

Testing of Fiber Reinforced Concrete



EDITORS

D. J. Stevens, *et. al.*

Testing of Fiber Reinforced Concrete



EDITORS

D. J. Stevens, *et. al.*

Full document details: ACI SP-155-95

DISCUSSION of individual papers in this symposium may be submitted in accordance with general requirements of the ACI Publication Policy to ACI headquarters at the address given below. Closing date for submission of discussion is January 1, 1996. All discussion approved by the Technical Activities Committee along with closing remarks by the authors will be published in the May/June 1996 issue of either ACI Structural Journal or ACI Materials Journal depending on the subject emphasis of the individual paper.

The Institute is not responsible for the statements or opinions expressed in its publications. Institute publications are not able to, nor intended to, supplant individual training, responsibility, or judgment of the user, or the supplier, of the information presented.

The papers in this volume have been reviewed under Institute publication procedures by individuals expert in the subject areas of the papers.

Copyright © 1995
AMERICAN CONCRETE INSTITUTE
P.O. Box 19150, Redford Station
Detroit, Michigan 48219

All rights reserved including rights of reproduction and use in any form or by any means, including the making of copies by any photo process, or by any electronic or mechanical device, printed or written or oral, or recording for sound or visual reproduction or for use in any knowledge or retrieval system or device, unless permission in writing is obtained from the copyright proprietors.

Printed in the United States of America

Editorial production Victoria Lunick

Library of Congress catalog card number 95-76808

PREFACE

Fiber Reinforced Concrete (FRC) is a composite material that possesses many mechanical, physical, and chemical properties that are distinct from unreinforced concrete. Recent advances in testing techniques, instrumentation, and interpretation of test results for FRC are the subject of the papers included in this symposium volume. These papers were presented at two technical sessions on the Testing of Fiber Reinforced Concrete, held during the 1995 ACI Spring Convention in Salt Lake City, Utah. These sessions and this special volume were cosponsored by Committees 544, Fiber Reinforced Concrete; and 444, Experimental Analysis of Concrete Structures.

One mechanical property of great practical significance is FRC's superior ability to absorb energy. Many other useful properties such as crack propagation resistance, ductility, impact resistance, fatigue performance, freeze-thaw resistance, and durability are directly or indirectly connected to the composite's energy absorption capacity. Hence, reliable characterization of energy absorption is essential for evaluation of material performance, as well as for design. Static flexural tests have been popularly used to determine toughness values that represent a measure of energy absorption capacity, and interest in toughness characterization has resulted in numerous standards for toughness testing. The first set of papers is dedicated to discussions of recent developments on the subject.

The second session focuses on testing for several other unique properties of FRC including: restrained shrinkage testing, durability testing of FRC with synthetic fibers, impact testing for toughness characterization, early-age tensile testing, pullout resistance of embedded fibers, and impact testing on full scale FRC structures.

The editors would like to thank the authors and presenters for their excellent contributions. All the papers included in this special volume were peer-reviewed, per ACI requirements, by researchers within the two sponsoring committees, as well as by outside experts. We would like to thank the reviewers, as well as the ACI staff who have made this special volume a timely contribution to the state-of-the-art on the testing of Fiber Reinforced Concrete.

Dave Stevens, Nemy Banthia,
Vellore S. Gopalaratnam, and Peter C. Tatnall
Editors

ACI Committee 544
FIBER REINFORCED CONCRETE

James I. Daniel
Chairman

Vellore Gopalaratnam
Secretary

Shuaib H. Ahmad
M. Arockiasamy
P. N. Balaguru
Hiram P. Ball, Jr.
Nemkumar Banthia
Gordon B. Batson
M. Ziad Bayasi
Marvin E. Criswell
Daniel P. Dorfmueller
Marsha Feldstein
Antonio V. Fernandez
Sidney Freedman
David M. Gale
Melvyn A. Galinat
Antonio J. Guerra
Lloyd E. Hackman
C. Geoffrey Hampson
M. Nadim Hassoun

Carol D. Hays
George C. Hoff
Roop L. Jindal
C. D. Johnston
Mark A. Leppert
Clifford MacDonald
Pritpal S. Mangat
Henry N. Marsh, Jr.
Nicholas C. Mitchell
Henry J. Molloy
Dudley R. Morgan
Antoine E. Naaman
Antonio Nanni
Seth L. Pearlman
Max L. Porter
V. Ramakrishnan
Ken B. Rear
D. V. Reddy

Ernest K. Schrader
Morris Schupack
Surendra P. Shah
George Smith
Philip A. Smith
Parviz Soroushian
Jim D. Speakman
David J. Stevens
Narayan Swamy
Peter C. Tatnall
Ben L. Tilsen
George J. Venta
Gary L. Vondran
Methi Wecharatana
Gilbert R. Williamson
Spencer T. Wu
Robert C. Zellers
Ronald F. Zollo

ACI Committee 444
EXPERIMENTAL ANALYSIS FOR CONCRETE STRUCTURES

Kirk A. Marchand
Chairman

John R. Hayes, Jr.
Secretary

Leslie A. Clark
Fernando E. Fagundo
Fikry K. Garas
T. Russell Gentry
Harry G. Harris
Cheng-Tzu Thomas
Hsu
Lorenzo Imperato

Mohsen A. Issa
Moussa A. Issa
Theodor Krauthammer
Robert L. Nigbor
John E. Pearson
Philip C. Perdikaris
Gajanan M. Sabnis

Rajan Sen
Kwok-Nam Shiu
David J. Stevens
Stuart E. Swartz
George V. Teodoru
Richard N. White
David Z. Yankelevsky

CONTENTS

PREFACE	iii
DEFLECTION MEASUREMENT CONSIDERATIONS IN EVALUATING FRC PERFORMANCE USING ASTM C 1018 by C. D. Johnston	1
TOUGHNESS OF FIBER-REINFORCED HIGH-STRENGTH CONCRETE FROM NOTCHED BEAM TESTS by D. Jamet, R. Gettu, V. S. Gopalaratnam, and A. Aguado	23
COMPARATIVE TOUGHNESS TESTING OF FIBER REINFORCED CONCRETE by L. Chen, S. Mindess, D. R. Morgan, S. P. Shah, C. D. Johnston, and M. Pigeon	41
MEASURING TOUGHNESS CHARACTERISTICS OF SFRC — A CRITICAL VIEW OF ASTM C 1018 by D. E. Nemegeer and P. C. Tatnall	77
EXPERIMENTAL R-CURVES FOR ASSESSMENT OF TOUGHENING IN FIBER REINFORCED CEMENTITIOUS COMPOSITES by B. Mobasher, C. Y. Li, and A. Arino	93
TEST METHODS FOR DURABILITY OF POLYMERIC FIBERS IN CONCRETE AND UV LIGHT EXPOSURE by P. Balaguru and K. Slattum	115
RESTRAINED SHRINKAGE TESTS ON FIBER REINFORCED CEMENTITIOUS COMPOSITES by N. Banthia, M. Azzabi, and M. Pigeon	137
DIRECT TENSILE STRENGTH TESTING AT 6 HOURS OF FIBER REINFORCED CONCRETE MORTAR FRACTIONS by P. P. Kraai and G. L. Vondran	153

IMPACT TESTS ON CEMENT-BASED FIBER REINFORCED COMPOSITES	
by N. Banthia, K. Chokri, and J. F. Trottier	171
MEASUREMENT OF THE PULLOUT FORCE AT DIFFERENT RATES OF LOADING	
by A. Pacios and S. P. Shah	189
TESTING APPLIED TO THE EVALUATION OF DAMAGE TO FRC AND OTHER MATERIAL SYSTEMS CAUSED BY LARGE MISSILE IMPACT TO BUILDING ENVELOPES DURING STORM EVENTS	
by C. D. Hayes and R. F. Zollo	217
FIBER REINFORCED CONCRETE TESTING FOR PRACTICAL APPLICATION	
by G. Spadea, R. Cava, D. Gallo, and R. N. Swamy	233
SI (Metric) TABLES	241
INDEX	243

SP 155-1

Deflection Measurement Considerations in Evaluating FRC Performance Using ASTM C 1018

by C. D. Johnston

Synopsis: The issue of how the method of determining midspan deflection in ASTM C1018 toughness tests influences first-crack strength, first-crack deflection, toughness indices and residual strength factors is addressed by comparing results obtained using the method now required in the current standard, which is based on net midspan deflection determined as the nominal midspan deflection minus the average of the deflections measured at the beam supports, with corresponding same-specimen results based on nominal midspan deflection only which was not explicitly excluded in earlier versions of the standard. The problem of dealing with the portion of load-deflection relationship immediately after first crack when it is unstable is also discussed.

The range of test specimens for which comparative data are reported includes a series of third-point-loaded 500x150x150 mm beams with three different steel fibers ranging in length from 18 mm to 63 mm, and a second smaller series of 350x100x100 mm beams that allows for assessment of the effects of beam size and fiber alignment. Fiber contents vary from 20 to 75 kg/m³ (0.25 to 0.94% by volume). Also included is a series of 350x100x100 mm beams with a single type of fibrillated polypropylene fiber of length 38 to 64 mm in amounts of 0.5 to 0.75% by volume.

The results illustrate the extent to which the C1018 parameters I_5 , I_{10} , I_{20} , $R_{5,10}$, and $R_{10,20}$ are effective in distinguishing the performance of the various FRC mixtures in terms of fiber type, geometry and amount. The index I_5 is found to be least effective and a case is made for greater emphasis on use of residual strength factors, especially $R_{10,20}$, when employing the test to specify and control the quality of FRC.

Keywords: Beams (supports); cracking (fracturing); deflection; fiber reinforced concretes; fibers; polypropylene fibers; strength; tests; toughness

2 Johnston

Colin Johnston is an ACI Fellow, a past-president of the Alberta Chapter of ACI and a professor of civil engineering at The University of Calgary. He was awarded ACI's Wason Medal for Materials Research in 1976. He was chairman of ASTM C09.42 on Fiber-Reinforced Concrete from 1980 to 1990 and is a member of ACI Committee 544 on Fiber-Reinforced Concrete.

INTRODUCTION

The issues of exactly how deflection should be measured and the possible effects of different methods of determining deflection on the load-deflection relationship and the toughness parameters derived from it have been the subject of much discussion and some controversy since ASTM C1018(1) was first introduced in 1984.

Since 1984 the Apparatus section of the standard has required deflection-measuring equipment to "accurately determine the net deflection of the specimen under load exclusive of any effects due to seating or twisting of the specimen on its supports, and the Procedure section of the standard has included the imperative statement in mandatory language "Exercise care to ensure that the measured deflections are the net values exclusive of any extraneous effects due to seating or twisting of the specimen on its supports or deformation of the support system". While the intent of these statements was clear enough, the specifics of how it might be met were contained in a non-mandatory note recommending the use of additional deflection-measuring devices at each beam support. The 1984, 1985, 1989 and 1992 editions of the note also acknowledged that the increased number of deflection-measuring devices makes the processing of data to obtain average net deflection more complex and stated that a recommended correction procedure for drawing a tangent to the initially concave upwards portion of the load-deflection curve "allows the net deflection to be obtained reasonably accurately."

In a 1985 state-of-the-art paper on toughness(2), the writer questioned whether deflection measurement at the midspan only, termed nominal deflection, was "reasonably accurate" and showed that it was in fact largely responsible for the wide variation in midspan deflections at first crack reported in various publications available at the time. The paper also illustrated the effect on toughness indices for same-specimen load-deflection relationships obtained using net and nominal midspan deflection measurement (Fig. 1), and acknowledged that nominal deflection measurement at the midspan was a common and imperfect compromise which is simpler and more convenient for routine use than net deflection measurement requiring additional deflection-measuring devices at the supports along with calculations to average the beam support deflections and subtract the average from the midspan deflection.

Unfortunately, the notion conveyed by the note in the 1984 and 1985 editions of ASTM C1018 that nominal deflection measurement was reasonably accurate and good enough for most testing was widely believed until well after 1985. However, in the 1989 edition the standard was modified to delete reference to this notion and

replace it with wording stating that "Nominal deflections based only on midspan measurements may be much larger than corresponding net midspan deflections obtained by subtracting the average of the deflections measured at the two supports from the corresponding nominal deflection at the midspan". Also added was the comment that "Toughness indices based on nominal midspan deflections may be less than the equivalents calculated using net midspan deflection". This non-mandatory language stating the desirability of deflection measurement at the supports as well as the midspan and the possibly significant effect on toughness indices remained unaltered in the 1992 edition of ASTM C1018.

In 1994 the standard was modified to delete all reference to testing based on nominal midspan deflection measurement thus making even more explicit the requirement to determine net deflection and to compute toughness indices and residual strength factors solely on that basis. Detailed descriptions with photographs of two alternative deflection-measuring systems for doing so were added, and a formula for estimating the first-crack deflection in terms of the size of the test specimen and the modulus of the concrete was introduced to help users confirm the validity of deflection measurements.

