mathbf{R_{600}}$ (%)	Flexural Toughness Strength, $\mathbf{FTS_{600}}$ (MPa)	Flexural Strength Index, $\mathbf{FSI_{600}}$ (%)	Coefficient of Variation of $\mathbf{FSI_{600}}$ (%)
B1-S1	1.29	34.74%		0.95	29.17%		1.15	35.24%	
B1-S2	1.29	34.72%		0.84	23.38%		1.44	39.95%	
B1-S3	0.78	20.99%	22.63%	1.24	30.53%	10.52%	1.50	36.95%	6.52%
B1-S4	0.86	23.21%		1.11	29.76%		1.38	36.95%	
B1-S5	1.02	27.33%		0.95	26.49%		1.20	33.53%	
B2-S1	1.20	29.55%		0.85	21.29%		1.21	30.16%	
B2-S2	0.93	22.92%	24.57%	1.41	31.29%	19.06%	1.50	33.17%	5.89%
B2-S3	0.85	20.94%		0.98	26.13%		1.26	33.68%	
B3-S1	1.26	25.89%		0.66	12.72%		1.87	36.19%	
B3-S2	0.71	14.64%		0.76	16.62%		1.21	26.32%	
B3-S3	0.64	13.19%	30.31%	1.22	24.42%	28.92%	2.08	41.40%	21.67%
B3-S4	0.69	14.25%		1.16	24.53%		1.50	31.71%	
B3-S5	0.97	19.95%		0.75	15.40%		1.20	24.60%	
B4-S1	1.25	25.64%		0.61	11.59%		1.20	22.55%	
B4-S2	1.19	24.48%		0.93	18.33%		1.43	28.21%	
B4-S3	0.64	13.19%	21.47%	1.06	18.75%	20.85%	1.25	22.16%	11.85%
B4-S4	1.02	20.97%		1.08	21.62%		1.20	23.89%	
B4-S5	0.95	19.40%		0.85	18.01%		1.32	28.08%	
B5-S1	0.57	11.69%		0.69	12.04%		1.38	24.05%	
B5-S2	1.12	22.94%	32.69%	0.69	14.14%	31.75%	1.28	26.20%	12.32%
B5-S3	0.83	17.01%		1.14	21.68%		1.61	30.56%	

It is observed from the Figures and Table 4.1 that flexural strength index numbers calculated from the proposed Flexural Toughness Method are more consistent as they always have lower Coefficient of Variation than the other two methods. This is likely because in the FTSM method every single point on the curve is taken into count since the strength is obtained by converting area under the load deflection to a certain displacement. For a better understanding Batch 2 load deflection curve is reproduced in Figure 4-8.