Like most ASTM Standards, ASTM C1018 has evolved through consensus and compromise between those who advocated the need for net deflection measurement despite increased experimental complexity and those who argued for permitting nominal deflection measurement because of experimental simplicity and practicality. Naturally, fewer laboratories had the more complex equipment needed to determine net midspan deflection, and those that did not have it tended to test specimens using nominal deflection measurement despite failing to comply with the intent of the standard. For example, in an interlaboratory comparison of data(3), supposedly obtained according to ASTM C1018-89, only four of six participants measured net midspan deflection while the two others measured nominal deflection.

Despite much discussion and some controversy over the issue of deflection measurement, there is little published data comparing results obtained using net deflection measurement with those obtained using nominal deflection measurement. This paper makes same-specimen comparisons for a variety of steel and polypropylene fibers at different fiber contents. The results reflect the evolution of ASTM C1018 from 1984 when only toughness indices I_5 and I_{10} and the ratio I_{10}/I_5 were reported, to 1989 when the residual strength factor $R_{5,10}$ became a requirement and the index I_{20} was highlighted instead of I_{30} as a first option along with the residual strength factor $R_{10,20}$. This followed introduction of the concept of residual strength factor in 1986(4).

Since the validity and accuracy of deflection measurements influences the values of toughness indices (Fig. 1) along with the values of residual strength factors derived from them, the issue of how effective the various ASTM C1018 parameters are in distinguishing the performance of FRC's in terms of fiber type, geometry and content is also addressed in the paper. This issue is also a subject of much discussion and some controversy since the standard has been used in specifications to assess FRC performance using parameters ranging from the lowest permissible end-point deflection criterion and toughness index, I_5 , to higher end-point deflection criteria and

4 Johnston

corresponding residual strength factors, such as $R_{10,50}$. The merits of low-end point versus higher end-point deflections and the significance of toughness indices versus residual strength factors can also depend on the response of the testing system. The discussion also deals with the effect of the testing system response rate on the load-deflection function immediately following first crack when there is sometimes a rapid and unstable decrease in load and increase in deflection, particularly for low fiber contents.

EXPERIMENTAL PROGRAM

In the first set of tests reported, the fiber distributor who sponsored the tests in 1987 wished to compare the performance of several types of steel fiber available in Canada in the context primarily of the relatively low fiber contents and low to medium strength concrete matrices associated with industrial floor slab applications. In choosing the specimen size it was recognized that stiff fibers of length 50 to 63 mm would be subject to significant preferential fiber alignment if evaluated using 300x100x100 mm beams and that the ASTM C1018 requirement for thick sections that the minimum specimen dimension be at least 3 times the fiber length would be severely violated. In order to obtain results more representative of thick sections and meet more closely the specimen size/fiber length minimum of 3, the heavier less convenient 450x150x150 mm beam size was employed in the majority of the tests, although additional 300x100x100 mm beams were tested in two cases to get an idea of the effect of specimen size and associated preferential fiber alignment on test results.

In the second set of tests also in 1987 another fiber distributor who wished to evaluate the performance of fibrillated polypropylene fibers of length 38 to 64 mm chose the more economical 350x100x100 mm specimen size recognizing that flexible polypropylene fibers would probably be less subject to the effects of preferential fiber alignment.

Both sponsors selected specific fiber types, lengths and amounts to be evaluated in concretes of specified strength 25 MPa or 30 MPa as shown in Tables 1 and 2, and both wished to have the tests conducted with deflection measured strictly in accordance with intent of C1018-85 "that the measured deflections are the net values exclusive of any extraneous effects due to seating or twisting of the specimen on its supports or deformation of the support system". This was accomplished using the 3-transducer arrangement (Fig. 2), now included in the 1994 edition of ASTM C1018, in which the deflection reproduced on the x-y plotter is the net deflection obtained by subtracting the voltage representing average of the deflections at each beam support from the voltage representing the midspan deflection. However, since the writer was also interested in comparing net deflection with nominal midspan deflection and in the associated comparison of toughness parameters, an additional x-y plotter was added to allow same-specimen plots of load versus net deflection and load versus nominal deflection. Sets of four specimens were tested for each fiber-matrix combination, and the results in Tables 1 and 2 are the mean values for each set calculated in accordance with C1018-89 for toughness indices I_5 , I_{10} and I_{20} and residual strength factors $R_{5,10}$ and $R_{10,20}$. Values based on net deflection

measurement are in bold print while those based on nominal deflection are in normal print.

DISCUSSION OF RESULTS

The discussion is confined to toughness indices (I) and residual strength factors (R). First-crack parameters are included in Table 1 and 2. Generally, first-crack deflections for nominal deflection measurement are 2.8 to 3.2 times larger than corresponding values for net deflection measurement, while first-crack strengths are essentially equal for both methods of deflection measurement.

Effect of Deflection-Measuring Technique

The effect on I and R values of calculating them on the basis of net deflection versus calculating them on the basis of nominal deflection varies with fiber type, geometry and amount as illustrated by the specimen examples in Fig. 3 and 4.

Starting with 450x150x150 mm beams, the three relationships for steel fibers in Fig. 3 illustrate the main possibilities.

The case of gradual stable strain-softening after first crack (Fig. 3a) is perhaps the easiest to explain because of its approximately constant slope. In this case, values of toughness indices denoted by I' are derived from the relationship obtained using nominal midspan deflection (broken line in all figures), and they are less than the indices denoted by I determined in the proper way from the relationship obtained using net midspan deflection (solid line in all figures). Clearly, this results from the fact that the average vertical ordinate for the total area representing the numerator in any index decreases as the end-point deflection moves to the right, making this ordinate for any end point on the broken line less than the ordinate for the corresponding end point on the solid line, while the horizontal abscissae for the numerator representing total areas and the denominator representing first-crack areas remain in the same proportion for both broken and solid lines. Thus, I_5' , I_{10}' and I_{20}' are less respectively than I_5 , I_{10} and I_{20} . Likewise, residual strength factors, which are in fact the average ordinate between consecutive end-points divided by the first-crack ordinate, decrease as the end points move to the right, so $R'_{5,10}$ and $R'_{10,20}$ are less respectively than $R_{5,10}$ and $R_{10,20}$.

Considering the special case of elastic-plastic behaviour where both broken and solid lines are horizontal from first crack, it should readily be understood that I' and R' values will be the same as I and R values because the vertical ordinate remains the same for all parameters. Extending the analysis to the case of gradual stable strain-hardening where the ordinate actually increases as the end-point deflection moves to the right (Fig. 3b), it is expected that I' and R' values will exceed I and R values. This situation is relatively uncommon, but is known to happen for certain type-amount combinations of hooked-end fibers as in Fig. 3b.

The third case is unstable strain-softening immediately after first crack followed by stable nearly plastic behaviour thereafter (Fig. 3c). In these cases the average vertical ordinate for total area is again less for the broken line relationship

6 Johnston

than for the solid line one, so I' values are less than I values. However, when the portions of the relationships between consecutive end points used to calculate R' and R values are horizontal (plastic) R' and R values may be nearly equal, for example $R'_{5,10}$ and $R_{5,10}$ in Fig. 3c.

Similar situations arise with the 300x100x100 mm beams. For example, with steel fibers (Fig. 4a) the extensive plastic portion of both relationships following first crack accounts first for $R_{10,20}$ almost equal to $R_{5,10}$ and second for $R'_{5,10}$ almost equal to $R_{5,10}$. Polypropylene fibers (Fig. 4b) can also give rise to the situation where I' values are much less I values while R' and R values are not substantially different.

Effect of Unstable Strain-Softening Immediately After First Crack

Some fiber type-amount combinations, particularly those using low fiber contents, are associated with a rapid decrease in load and increase in deflection immediately after first crack which occurs so quickly that the response rate of the load and deflection-recording system may not be fast enough to reflect what is really happening. The relationships in Fig. 3c and 4b are examples typically obtained under open loop control conditions.

Part of Fig. 3c enlarged to highlight the unstable region immediately following first crack is shown as Fig. 5a. The unstable region in question is AY in general, although a portion of it, AX, appears stable initially from the clearly defined track of the pen on the plotter, while the portion XY is poorly defined with only a faint linear pen track. In Fig. 5b the transition corresponding to the change in slope (specimen stiffness) at X is not discernible and the whole of AY is likely unstable. In the limiting case of concrete without fibers, the beam breaks suddenly at A and has no residual strength thereafter, so the load drops instantaneously to zero before deflection can increase, AZB in Fig. 5, even though the plotter records a line somewhere between AXY and AZB. The same behavior applies at very low fiber contents. With sufficient fibers there is a deflection increase as the load drops from first crack to the residual value that can be sustained stably over a period of time, as depicted by the portion of the load-deflection relationship to the right of Y. Dealing with uncertainty regarding the position of AY which could be anywhere between AXY and AZY is the problem. Obviously, the same issue arises with regard to A'Y' (Fig. 5) for nominal deflection measurement.

Toughness indices calculated in the normal way using AXY will obviously decrease if recalculated using AZY. In Fig. 5a the effect will be greatest for I_5 , derived from the area AXYCDB, and relatively less for I_{10} , derived from AXYEFB in which the portion CEFD is unaffected by the position of AY, and of course less still for I_{20} . Perhaps the most important point is that $R_{5,10}$, which is based only on the area CEFD, is unaffected by any uncertainty in the position of AY because C is the right of Y, and likewise $R_{10,20}$ (CC63 fibres in Table 3). This applies also to Fig. 4a. However, in Fig. 5b the effect of uncertainty regarding the position of AY extends to $R_{5,10}$ because the area CEFD is affected since C is left of Y. Nevertheless, $R_{10,20}$ which is based on the area to the right of EF is unaffected (EE18

fibres in Table 3). This applies also to Fig. 3a, 3b and 4b, but Fig. 5b represents the worst case observed for the data set in terms of C being furthest to the left of Y.

Since the position of the unstable portion of the load-deflection relationship AXY is probably influenced by both the response rate of the data-recording system and the stiffness of the testing frame and that of the specimen, an approach to performance assessment that eliminates this uncertainty is desirable. Thus, it is imperative that the widely practised tendency to highlight I_5 , which is most severely affected, and not consider parameters which are less affected, such as I_{20} , or usually not affected at all, such as $R_{10,20}$, must change. Reaching the conclusion in the 1991 interlaboratory study (3) that "ASTM C1018 toughness indices are observed to be relatively insensitive to fiber type, volume fraction and specimen size", while highlighting I_5 and ignoring R values in the published analysis, despite the fact that reporting of $R_{5,10}$ was mandatory in the 1989 edition of the standard and $R_{10,20}$ was identified as optional, presented an incomplete impression of the effectiveness of ASTM C1018 for distinguishing performance in terms of fiber parameters.

Effectiveness of C1018 Parameters in Distinguishing FRC Performance

While the five single-specimen examples in Fig. 3 and 4 permit differences in performance to be distinguished in terms of the appearance of the load-deflection relationships and the numerical values of parameters such as I_{20} (range 10.2 to 17.3) and $R_{10,20}$ (range 38 to 87), the effects of variables like fiber geometry, amount and specimen size are best distinguished using the mean values from Tables 1 and 2 plotted graphically as in Fig. 6 for toughness indices and in Fig. 7 for residual strength factors. To make the indices graphically comparable, the scales for I_5 , I_{10} , and I_{20} are chosen to correspond to the values of 5, 10 and 20 corresponding to elastic-plastic or yield-like behavior which is the reference level against which actual performance is usually compared (Appendix XI of ASTM C1018). Accordingly, the I_5 scale is twice as large as the I_{10} scale and four times as large as the I_{20} scale. Naturally, $R_{5,10}$ and $R_{10,20}$ are plotted to the same scale as they both have the same range of 0 to 100.

Effect of Fiber Geometry and Amount--Three varieties of steel fibers designated CC63, EE18 and HE50 are compared in terms of fiber content for 450x150x150 mm beams (Fig. 6 and 7). A fourth, CW60, is included at one fiber content. Both the solid line trends (based on net deflection) and the broken line trends (based on nominal deflection) are definitive in illustrating the importance of fiber content, but the values in the latter case are lower except for some HE50 concretes consistent with the reasoning given earlier in discussing Fig. 3 and 4. Subsequent discussion is limited to the solid line trends which represent proper accurate measurement of deflection. However, they are subject to the uncertainty associated with the unstable AXY (Fig. 5) portion of the load-deflection relationship, especially for low fiber contents, the effect of which is to decrease some I values, particularly I_5 , below the values plotted.

Since the graphs for toughness indices (Fig. 6) are scaled vertically to make the I_5 , I_{10} , and I_{20} values graphically comparable, the steeper the slope the better the distinction of performance in terms of fiber content. Clearly, I_5 is least effective and

8 Johnston

I_{20} is the most effective with I_{10} almost as effective as I_{20} . In terms of residual strength (Fig. 7), both $R_{5,10}$ and $R_{10,20}$ appear effective in making the distinction by fiber content. However, $R_{10,20}$ is in fact better because the uncertainty associated with the unstable AXY portion (Fig. 5) of the load-deflection relationship which influences some $R_{5,10}$ values at low fiber contents is without exception eliminated for $R_{10,20}$.

The importance of fiber geometry for steel fibers reflects the influence of aspect ratio and improvements to pullout resistance by use of crimping (CC and CW) and hooked ends (HE) or enlarged ends (EE). For example, at 0.5% by volume or 40 kg/m³ of fibers $R_{10,20}$ values are about 88 for HE 50, 82 for CW 60, 62 for CC 63 and 42 for EE 18 over a range of aspect ratio of 100 for HE 50 fibers to 38 for EE 18 fibers.

Concrete matrix strength within the limited 25 to 30 MPa range examined has little influence on I or R values.

Effect of Fiber Type and Amount--Steel and polypropylene fibers are compared in terms of fiber content for 300x100x100 mm beams (Fig. 8 and 9 left). Differences between the solid line trends (based on net deflection) and the broken line trends (based on nominal deflection) are similar to those in Fig. 6 and 7, and are consistent with the reasoning given earlier in discussing Fig. 3 and 4. Only the solid line trends are discussed further.

Once again, the manner in which the graphs in Fig. 8 are scaled means that slope is an indicator of the effectiveness of each toughness index in distinguishing performance. Clearly, I_{20} and I_{10} are again more effective for this purpose than I_5 (Fig. 8 left), just as in Fig. 6. For residual strength, $R_{5,10}$ and $R_{10,20}$ are both effective (Fig. 9 left), and illustrate the expected influence of fiber content and in the case of the polypropylene fibers the effect of length or aspect ratio. For example, at 0.5% by volume of fibers the $R_{10,20}$ values are about 80 for the CC 63 steel fiber, 37 to 39 for the FP 38 and FP 64 polypropylene, and, by interpolation on Fig. 7, for the EE 18 fiber about 42 plus a small amount attributable to specimen size effect (see next section).

Effect of Specimen Size--From the limited data available, a significant increase in both I and R values is associated with reducing beam size from 450x150x150 mm to 300x100x100 mm for the long stiff CC 63 steel fibers when preferential fiber alignment by the mold surfaces is significant (Fig. 8 and 9, right). This is to be expected since the ratio of specimen cross-section to fiber length is 1.6 compared with the minimum of 3 required for thick sections in ASTM C1018. For the shorter EE 18 fiber, where the corresponding ratio is 5.3, the fiber alignment effect is probably minimal as suggested by the comparison at 75 kg/m³. This is consistent with the results of an earlier study (5) to examine the effect of preferential fiber alignment on test results using fibers of length 76 mm and 25 mm in molded and sawn specimens of 100 mm cross-section.

Precision of Results

Within-batch coefficients of variation for each set of four specimens are given in Tables 4 and 5 for the two different specimen sizes.

While there is no conclusive link between the individual values in terms of fiber type, geometry or amount, the highest values are associated mainly with low fiber contents of CC63 steel fibers. These are the largest in individual size. At low fiber contents this means the lowest number of fibers per unit volume of concrete, which may contribute to more marked nonuniformity in the fiber distribution and a more variable FRC than for the other fibers.

The most meaningful numbers are the mean values which indicate the level of precision that should be expected on average when testing multiple sets of specimens. In this regard, there are no major differences for the two specimen sizes, which if combined total 24 sets, and it is clear that the parameters which it has been argued should be highlighted more in future testing, that is I_{20} , $R_{5,10}$, and $R_{10,20}$, can be evaluated with reasonable precision. The highest mean within-batch coefficient of variation is about 13% for $R_{10,20}$ and 19 of 24 values are less than 18%. It must be recognized that the higher the end-point deflection the greater the variability of the results, but in view of the arguments previously discussed it is pointless to rely on I_5 determinations simply because of better precision.

The mean values reported in Tables 4 and 5 compare closely with values for another recent data set (6) analyzed in the same way for 300x100x100 mm beams tested at various ages and machine stroke rates.

CONCLUSIONS

1. Effectively utilizing ASTM C1018 to evaluate the performance of FRC and distinguish the importance of the material type, geometry and amount of the fibers depends on recognition of the following limitations:
 - (i) Toughness indices and residual strength factors derived using nominal deflection measurement are usually less, often considerably less, than values derived using net deflection measurement as required by the standard. However, there are exceptions where nominal deflection measurement may produce values equal to or slightly more than values obtained according to the standard.
 - (ii) Some fiber-matrix combinations, particularly with low fiber contents, exhibit rapid load decrease with deflection increase after first crack that is unstable and may not be detected accurately by the load and deflection-recording system. The uncertainty associated with this unstable portion of the relationship affects the toughness index I_5 most severely, and its effect lessens with increasing end-point deflection, and is usually quite small for I_{20} and higher deflection indices. Evaluating performance in terms of residual strength factors can

10 Johnston

eliminate the effect of this uncertainty entirely when using $R_{10,20}$ and the effect on $R_{5,10}$ is often quite minimal. It exists when C or E are to the left of Y in Fig. 5 and is eliminated when C and E are to the right of Y in Fig. 5.

- (iii) The use of the preferred 300x100x100 mm beam with long rigid fibers like steel tends to produce toughness indices and residual strength factors greater than for otherwise comparable 450x150x150 mm beams.
2. The data set discussed in the paper which is based on 22 fiber-matrix combinations and 88 tests demonstrates that some ASTM C1018 parameters are more effective than others for distinguishing the effects of fiber type, geometry and amount on the performance. The index I_5 is certainly least effective. Both I_{10} and I_{20} (which is a required test parameter in the 1994 standard) are much more effective.

The index I_{20} is least influenced by uncertainty regarding the position of the portion of the load-deflection immediately following first crack in cases where rapid unstable behaviour occurs. The fact that the effect of this uncertainty can be minimized or eliminated entirely by calculating residual strength factors, coupled with the fact that these factors are more easily understood than toughness indices and have more potential for direct application in strength-based design (7), is an argument for less emphasis on I values and more emphasis on R values, particularly $R_{10,20}$, in using the test method to specify and control the quality of FRC. Certainly, it is imperative that the widely practised tendency to highlight I_5 and ignore $R_{5,10}$ and $R_{10,20}$ must change.

3. The conclusion that parameters such as I_{20} and $R_{10,20}$, and to a slightly lesser degree I_{10} and $R_{5,10}$, are effective in distinguishing performance differences by fiber type, geometry and amount is not unique to these two series of specimens or to the particular equipment used. Similar results were reported (8) in the same format as Fig. 6, 7, 8 and 9 for a series of 108 tests on 18 steel fiber-matrix combinations performance on different equipment using 750x150x100 mm beams and supervised by the writer in Stockholm in 1989. However, both investigations employed the 3-transducer deflection-measuring system (Fig. 2) rather than the rectangular jig arrangement identified as an alternative in the 1994 edition of ASTM C1018.

ACKNOWLEDGEMENT

Financial support provided by Domecrete Canada Ltd., Elsro Construction Products, Forta Corporation, and the Canadian Government IRAP Program is gratefully acknowledged, along with the efforts of Geoff Hampson, Brian Richardson and Bob Zellers in planning the program and providing fibers.

REFERENCES

1. ASTM C1018, Standard Test Method for "Flexural Toughness and First-Crack Strength of Fiber-Reinforced Concrete (Using Beam with Third-Point Loading)", ASTM Standards Vol. 04.02, Concrete and Aggregates, 1992.
2. Johnston, C.D., "Toughness of Steel Fiber-Reinforced Concrete", Proceedings of a U.S.-Sweden Joint Seminar, Swedish Cement and Concrete Institute, Ed. S.P. Shah and A. Skarendahl, Elsevier Applied Science, London, 1986, pp. 333-360.
3. Gopalaratnam, V.S., et. al., "Fracture Toughness of Fiber Reinforced Concrete", ACI Materials Journal, Vol. 88, July-August 1991, pp. 339-353 and Discussion by C.D. Johnston, May-June 1992, pp. 304-309.
4. Johnston, C.D. and Gray, R.J., "Flexural Toughness and First-Crack Strength of Fiber-Reinforced Concrete Using ASTM Standard C1018", Proceedings of Third International RILEM Symposium of Developments in Fibre Reinforced Cement and Concrete, Ed. R.N. Swamy, University of Sheffield, 1986, Paper 5.1.
5. Johnston, C.D., "Effects on Flexural Performance of Sawing Plain Concrete and of Sawing and Other Methods of Altering the Degree of Fiber Alignment in Fiber-Reinforced Concrete", ASTM Cement, Concrete and Aggregates, CCAGDP, Vol. 11, No. 1, Summer 1989, pp. 23-29.
6. Johnston, C.D., "Effects of Testing Rate and Age on ASTM C1018 Toughness Parameters and Their Precision for Steel Fiber-Reinforced Concrete", ASTM Cement, Concrete and Aggregates, CCAGDP, Vol. 15, No. 1, Summer 1993, pp. 50-58.
7. Johnston, C.D., "Fiber-Reinforced Concrete" Chapter 51 of Significance of Tests and Properties of Concrete", ASTM STP 169C, 1994, pp. 547-561.
8. Johnston, C.D. and Skarendahl, A., "Comparative Flexural Performance Evaluation of Steel Fiber-Reinforced Concrete According to ASTM C1018 Shows Importance of Fiber Parameters", RILEM Materials and Structures, Vol. 25, No. 148, May 1992, pp. 191-200.

12 Johnston

TABLE 1 — TEST RESULTS FOR 450 x 150 x 150 mm BEAMS (VALUES BASED ON NET DEFLECTION SHOWN BOLD)

Matrix Type ^a	Fiber Code ^b	Amount kg/m ³	Vol %	First-Crack		Toughness Indices			Residual Strength Factors	
				σ_{fc}	δ_{fc}	I_5	I_{10}	I_{20}	$R_{5,10}$	$R_{10,20}$
25	-	0	0	4.45	0.052 0.168					
25	CC63	25	0.32	4.53	0.061 0.179	4.11 3.64	6.27 5.67	10.41 8.38	43.2 40.6	41.4 27.1
25	CC63	40	0.51	4.68	0.060 0.181	4.38 3.95	7.67 6.91	13.90 11.16	65.8 59.2	62.3 42.5
25	CC63	55	0.70	4.88	0.060 0.193	4.66 4.29	8.63 7.80	15.95 14.09	79.4 70.2	73.2 62.9
25	EE18	25	0.32	4.44	0.060 0.174	4.40 3.61	6.77 5.13	10.08 7.01	47.4 30.4	33.1 18.8
25	EE18	50	0.64	4.80	0.064 0.186	4.54 3.71	7.66 5.87	12.45 8.64	62.4 43.2	47.9 27.7
25	EE18	62.5	0.80	5.15	0.060 0.189	4.60 3.97	8.12 6.94	14.54 11.16	70.4 59.4	64.2 42.2
25	EE18	75	0.96	5.11	0.059 0.193	4.64 4.10	8.39 7.34	15.39 12.36	75.0 64.8	70.0 50.2
30	EE18	62.5	0.80	5.33	0.062 0.193	4.64 3.95	8.24 6.37	14.27 9.07	72.0 48.4	60.3 27.0
30	EE18	75	0.96	5.62	0.065 0.216	4.64 4.07	8.33 7.03	15.07 10.79	73.8 59.2	67.4 37.6
30	CC63	40	0.51	5.01	0.052 0.184	4.68 3.94	8.30 7.09	14.76 11.52	72.4 63.0	64.6 44.3
30	CC63	55	0.70	5.16	0.059 0.189	4.67 4.41	8.85 8.32	16.97 14.36	83.6 78.2	81.2 60.4
25	HE50	20	0.25	4.68	0.054 0.175	4.44 3.56	7.17 5.81	11.27 10.38	54.6 45.0	41.0 45.7
25	HE50	33	0.42	4.47	0.054 0.165	4.49 3.95	7.73 7.66	14.70 13.95	64.8 74.2	69.7 62.9
30	HE50	40	0.51	4.86	0.053 0.175	4.71 4.38	8.71 8.90	17.50 18.13	80.0 90.4	87.9 92.3
30	CW60	40	0.51	5.17	0.053 0.197	4.63 4.31	8.54 8.48	16.78 15.88	78.2 83.4	82.4 74.0
30	-	0	0	5.17	0.058 0.186					

^a - Specified Compressive Strength in MPa^b - CC is Crimped Crescent-Shaped Steel, CW is Crimped Wire, HE is Hooked-End Wire, EE is Enlarged-End Slit Sheet Metal. Numbers are Nominal Length in mm.