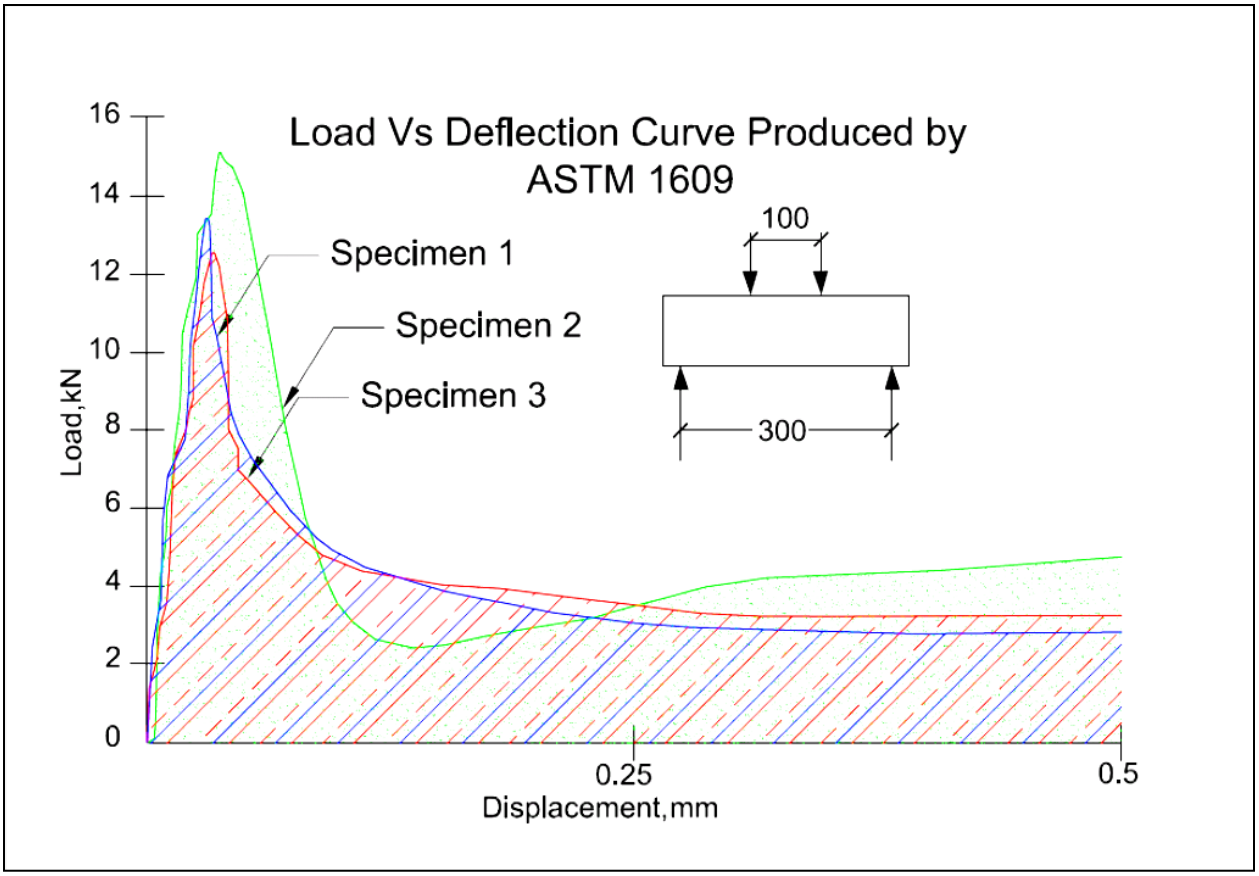

Figure 4-8 Detailed Load Deflection Curve for Batch 2.

From Figure 4-8 it is clear that at displacement 0.5mm the load values are highly variable for each curve which causes high variability in R_i values. On the other hand if the area is considered for calculation, variation smooth out and a lower coefficient of variation occurs. The other positive side of this method is the results aren't dependent of the first crack and the detection of first crack isn't necessary.

4.5 **Conclusion**

An alternate method of analyzing the curves by calculating Flexural Toughness Strength value at L/600 deflections is proposed. The proposed Flexural Toughness Strength Method leads to FRC attributes that are not susceptible to human judgmental errors since it is not dependent on first crack identification and the considered deflection is within the acceptable limit. The characterization of flexural toughness based on the FTSM approach is very simple and is independent of the type of deflection measuring technique. No sophisticated instrumentation is required to determine the toughness factor. The determination of first crack, which is very difficult to identify, is not required. The Flexural Strength Index calculated using this approach has consistently lower coefficient of variation.

In this study the proposed FTSM method outcomes are compared with that of CHBDC and ASTM C1609. There is room for further comparison with other similar method such as JSCE- G552 (former JSCE- SF4) and Post-Crack Strength Method (Banthia and Trottier 1995).

5 Conclusion and Recommendations

5.1 Loading Rate Concerns in ASTM C 1609

For FRC Toughness Characterization initial loading rate described in ASTM C1609 is too high to fail the specimen and might only be efficiently performed for all kind specimens with different fiber volume fraction with the proposed new loading rate described in table 2-4 which is 25 times lower than the existing one. Even though initially it's going to take longer time but sudden failure of specimen can be avoided which occurs due to high initial loading rate and also a consistent result will be produced eventually. In performing ASTM C1609-10, either the test method should clearly indicate it's limitation for brittle materials or the lower initial loading rate proposed at Table 2-4 Series II should be applied for all kind of concrete specimens including both regular strength and high strength concrete.

In this study only one set of reduced loading rate is evaluated. There are room for investigate the optimum loading rate for different specimen based on various volume fraction, age, matrix strength because as being a brittle materials itself concrete behave differently under different condition.