TABLE 2 — TEST RESULTS FOR 300 x 100 x 100 mm BEAMS (VALUES BASED ON NET DEFLECTION SHOWN IN BOLD)

Matrix Type ^a	Fiber Code ^b	Amount kg/m ³	Vol %	First-Crack		Toughness Indices			Residual Strength Factors	
				σ_{fc}	δ_{fc}	I_5	I_{10}	I_{20}	$R_{5,10}$	$R_{10,20}$
25	CC63	25	0.32	4.81	0.039 0.119	4.45 3.66	7.40 6.28	12.80 10.44	59.0 52.4	54.0 41.6
25	CC63	40	0.51	4.96	0.040 0.114	4.68 4.36	8.68 8.33	16.69 15.51	80.0 79.4	80.1 71.8
30	EE18	75	0.96	5.86	0.044 0.134	4.76 4.12	8.61 6.97	15.41 10.05	77.0 57.0	68.0 30.8
30	FP38	4.55	0.50	5.58	0.044 0.123	4.12 3.45	6.22 5.35	9.92 9.21	42.0 38.0	37.0 38.6
30	FP38	5.91	0.65	5.86	0.047 0.133	4.12 3.43	6.29 5.44	10.19 9.30	43.4 40.2	39.0 38.6
30	FP38	6.83	0.75	5.81	0.050 0.135	4.19 3.51	6.84 6.02	11.76 10.84	53.0 50.2	49.2 48.2
30	FP57	5.91	0.65	5.53	0.045 0.126	4.30 3.45	6.95 5.90	11.66 11.21	53.0 49.0	47.1 53.1
30	FP64	5.91	0.65	5.69	0.046 0.127	4.34 3.51	6.91 5.90	11.51 10.97	51.4 47.8	46.0 50.7
30	FP64	4.55	0.50	5.93	0.049 0.130	4.08 3.41	6.24 5.39	10.10 9.29	43.2 39.6	38.6 39.0
30	-	0	0	5.71	0.047 0.124					

^a - Specified Compressive Strength in MPa^b - CC is Crimped Crescent-Shaped Steel,
EE is Enlarged-End Slit Sheet Metal,

FP is Fibrillated Polypropylene. Numbers are Nominal Length in mm.

TABLE 3 — COMPARATIVE I AND R VALUES CALCULATED FROM AREAS BOUNDED BY AXI AND AZI (Fig. 5)

Fiber Code	Amount kg/m ³	Boundary	I_5	I_{10}	I_{20}	$R_{5,10}$	$R_{10,20}$
CC63	25	AXY ^a	4.11	6.27	10.41	43.2	41.4
		AZI ^a	3.39	5.54	9.69	43.0	41.5
EE18	25	AXY ^b	4.40	6.77	10.08	47.4	33.1
		AZI ^b	2.37	4.09	7.39	34.4	33.0

^a - See Fig. 5a^b - See Fig. 5b

14 Johnston

TABLE 4 — WITHIN-BATCH COEFFICIENTS OF VARIATION FOR SETS OF FOUR 450 x 150 x 150 mm BEAMS

Matrix Type ^a	Fiber Code ^b	Amount kg/m ³	Vol %	First-Crack		Toughness Indices			Residual Strength Factors	
				σ_{fc}	δ_{fc}	I_5	I_{10}	I_{20}	$R_{5,10}$	$R_{10,20}$
25	CC63	25	0.32	2.3	3.6	3.3	10.1	13.6	25.0	19.0
25	CC63	40	0.51	4.4	4.6	7.9	15.1	21.6	24.9	29.7
25	CC63	55	0.70	3.1	13.1	1.1	7.1	16.0	14.6	26.6
25	EE18	25	0.32	8.1	10.2	1.9	1.8	3.3	3.6	11.3
25	EE18	50	0.64	2.6	3.6	2.2	4.3	3.6	7.5	3.5
25	EE18	62.5	0.80	4.3	2.6	1.0	1.6	2.2	2.4	3.0
25	EE18	75	0.96	4.2	4.0	1.2	3.7	6.5	6.8	9.9
30	EE18	62.5	0.80	3.0	1.2	1.1	4.3	8.5	8.9	14.4
30	EE18	75	0.96	7.3	12.7	1.4	3.6	7.6	7.2	12.6
30	CC63	40	0.51	3.8	8.2	0.9	1.9	6.2	4.2	12.3
30	CC63	55	0.70	3.5	4.4	2.1	6.8	10.4	12.1	14.5
25	HE50	20	0.25	5.4	12.5	0.9	2.4	6.5	5.0	17.6
25	HE50	33	0.42	8.0	9.9	3.4	5.4	8.2	9.4	13.1
30	HE50	40	0.51	7.1	10.3	1.1	2.1	5.4	4.7	9.0
30	CW60	40	0.51	3.5	8.0	1.8	3.3	4.4	6.0	6.2
Mean Values				4.7	7.3	2.1	4.9	8.3	9.5	13.5

^a - Specified Compressive Strength in MPa^b - CC is Crimped Crescent-Shaped Steel, CW is Crimped Wire, HE is Hooked-End Wire, EE is Enlarged-End Slit Sheet Metal. Numbers are Nominal Length in mm.

TABLE 5 — WITHIN-BATCH COEFFICIENTS OF VARIATION FOR SETS OF FOUR 300 x 100 x 100 mm BEAMS

Matrix Type ^a	Fiber Code ^b	Amount kg/m ³	Vol %	First-Crack		Toughness Indices			Residual Strength Factors	
				σ_{fc}	δ_{fc}	I_5	I_{10}	I_{20}	$R_{5,10}$	$R_{10,20}$
25	CC63	25	0.32	1.5	1.8	1.9	8.3	14.5	18.1	23.5
25	CC63	40	0.51	1.7	9.2	3.3	5.9	7.2	10.6	8.7
30	EE18	75	0.96	3.5	13.0	5.5	5.9	7.8	7.5	11.0
30	FP38	4.55	0.50	10.7	18.6	2.3	5.3	5.4	11.5	6.3
30	FP38	5.91	0.65	6.1	5.5	1.9	5.9	10.3	15.0	17.6
30	FP38	6.83	0.75	3.9	9.1	4.6	4.2	3.0	5.2	5.4
30	FP57	5.91	0.65	4.1	1.6	1.9	2.4	7.8	5.3	17.4
30	FP64	5.91	0.65	2.6	3.1	4.0	7.7	9.8	14.3	13.9
30	FP64	4.55	0.50	5.3	12.0	2.4	4.4	5.4	8.9	7.2
Mean Values				4.4	8.2	3.1	5.6	7.9	10.7	12.3

^a - Specified Compressive Strength in MPa^b - CC is Crimped Crescent-Shaped Steel, EE is Enlarged-End Slit Sheet Metal, FP is Fibrillated Polypropylene. Numbers are Nominal Length in mm.

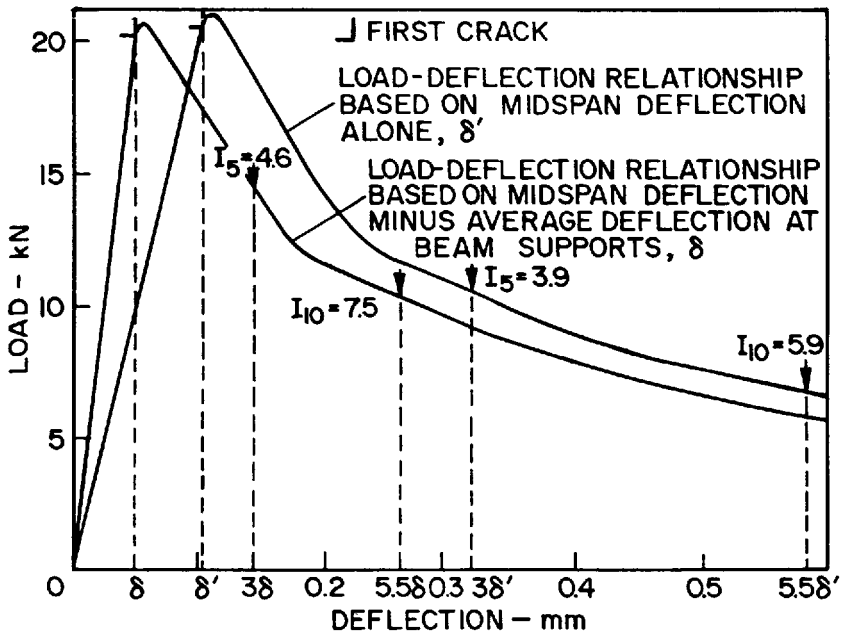


Fig. 1—Load-deflection relationships and associated toughness indices I_5 and I_{10} compared on the basis of nominal versus net deflection in 1985 (2)

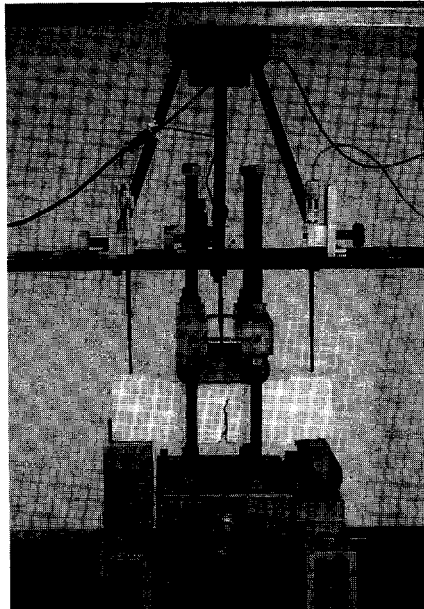


Fig. 2—Flexural testing equipment with 350 x 100 x 100 mm specimen on a 300 mm span showing the 3-transducer arrangement to determine net deflection

16 Johnston

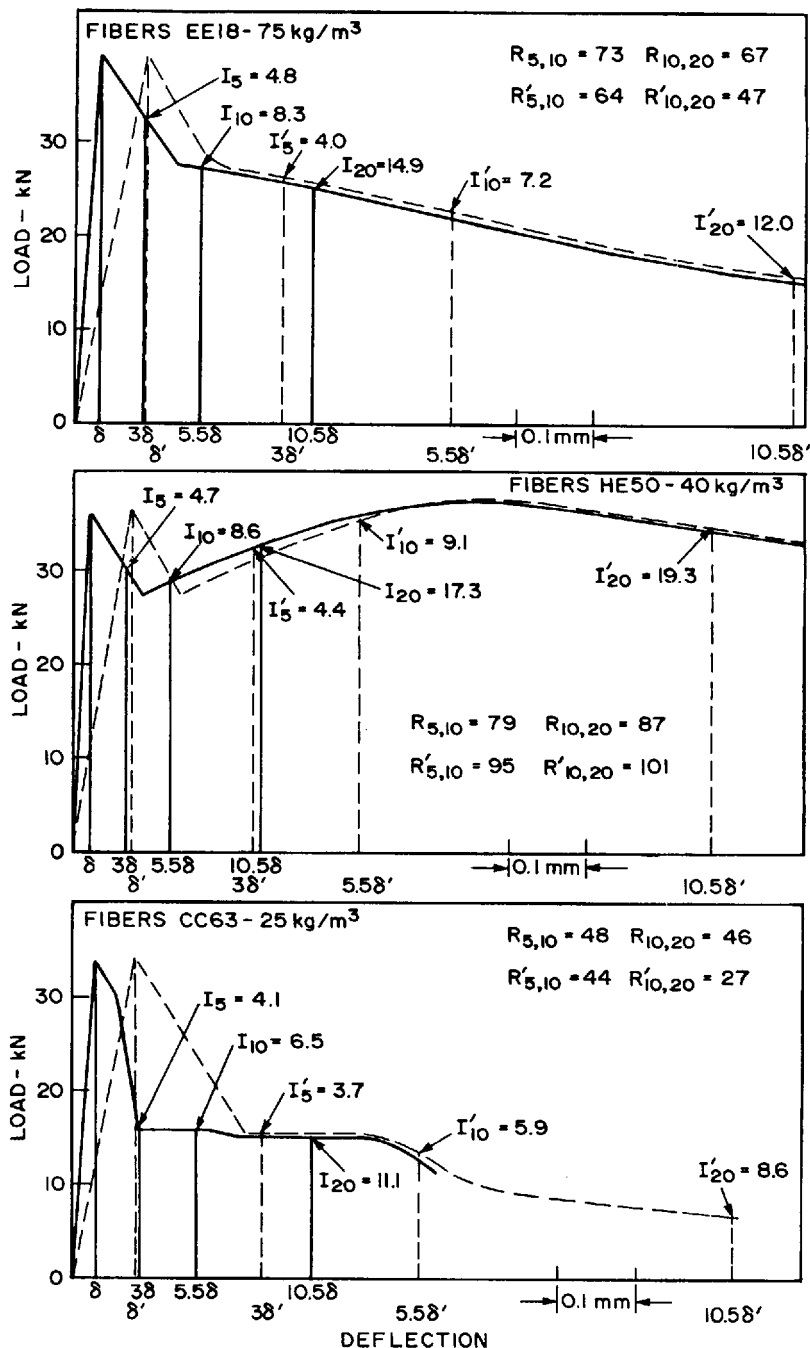


Fig. 3—Single-specimen load-deflection relationships for 450 x 150 x 150 mm beams showing end-point deflections and derived I and R values for nominal deflection (broken) and net deflection (solid) with 75 kg/m³, 0.94 percent volume of EE 18 steel fibers (top), 40 kg/m³, 0.34 percent volume of HE50 steel fibers (center), and 25 kg/m³, 0.31 percent volume of CC63 steel fibers (bottom)

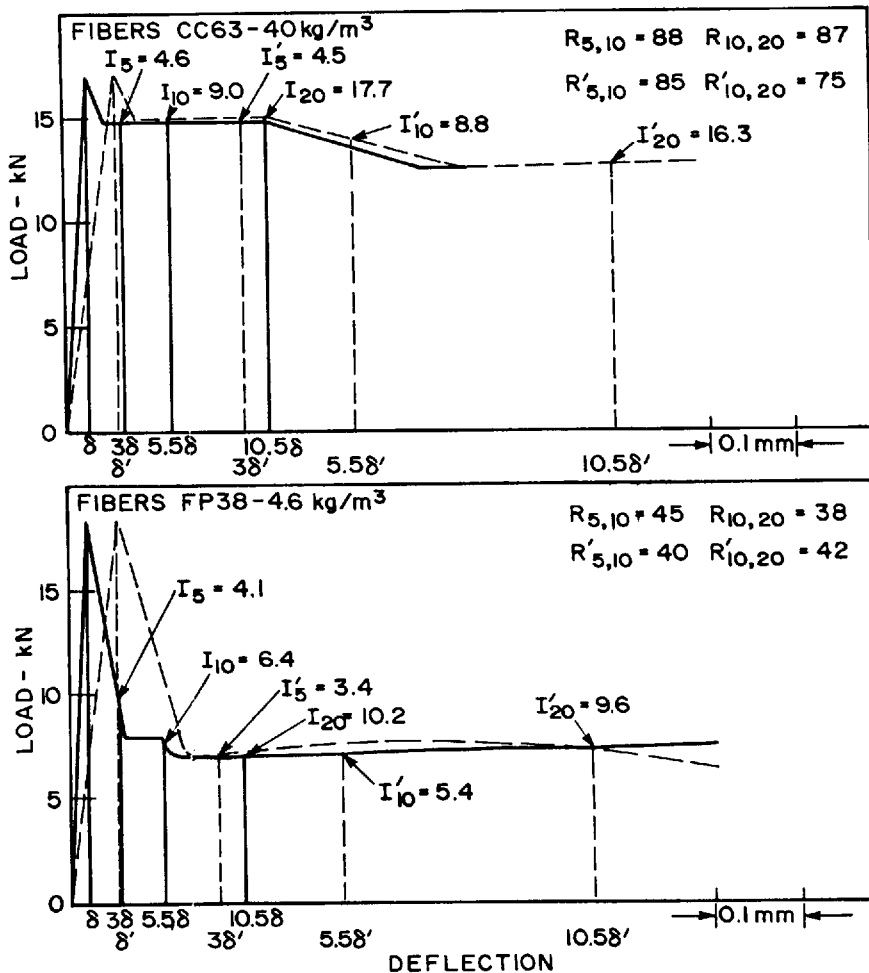


Fig. 4—Single-specimen load-deflection relationships for 300 x 100 x 100 mm beams showing end-point deflections and derived I and R values for nominal deflection (broken) and net deflection (solid) with 40 kg/m³, 0.5 percent volume of CX 63 steel fibers (top) and 4.55 kg/m³, 0.5 percent volume of FP 38 polypropylene fibers (bottom)

18 Johnston

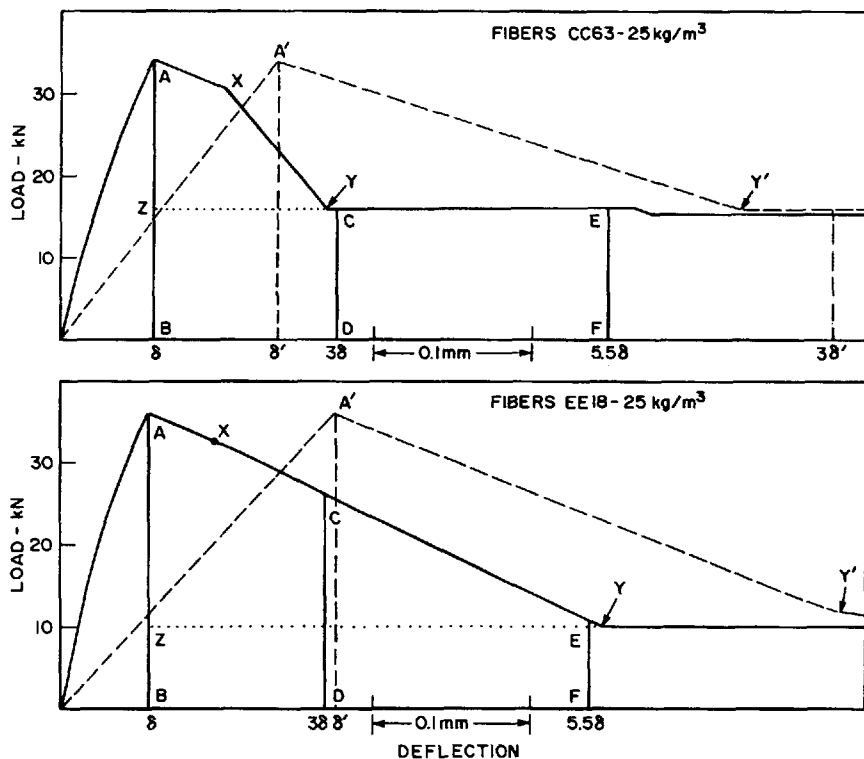


Fig. 5—Effect of uncertainty about the nature of load-deflection relationship immediately following first crack on I and R values

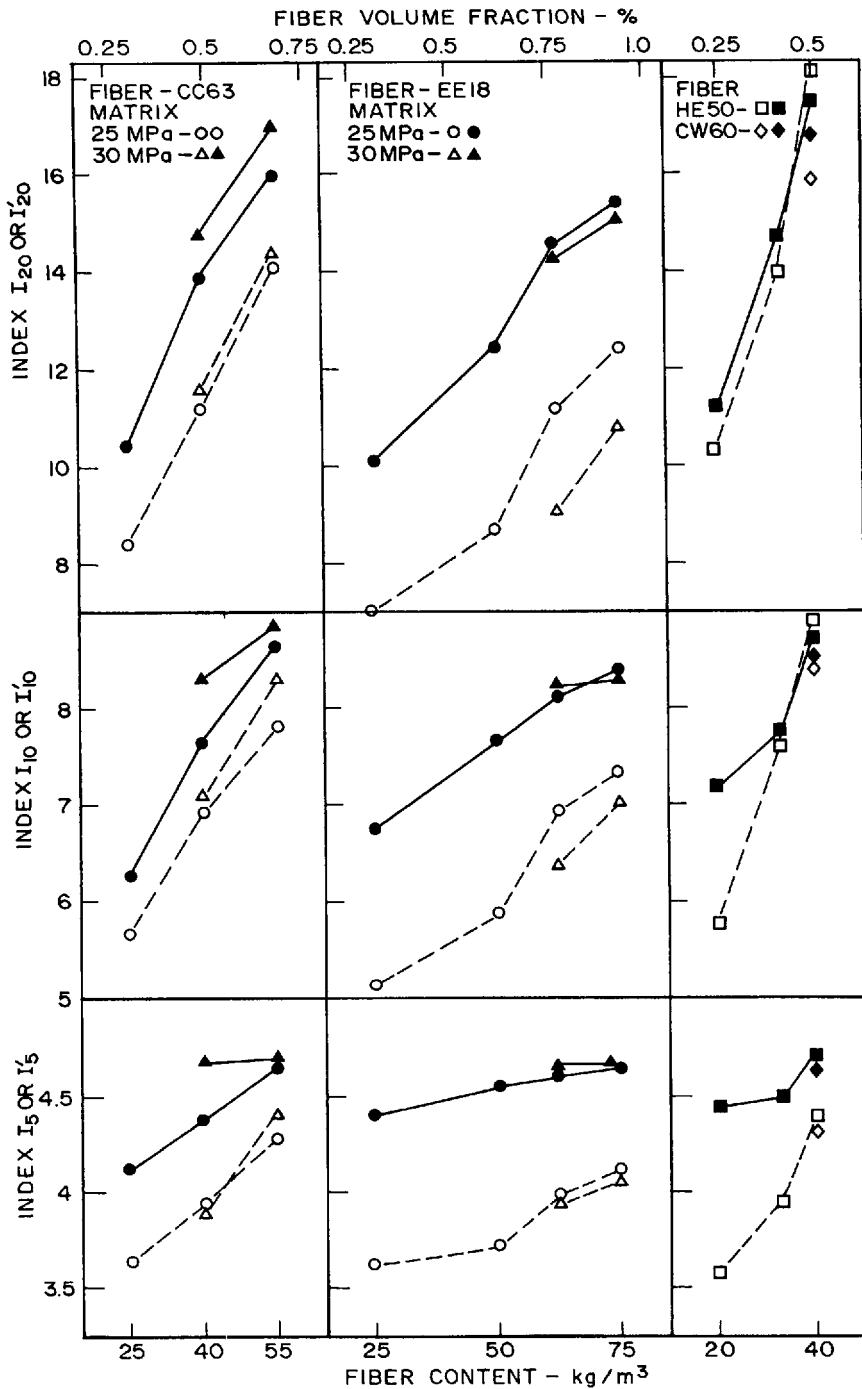


Fig. 6—Effects of fiber geometry and amount on I values based on nominal deflection (broken) and net deflection (solid) for 450 x 150 x 150 mm beams with four different steel fibers

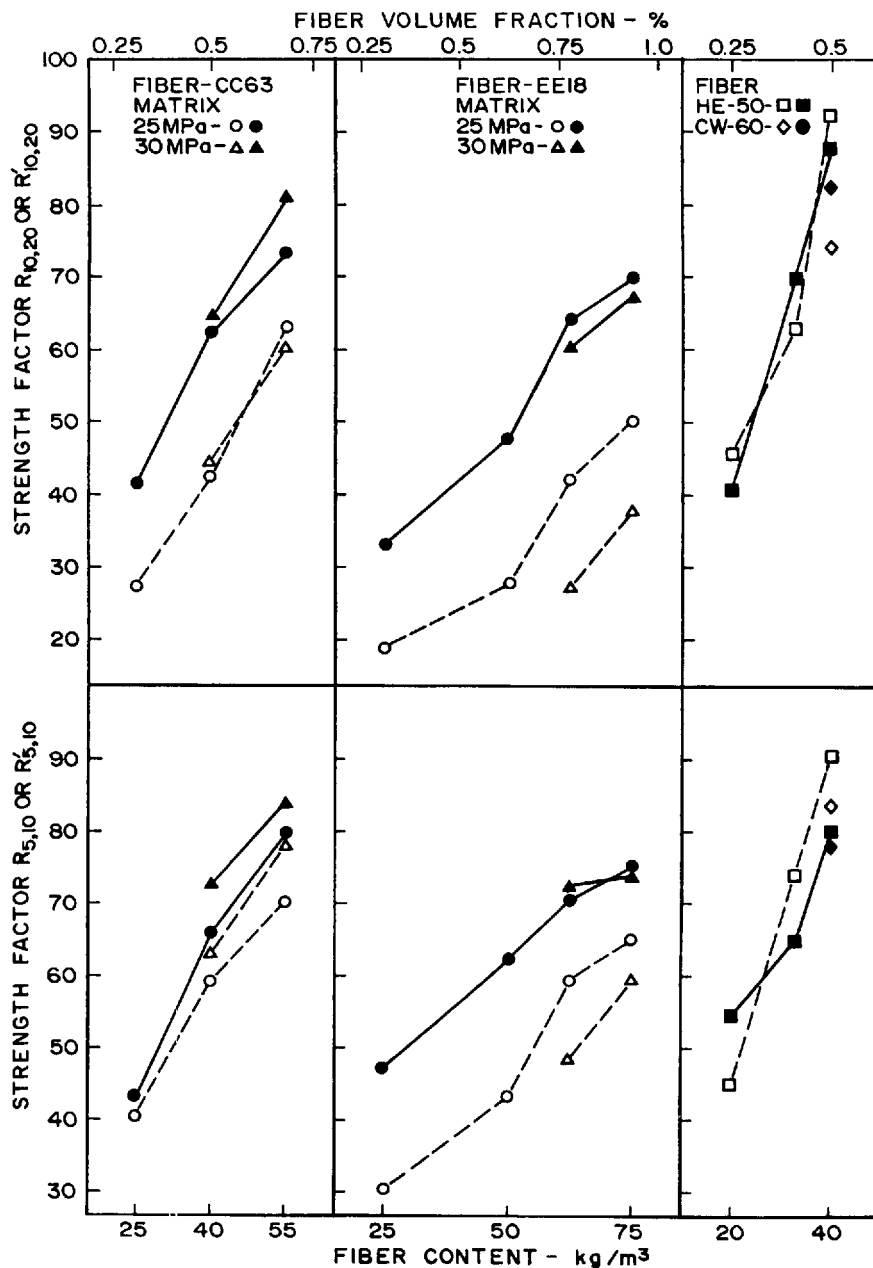


Fig. 7—Effects of fiber geometry and amount on R values based on nominal deflection (broken) and net deflection (solid) for 450 x 150 x 150 mm beams with four different steel fibers