5.2 Comparison between ASTM C1609 and ASTM C1399

5.2.1 Reduction of Experiment Time and Specimen Number

Since only one set of test has to be run instead of two set number of specimens is cut down to half which is more convenient and practical. Extra time will also allow running more specimens testing if required and thus larger sets of data can be achieved. Besides by using ASTM C1609 a significant amount of time can be saved as the methods requires less than half of the time of CHBDC method.

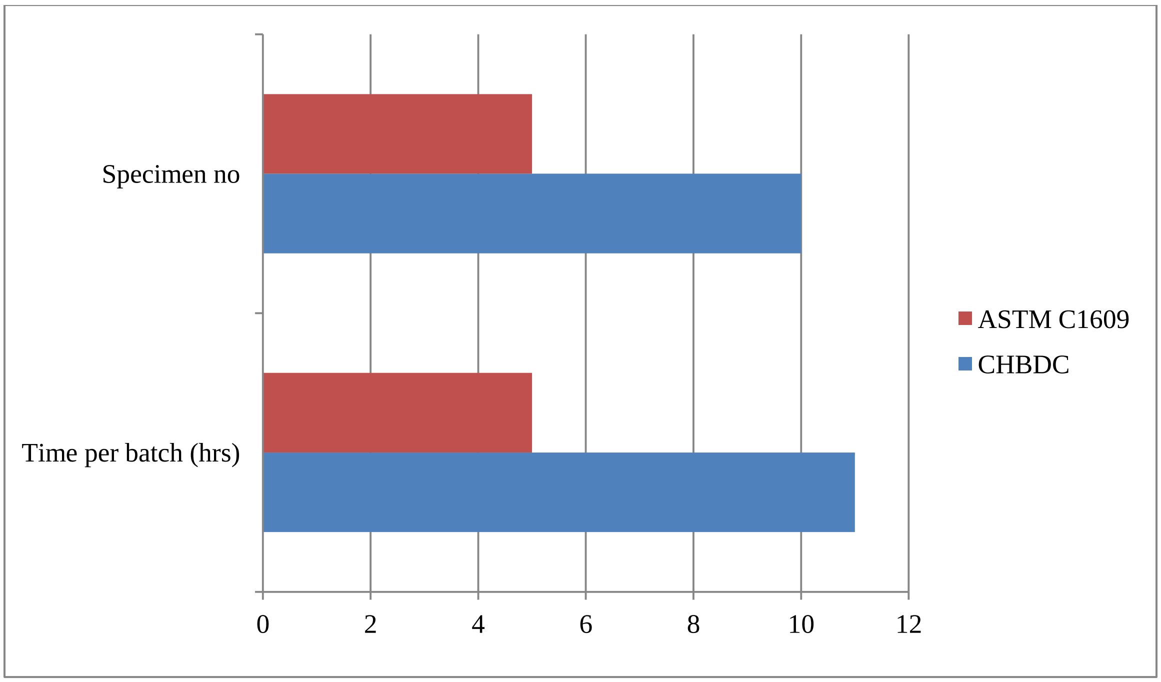

Figure 5-1 Comparison Of Experimental Time Consumption for CHBDC and Proposed ASTM C1609 Method

5.2.2 Reliability of the Data Produced

Specimens at 7th days of casting are more unstable and vulnerable because these are in a continuous process of strength increase. ASTM C1609 is more reliable for all kinds specimens than that of ASTM C1399 as it is a feedback controlled or closed loop system which is very helpful for unstable specimens such as lower age or/and higher strength. Moreover the data produced by proposed method is more consistent than that of CHBDC since the Modulus of Rupture (MOR) is being calculated along with the ARS values from same load deflection curve separately for every specimen where as in CHBDC method MOR is calculated and averaged from ASTM C78 method values. In case of the Specimens from construction site they can be highly variable and data produced can have high Standard Deviation. It also found that nevertheless the specimens are from different construction sites, Ri value produced by ASTM C1609 data having lower standard deviation that that of ASTM C1399 in 4 out of 5 batches (Figure 3-20). So ASTM C1609 can be effectively used to calculate Residual Strength Index with the average result only 0.63% higher than the prescribed one.

Since concrete behaves differently under different condition there is room for further investigation to find the optimum number of specimens based on various volume fraction, age, matrix strength to produce more efficient results with less variability.