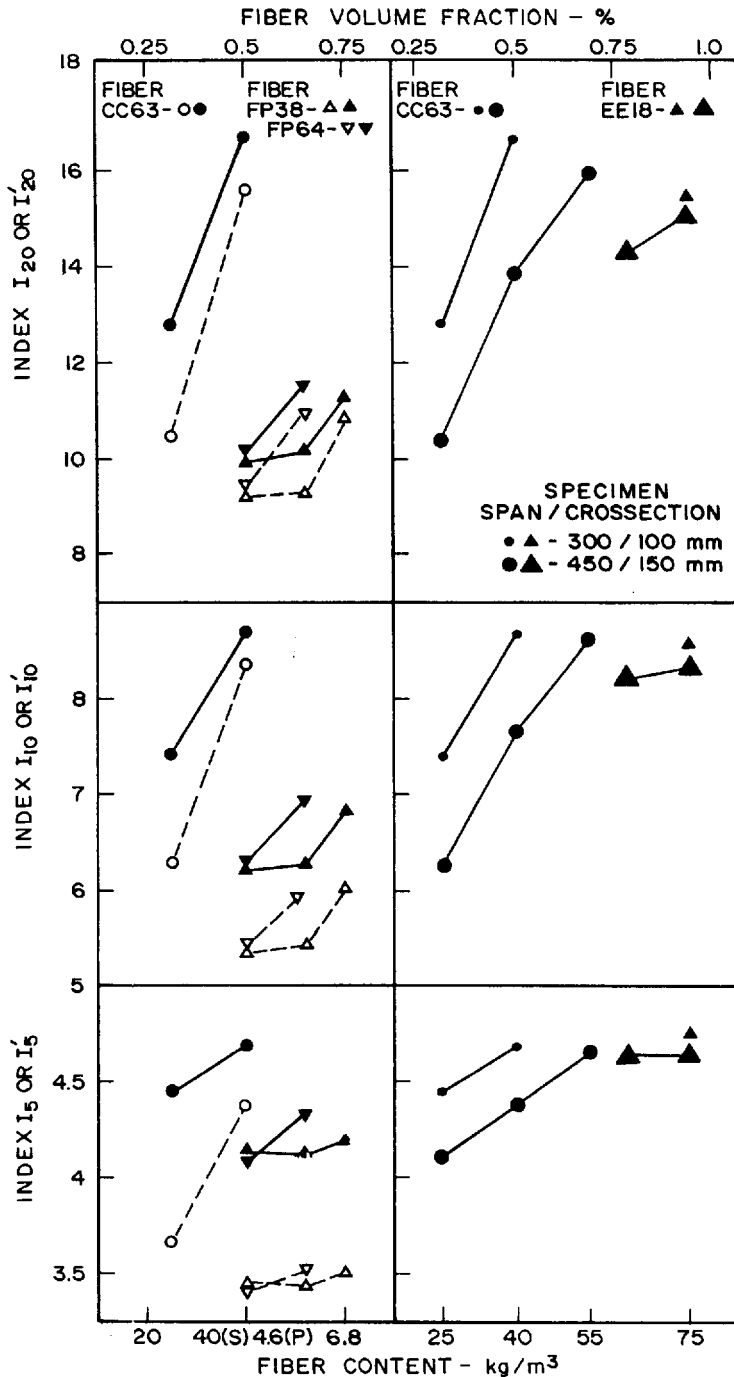


Fig. 8—Effects of fiber type and amount on I values for 300 x 100 x 100 mm beams with steel and polypropylene fibers (left) with an indication of the importance of specimen size for rigid steel fibers (right)

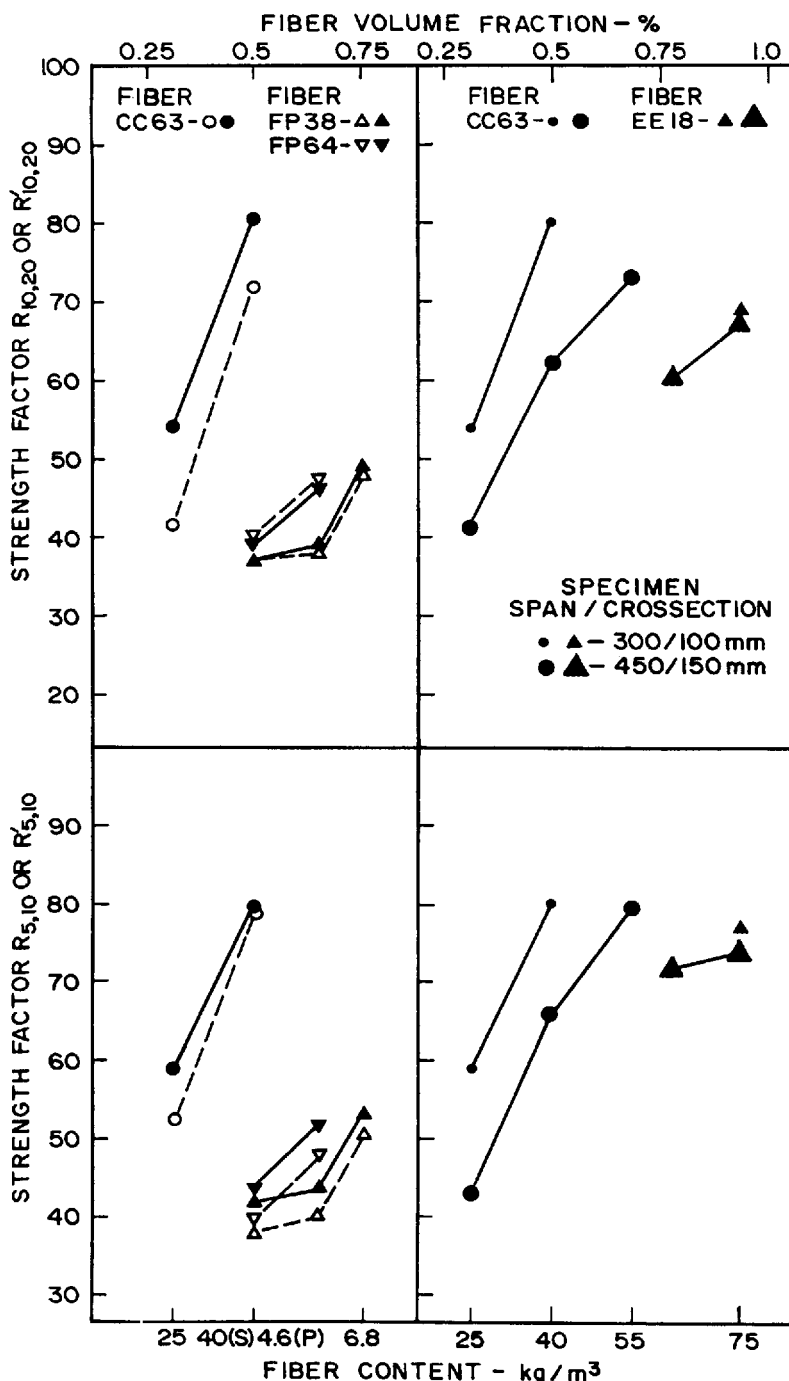


Fig. 9—Effects of fiber type and amount on R values for 300 x 100 x 100 mm beams with steel and polypropylene fibers (left) with an indication of the importance of specimen size for rigid steel fibers (right)

SP 155-2

Toughness of Fiber-Reinforced High-Strength Concrete from Notched Beam Tests

by D. Jamet, R. Gettu, V. S. Gopalaratnam, and A. Aguado

Synopsis: The toughness of fiber-reinforced concretes (FRC) is characterized from notched beam tests. The tests are performed under CMOD control in a servo-hydraulic machine to obtain the stable response of both the unreinforced concrete and the FRC. Several toughness measures are defined in terms of the experimentally obtained load versus crack opening (CMOD) curves. They give a better indication of the fundamental behavior of the concrete, avoid the problems associated with the approach based on the deflection of unnotched beams, and are amenable to the incorporation of serviceability considerations (e.g., crack widths). The effect of specimen size on the toughness is found to be significant in both the matrix- and fiber-dominated regimes of the FRC behavior. In general, toughness increases with specimen size and needs to be accounted for in the characterization.

The study was conducted on beams of a 70 MPa compressive strength silica fume concrete, with and without high-strength hooked steel fibers. It was found that the incorporation of a low volume fraction (1%) of steel fibers is sufficient to significantly decrease the brittleness of high strength concretes.

Keywords: Beams (supports); cracking (fracturing); ductility; fiber reinforced concretes; fibers; high-strength concretes; silica fume; steels; tests; toughness

24 Jamet et al

David Jamet is an Assistant Lecturer at the School of Civil Construction (University of Valparaiso, Chile). The experimental work reported in this paper was conducted during his stay at the Universitat Politècnica de Catalunya (UPC), Barcelona, Spain, funded by the Instituto de Cooperación Ibero-Americano.

ACI member **Ravindra Gettu** is Director of the Structural Technology Laboratory, UPC, Barcelona. He is a member of ACI Committee 446, Fracture Mechanics. The current interests of Dr. Gettu include experimental techniques, high strength concrete, fracture mechanics, toughness of FRC and concrete technology.

ACI member **Vellore S. Gopalaratnam** is Associate Professor of Civil Engineering at the University of Missouri-Columbia. He currently chairs ACI Committee 446, Fracture Mechanics. He is the secretary of ACI Committee 544, Fiber Reinforced Concrete, and chairs its Subcommittee on Testing. His current research interest includes fracture mechanics of cement and concrete composites.

ACI member **Antonio Aguado** is Chair Professor of Structural Concrete in the Department of Construction Engineering, UPC, Barcelona. Currently he is the Sub-director for Research at the School of Civil Engineering, Barcelona. The research interests of Dr. Aguado include concrete technology, high strength concrete, polymer concretes, fatigue, concrete dams and structural design.

INTRODUCTION

Toughness characterization is essential for the utilization of fiber reinforced concretes (FRC) since it provides the basis for quantifying the benefits of incorporating fibers, such as the post-crack load-carrying capacity and the ductility of the composite. It can also be used for relating the fundamental material behavior to structural performance. Conventionally, FRC toughness is obtained experimentally from a beam test. Such simple "engineering" measures can be correctly employed only when they are correlated to basic parameters, such as fracture toughness and fiber pullout characteristics. Moreover, effects of specimen size and geometry have to be accounted for when extrapolating laboratory results to larger structures. These effects depend on volumetric and planar energy dissipation mechanisms, whose proportions vary with the structural dimensions and crack configuration (1).

The present work studies the characterization of toughness based on notched beam tests that are conducted under crack-mouth opening displacement (CMOD) control in a closed-loop servo-hydraulic testing machine. This approach deals with the planar energy dissipation at a predefined critical cross-section and directly

uses the displacements associated with this plane. The size effects on the matrix- and fiber-dominated regimes of the specimen response are studied by testing three different sizes of geometrically-similar beams.

RESEARCH SIGNIFICANCE

The toughness characterization of fiber reinforced concretes has several objectives. For materials engineering purposes, toughness can be used to quantify the effectiveness of fibers, the ductility of the composite and the resistance offered by the material against crack propagation. In structural design, the toughness concept has been employed in the design against brittle failure and in the computation of load-carrying capacities at large deformations, and seems promising. The significance of toughness is especially important for high-strength concretes (HSC), which characteristically exhibit higher brittleness than normal concretes. One of the most rational methods of reducing the brittleness significantly is through the incorporation of steel fibers. In this paper, the toughness of a 70 MPa-strength silica fume concrete, reinforced with hooked steel fibers, is evaluated using the notched beam test. The aim is to use toughness as a measure of the effectiveness of fibers in increasing the ductility and energy absorption capacity of the concrete.

TOUGHNESS FROM NOTCHED BEAM TESTS

In the context of fiber reinforced concretes, toughness may be generally defined as an experimentally obtained measure of the energy absorption capacity of the composite. The most common test configuration used for this purpose is the bending of prismatic specimens. A recent review (1) of the popular methods for the determination of flexural toughness lists their merits, as well as problems associated with some of them (e.g., References 2-4). One approach that avoids many of those problems and seems to have a sound fundamental basis is the quantification of toughness through tests of notched specimens. Several such geometries and loading configurations have been proposed for concrete (5-9), including the single-edge notched plate under tension, double-notched prism under eccentric compression and the single-edge notched beam under three-point bending.