5.3 Characterizing FRC by Using Flexural Toughness Strength Method

So it can be concluded that the proposed Flexural Toughness Strength Method (FTSM) leads to FRC attributes that are not susceptible to human judgmental errors since it is not dependent on first crack identification. The considered deflection is within the acceptable limit which is been an issue with JSCE method. The characterization of flexural toughness based on the FTM approach is very simple and is independent of the type of deflection measuring technique. No sophisticated instrumentation is required to determine the toughness factor. The determination of first crack, which is very difficult to identify, is not required in this method. The flexural toughness factor calculated using this approach has consistently lower coefficient of variation (Figure 5-2)

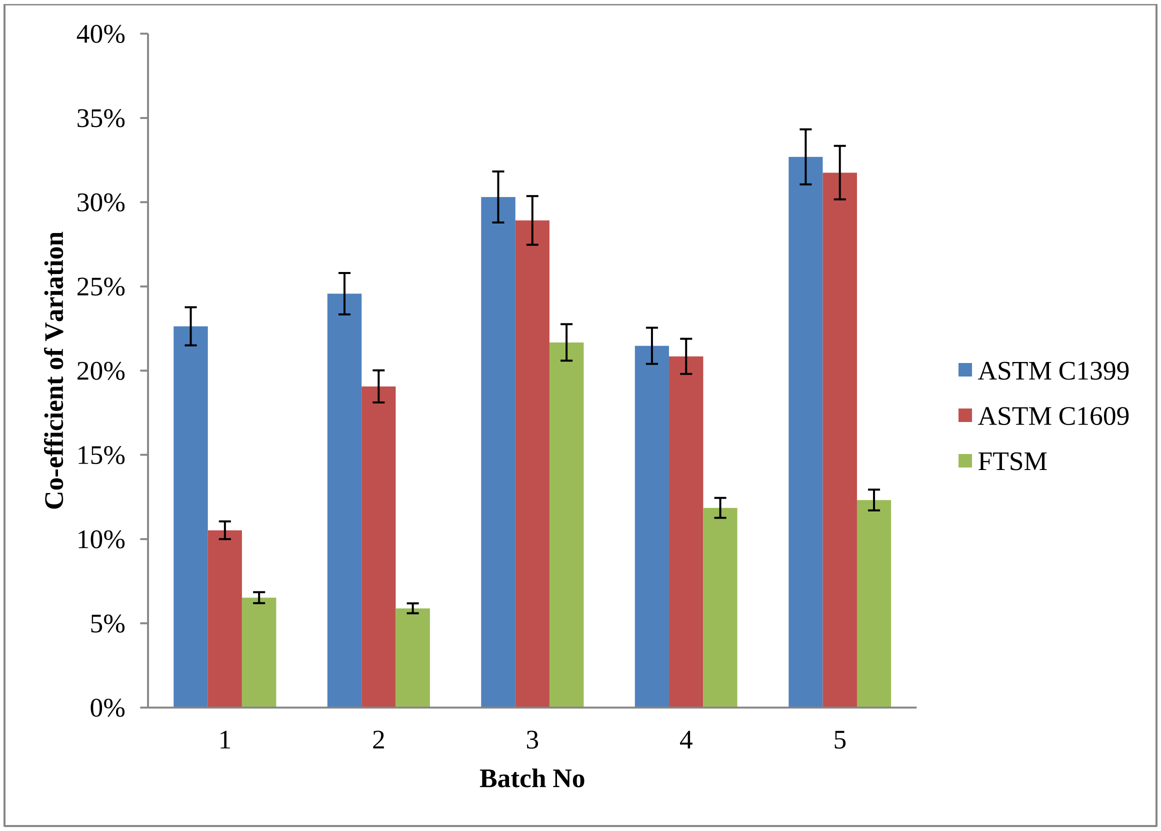

Figure 5-2 Co-efficient of Variation for Different FRC Characterization Techniques.

In this study the proposed FTSM method outcomes are compared with that of CHBDC and ASTM C1609. There is room for further comparison with other similar method such as JSCE- G552 (former JSCE- SF4) and Post-Crack Strength Method (Banthia and Trottier 1995).

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