In the present study, the three-point bend (3PB) specimen shown in Figure 1 has been adopted as the test geometry. Nevertheless, most of the discussion and its implications also apply to other notched specimen geometries. The 3PB specimen has several advantages: (i) it is relatively easy to fabricate and test;

(ii) it deals with just one critical section and one propagating macro-crack, making its analysis and modeling simpler; (iii) the notch localizes the crack along its plane, reducing the variability of the crack path and non-symmetry of the deformations, both of which can occur in unnotched third-point loaded beam tests; (iv) a stable test can be performed, even for plain concrete, under crack-opening displacement control and (v) it has been extensively studied and documented in the fracture mechanics literature (see Reference (10) for a review).

Since the crack-mouth opening displacement (CMOD) of the notched beam can be measured accurately, prescribing it as the controlled variable in a closed-loop testing machine leads to stable post-peak response. Moreover, toughness based directly on CMOD, instead of beam deflection, avoids the errors related to extraneous deformations and unstable control beyond the peak load (8, 11, 12). In the present work, several toughness measures that have been previously defined on the basis of the deflection of unnotched beams are adapted for the CMOD of notched specimens. As shown later, the beam deflection and CMOD are related almost linearly in the post-peak regime, making it possible to convert deflection-based measures to analogous CMOD-based ones, and vice versa. Furthermore, the area under the experimentally obtained load-CMOD curve also reflects the energy absorbed during failure.

MATERIALS USED

The base (unreinforced) concrete had a composition of cement:sand:gravel: microsilica:water as 1:1.32:2.2:0.1:0.42, by weight. ASTM Type I (Spanish Type I 45-A, rapid hardening) cement, crushed limestone aggregates and silica fume slurry were used. DRAMIX® ZC 30/50 collated steel fibers with hooked ends (tensile strength of about 2000 MPa, elongation of 0.8-1.0%, 30 mm in length and 0.5 mm in diameter) were added to this concrete. The base concrete is hereafter denoted as HSC-0.0, and the concretes with 40 kg/m³ (volume fraction, V_f , of approximately 0.5%) and 80 kg/m³ ($V_f = 1\%$) of fibers as HSC-0.5 and HSC-1.0, respectively. All the concretes were prepared in a 250-liter paddle mixer. Other characteristics of the concretes are shown in Table 1. Note that different amounts of super-plasticizer were used in the three concretes since the addition of fibers decreased the workability considerably. Also, the slump was measured with a standard 300 mm cone and the air content with a mini air-meter. The compressive strength was obtained using standard 150×300 mm cylinders tested under stable conditions using circumferential deformation as the feedback in a closed-loop servo-hydraulic system. This strength was observed to decrease slightly due to the addition of fibers, which may be attributed to the increase in air content.

DETAILS OF BEAM TESTS

Beams of three different sizes, three specimens in each size, were cast from each of the concretes. The specimens were geometrically similar (fig. 1), with thickness = 90 mm and depth (d) = 90, 180 and 320 mm. The beams were cast in two layers normal to the loading plane and vibrated with a finishing screed. Notches of length $a_0 = 0.275d$ were cut with a diamond disc before testing. The beams were tested at the age of 25-28 days in a 100 T closed-loop INSTRON 8505 servo-hydraulic system. The tests were conducted at constant CMOD rates that were chosen to give peak loads in the HSC-0.0 specimens at approximately 3 minutes. The CMOD and net mid-span deflection (fig. 2) were monitored with an INSTRON clip-on crack-opening displacement gage and a Schaevitz LVDT, respectively. These measurements, as well as the load and piston displacement, were recorded electronically using the INSTRON FLAPS computer program. The data sampling was done at 0.6 seconds/point initially, and after substantial crack propagation was observed, the acquisition was slowed to 30 seconds/point.

TEST RESULTS AND SIZE EFFECT

Typical load-CMOD curves for the different concretes are shown in Figures 3-5. It can be seen that the tendency toward a hardening-type curve increases with fiber volume fraction and specimen size. The phenomenon was also observed previously for concrete with shorter straight fibers (9). This fiber-dominated size effect seems to be stronger in less ductile composites, where smaller specimens exhibit softening after the first-crack or first-peak, while much larger (geometrically similar) specimens undergo considerable hardening. This is contrary to the accepted trend for unreinforced concrete where smaller specimens exhibit more ductile post-peak responses (10). It obviously leads to a pronounced effect of the specimen size and geometry on toughness, and has to be accounted for adequately. A quantitative analysis of other size effects can be made from Table 2, where the maximum nominal stresses and relative displacements corresponding to first-crack, first-peak (or bend-over-point) and maximum load are given. Note that the nominal stress is the maximum stress in the ligament (i.e., net section in the notch plane) calculated using elastic beam theory. The relative displacements shown in Table 2 are the deflections and CMOD divided by the beam depth. The first-crack load corresponds to the point at which the load-CMOD curve exhibited a significant change in slope or a kink. The identification was made graphically and is admittedly subjective. The first-peak or bend-over-point is more easily determined, at least when non-hardening behavior is observed, and is taken to be the end of the matrix-dominated response. The

maximum load is obviously the most objective but some tests had to be terminated before it was reached due to practical reasons (i.e., large displacements and long test durations).

From Table 2, it can be observed that the scatter of the stresses is much less than those of the displacements. Also, the variations in the stresses generally increase with V_f . The first-peak stress exhibits much less scatter than the first-crack stress in all the concretes. Strong size effects are observed in both the matrix- and fiber-dominated responses for HSC-0.5. The stresses and relative displacements decrease with an increase in specimen size, the latter much more significantly. This is similar to that seen in HSC-0.0 as expected (10). However, this effect is observed only at the maximum loads of HSC-1.0 and not in the matrix-dominated response, implying that higher fiber volume-fractions may make the size effect much milder.

Another interesting aspect seen in Table 2 and Figure 6 is the relation between CMOD and deflection. The rate of change of deflection during the test decreases progressively (see inset of Figure 6), especially after first-cracking, to ultimately reach a constant value. This demonstrates why the test can always be controlled in a stable manner with CMOD control. Moreover, in the post-crack regime, the CMOD-deflection curve is linear with a deflection/CMOD ratio of approximately 0.65, for most of the specimens tested. This behavior may be attributed to rigid-body rotation of the beam halves, indicating the presence of a predominantly planar energy dissipation mechanism due to localization of the deformations. Another implication is that CMOD will be a more sensitive parameter to base toughness on since it increases much more than deflection, for a given change in load.

CMOD-BASED TOUGHNESS MEASURES

Several toughness quantifiers based on CMOD, more specifically the area under the load-CMOD curve, are developed and presented in this section. Most of them are analogous to the toughness measures defined by existing standards in terms of the load-deflection curves of unnotched beams.

Toughness Based on Absolute Load-CMOD Area

JCI (2) and RILEM (13) recommendations define toughness as the work done (i.e., area under the load-deflection curve) to reach a prescribed deflection. A similar measure of toughness, T_n^d , is defined here as the area under the load-CMOD curve. The CMOD limit is prescribed as a fraction of the beam depth

d/n , where values of 250, 500, 1000 and 2000 have been chosen for n . The corresponding values of T_n^d are tabulated for the three concretes in Table 3. Obviously, T_n^d increases with V_f , for all n and for all d . This type of toughness measure can only be used for comparing the values obtained with a specific specimen size and geometry.

For better comparison, T_n^d can be divided by the ligament area to get a more fundamental quantity that is analogous to fracture energy. This, denoted as T_n^E , is the area under the load-CMOD curve, until the prescribed CMOD limit d/n , divided by the ligament area. These values are given in Table 3 for different d and n . Also evident from the table is that the value of T_n^E increases with the specimen size. This effect increases with V_f , especially during post-crack hardening where the larger specimens exhibit more significant increases in load-carrying capacity than the smaller specimens. Also, T_n^E increases considerably with CMOD limit (i.e., n), which should, therefore, be based on practical crack width considerations. The increase in T_n^E with CMOD limit is greater for a larger V_f indicating the greater effectiveness of the fibers at larger deformations.

Toughness Based Only on the Fiber Contribution

Figures 7 and 8 compare typical load-CMOD curves of the two FRCs with those of HSC-0.0. The difference between the FRC and unreinforced concrete can be attributed to the presence of the fibers, and can be used to quantify the fiber performance. Such toughness measures, which explicitly deduct the toughness of the unreinforced matrix, have been recommended by ACI (3) and some German standards (e.g., Reference 14). Following this approach, a relative toughness can be computed as $T_n^{Er} = T_n^E$ (for FRC) - T_n^E (for HSC-0.0), for the values of n used earlier. These values presented in Table 4 show clearly the increase in toughening with fiber effectiveness and the specimen size effects. This type of toughness measure obviously has several advantages but would always require the testing of a companion specimen of the base concrete. This could be considered as a disadvantage. It can, however, be handled by defining an idealized curve for the unreinforced concrete based empirically on other tests (as in Reference 14) or by determining it analytically using an approach such as nonlinear fracture mechanics.

Nondimensional Toughness Indices Based on Relative Displacements

The ASTM Standard C 1018 (4) defines toughness indices that are equal to the areas under the load-deflection curve of an unnotched beam until prescribed multiples of the first-crack deflection divided by the area until first-crack. A

similar approach was used by Barr and Hasso (5) for defining toughness based on notched and unnotched specimens.

Bryars et al. (9) recently used a toughness index based on the load-CMOD area of a notched beam defined in terms of the first-peak, instead of the first-crack. They found that the value of such an index increases with the CMOD limit, is practically size-independent and reflects the effectiveness of the fibers satisfactorily. Two similar indices are evaluated here: T_m^{Ic} defined as the ratio of the area of the load-CMOD curve until m times the first-crack CMOD and the area until the first-crack; and T_m^{Ip} is defined similarly in terms of the first-peak, instead of the first-crack. The results for $m = 5, 10$ and 20 are given in Table 5.

From Table 5 it can be seen that all the indices reflect the fiber effectiveness and are practically size-independent. Further testing is required to confirm whether these indices are influenced by specimen geometry and dimensions. However, as concluded by other researchers (1, 8), the indices are not very sensitive to the reinforcement parameters at small values of m . This implies that tests have to be continued until larger displacements are attained. Moreover, comparisons with previous results of Bryars et al. (9) suggest that the sensitivity or the dependence on the factor m may depend on the material. In their study, conducted on a HSC with shorter straight fibers, use of $m = 5$ yielded T_m^{Ip} values that were reasonably sensitive to the increase in fiber length (i.e., fiber effectiveness).

CONCLUSIONS

1. The load-CMOD curve obtained experimentally from notched beam tests provides a good basis for the toughness characterization of fiber reinforced concretes. This approach avoids the problems associated with the characterization based on the deflection of unnotched beams and is amenable to the incorporation of serviceability limits (such as crack widths). Since the toughness based on CMOD is derived from the deformation of one critical cross-section and a single crack, it is more closely related to the fundamental behavior of the composite.
2. Several toughness measures have been analyzed in this study. They have been based on (i) the absolute area under the load-CMOD curve, (ii) area between the curves of unreinforced and fiber reinforced concretes and (iii) nondimensional indices defined as ratios of the post- and pre-cracking areas. All such quantities reflect the effectiveness of the fibers adequately and warrant further research.

3. There is a significant effect of the size of the specimen on its behavior, in both the matrix- and fiber-dominated regimes. This should be adequately considered in toughness characterization.
4. The brittleness of high-strength silica fume concrete can be decreased considerably with the addition of a low volume-fraction (1% or less) of randomly distributed steel fibers.

Acknowledgements

This research was partially supported by Spanish DGICYT grants PB90-0598 and MAT93-0293 to the Universitat Politècnica de Catalunya, Barcelona. The authors are grateful to D.Nemegeer of N.V.Bekaert (Belgium), J.Puig of Cementos Molins (Spain) and J.L.Rodríguez of GRACE (Spain) for providing the materials used in this study. V.S. Gopalaratnam was supported, during his sabbatical stay as Visiting Professor at the UPC, by the DGICYT (grant SAB93-0190) and the University of Missouri-Columbia. The help of S.Carmona, who performed the compression tests, is deeply appreciated.

REFERENCES

1. Gopalaratnam, V.S., and Gettu, R., "On the Characterization of Flexural Toughness in FRC", *Proc., US NSF - University of Sheffield Workshop on Fibre Reinforced Cement and Concrete* (Sheffield, U.K.), eds. R.N.Swamy and V.Ramakrishnan, pp. 161-180, 1994.
2. JCI Standard SF-4, "Method of Tests for Flexural Strength and Flexural Toughness of Fiber Reinforced Concrete", *Japan Concrete Institute Standards for Test Methods of Fiber Reinforced Concrete*, Tokyo, pp. 45-51, 1984.
3. ACI Committee 544, "Measurements of Properties of Fiber Reinforced Concrete", *ACI Mater. J.*, V.85, No.6, pp. 583-593, 1988.
4. ASTM C 1018-92, "Standard Test Method for Flexural Toughness and First-Crack Strength of Fiber-Reinforced Concrete Using Beam With Third-Point Loading", *Annual Book of Standards*, ASTM, Philadelphia, V. 04.02, pp. 510-516, 1992.
5. Barr, B.I.G., and Hasso, E.B.D., "A Study of Toughness Indices", *Mag. Concr. Res.*, V.37, No.132, pp. 162-173, 1985.

32 Jamet et al

6. RILEM Committee 50-FMC, "Determination of Fracture Energy of Mortar and Concrete by Means of Three-Point Bend Tests on Notched Beams", Draft Recommendation, *Mater. Struct.*, V.18, No.106, pp. 285-290, 1985.
7. RILEM Committee 89-FMT, "Determination of Fracture Parameters (K_{Ic}^s and $CTOD_c$) of Plain Concrete Using Three-Point Bend Tests", Draft Recommendation, *Mater. Struct.*, V.23, No.138, pp. 457-460, 1990.
8. Gopalaratnam, V.S., Shah, S.P., Batson, G.B., Criswell, M.E., Ramakrishnan, V., and Wecharatana, M., "Fracture Toughness of Fiber Reinforced Concrete", *ACI Mater. J.*, V.88, No.4, pp. 339-353, 1991.
9. Bryars, L., Gettu, R., Barr, B., and Ariño, A., "Size Effect on the Fracture of Fiber-Reinforced High-Strength Concrete", *Proc., Europe-U.S. Workshop on Fracture and Damage in Quasibrittle Structures* (Prague), E&FN Spon, London, 1994.
10. Gettu, R., and Shah, S.P., "Fracture Mechanics", in *High Performance Concrete and Applications*, eds. S.P.Shah and S.H.Ahmad, Edward Arnold Publishers, London, pp. 161-212, 1994.
11. Trottier, J.-F., and Banthia, N., "Toughness Characterization of Steel-Fiber Reinforced Concrete", *J. Civil Engng. Mater.*, V.6, No.2, pp. 264-289, 1994.
12. Mindess, S., Chen, L., and Morgan, D.R., "Determination of the First-Crack Strength and Flexural Toughness of Steel Fiber-Reinforced Concrete", *Advn. Cem. Bas. Mat.*, V.1, No.5, pp. 201-208, 1994.
13. RILEM Committee 49-TFR, "Testing Method for Fibre Reinforced Cement-Based Composites", *Mater. Struct.*, No.102, pp. 441-456, 1984.
14. "Technologie des Stahlfaser-betons und Stahlfaserspritzbetons" (Technology of Steel Fiber Reinforced Concrete and Steel Fiber Shotcrete, in German), Deutschen Beton-Vereins, pp. 3-18, 1992.

Unit Conversions

1 mm = 0.0394 inches
 1 m = 3.28 ft.
 1 MPa = 145 psi
 1 kg = 2.2 lbs
 1 N = 0.224 lbs-force
 1 liter = 0.0353 cu. ft.

TABLE 1 — CONCRETE PROPERTIES

Material	Fiber Content (kg/m ³)	Super-plasticizer used* (liters/m ³)	Slump (cm)	Air content (%)	21-day cylinder compressive strength (MPa)
HSC-0.0	0	8	17	3	73 (± 4)
HSC-0.5	40	15	25	4	66 (± 1)
HSC-1.0	80	20	19	8	64 (± 2)

*1:2 mixture, by weight, of solid super-plasticizer and water.

TABLE 2 — MEAN VALUES AND COEFFICIENTS OF VARIATION OF STRESS, CMOD AND DISPLACEMENT AT FIRST-CRACK, FIRST-PEAK, AND MAXIMUM LOAD

Material	d (mm)	First-crack			First-peak or Bend-over-point			Maximum load		
		Stress (MPa)	Relative CMOD (μm/mm)	Relative deflection (μm/mm)	Stress (MPa)	Rel. CMOD (μm/mm)	Rel. defl. (μm/mm)	Stress (MPa)	Rel. CMOD (μm/mm)	Rel. defl. (μm/mm)
HSC-0.0	90	4.53 ±4.7%	0.156 ±12.4%	0.156 ±12.4%	5.28 ±0.7%	0.252 ±6.7%	0.237 ±2.7%	5.28 ±0.6%	0.252 ±6.7%	0.237 ±2.7%
	180	4.12 ±6.6%	0.120 ±9.6%	0.136 ±20.2%	4.40 ±2.5%	0.167 ±20.0%	0.156	4.40 ±2.5%	0.167 ±20.0%	0.156
	320	3.34 ±5.1%	0.098 ±9.7%	0.099 ±7.9%	3.71 ±1.3%	0.143 ±11.2%	0.135 ±10.9%	3.71 ±1.3%	0.143 ±11.2%	0.135 ±10.9%
HSC-0.5	90	4.07 ±19.7%	0.137 ±23.4	0.159 ±21.3%	5.49 ±6.9%	0.296 ±22.9%	0.289 ±20.4%	--	--	--
	180	3.65 ±4.5%	0.107 ±6.0%	0.120 ±10.7%	5.08 ±6.2%	0.257 ±12.6%	0.244 ±10.4%	5.60 ±13.8%	3.05 ±2.3%	2.03 ±1.7%
	320	3.51 ±8.6%	0.104 ±21.1%	0.104 ±9.6%	4.20 ±5.3%	0.192 ±17.4%	0.174 ±8.9%	4.85 ±11.8%	2.07 ±8.2%	1.38 ±8.3%
HSC-1.0	90	3.96 ±8.9%	0.126 ±22.2%	0.133 ±14.4%	5.71 ±2.5%	0.307 ±9.1%	0.259 ±2.5%	9.48 ±21.9%	6.94 ±2.0%	4.49 ±1.1%
	180	4.62 ±13.4%	0.150 ±13.4%	0.161 ±13.8%	5.71 ±3.4%	0.265 ±15.2%	0.257 ±9.0%	8.59 ±3.9%	3.17 ±6.2%	2.24 ±2.9%
	320	3.75 ±22.2%	0.138 ±31.5%	0.117 ±29.6%	6.09 ±26.3%	0.675 ±58.1%	0.514 ±51.2%	6.94 ±24.7%	2.81 ±14.0%	1.91 ±14.8%

Relative CMOD = CMOD/Beam depth

Relative deflection = Deflection/Beam depth

Stress = maximum nominal stress computed for net section

Notes: Coefficients of variation are omitted when less than three data values were used.

Maximum load data are omitted when there was no clear maximum in the test range.

TABLE 3 — AVERAGE TOUGHNESS VALUES FOR DIFFERENT CONCRETES BASED ON LOAD-CMOD AREA

Material	d (mm)	T_n^A = area until CMOD limit d/n (N-m)				$T_n^E = T_n^A$ / ligament area (N/mm)			
		$n=2000$	$n=1000$	$n=500$	$n=250$	$n=2000$	$n=1000$	$n=500$	$n=250$
HSC-0.0	90	0.195	0.301	0.379	---	0.033	0.051	0.065	---
	180	0.604	0.800	0.912	---	0.051	0.068	0.078	---
	320	1.396	1.771	---	---	0.069	0.085	---	---
HSC-0.5	90	0.221	0.447	0.903	1.90	0.038	0.076	0.154	0.324
	180	0.839	1.793	3.882	8.39	0.071	0.153	0.331	0.714
	320	2.321	5.188	11.151	22.35	0.111	0.248	0.534	1.071
HSC-1.0	90	0.234	0.546	1.243	3.94	0.040	0.093	0.217	0.671
	180	0.925	2.165	5.187	12.18	0.079	0.184	0.442	1.037
	320	2.629	6.368	14.825	29.62	0.126	0.305	0.710	1.418

Note that values are omitted when the experimental CMOD range is smaller than the limit.

TABLE 4 — FIBER CONTRIBUTION TO TOUGHNESS

Material	d (mm)	T_n^{Er} (N/mm)		
		$n = 2000$	$n = 1000$	$n = 500$
HSC-0.5	90	0.005	0.025	0.089
	180	0.020	0.085	0.253
	320	0.042	0.163	---
HSC-1.0	90	0.007	0.042	0.149
	180	0.028	0.116	0.364
	320	0.057	0.220	---

$$T_n^{Er} = T_n^E \text{ (of FRC)} - T_n^E \text{ (of base concrete)}$$

TABLE 5 — MEAN VALUES OF NONDIMENSIONAL TOUGHNESS INDICES

Material	d (mm)	Index based on first-crack CMOD, T_m^{Ic}			Index based on first-peak CMOD, T_m^{Ip}		
		$m = 5$	$m = 10$	$m = 20$	$m = 5$	$m = 10$	$m = 20$
HSC-0.0	90	6.0	8.1	10.1*	3.5	4.2	---
	180	5.8	7.4*	---	4.4*	4.3*	---
	320	5.7	7.3	---	3.7	4.3	---
HSC-0.5	90	9.1	18.2	37.4	5.7	11.9	22.5*
	180	9.9	19.4	45.9	6.3	13.7	28.0*
	320	9.3	20.8	45.6	6.9	15.2	38.6*
HSC-1.0	90	10.8	24.8	56.6	7.4	17.1	---
	180	9.7	23.0	54.7	7.7	18.3	---
	320	10.1	23.9	53.6	6.9*	14.6*	---

$T_m^{Ic} = (\text{load-CMOD area until the CMOD value of } m\delta_c) / (\text{area until } \delta_c)$

$T_m^{Ip} = (\text{load-CMOD area until the CMOD value of } m\delta_p) / (\text{area until } \delta_p)$

δ_c = CMOD at first-crack, δ_p = CMOD at first-peak

Values marked with * were obtained with less than three specimen data

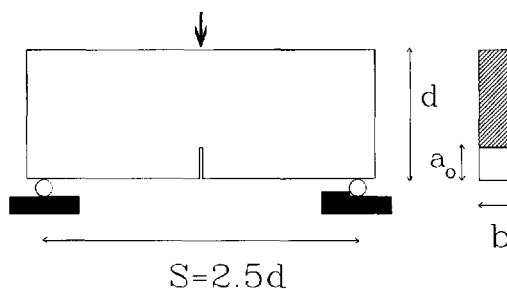


Fig. 1—Three-point bend geometry

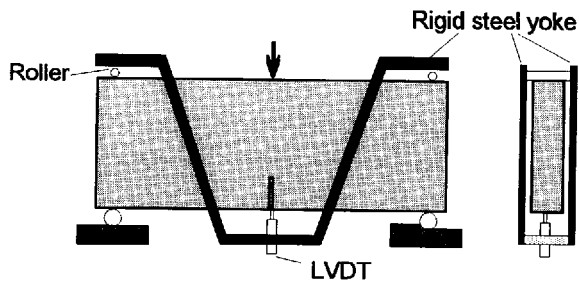


Fig. 2—Schematic of specimen and yoke used to measure deflection

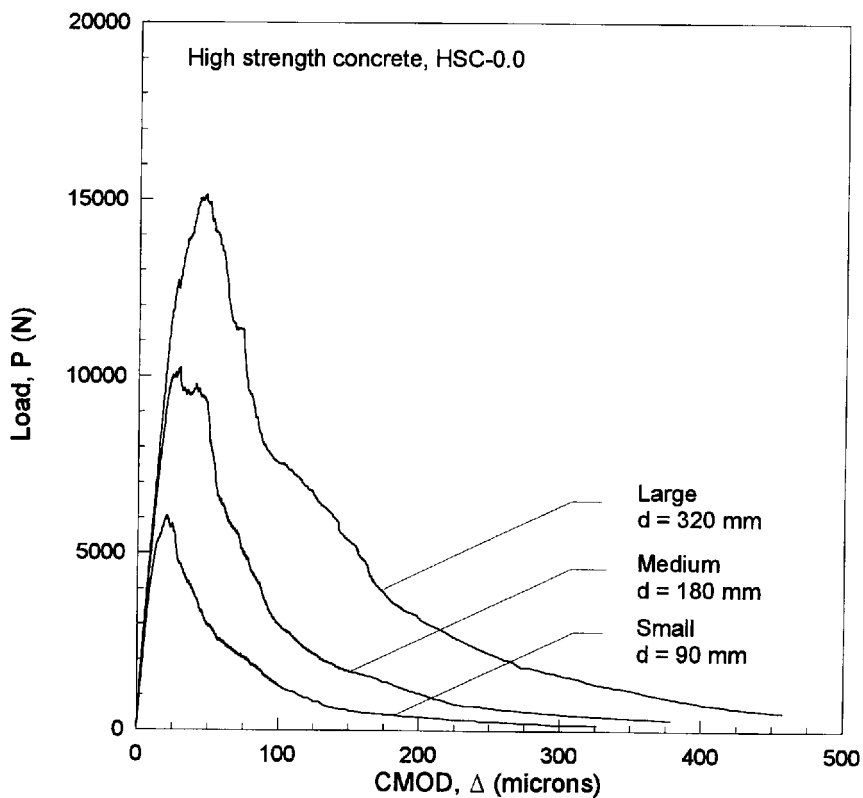


Fig. 3—Typical load-CMOD curves for unreinforced concrete

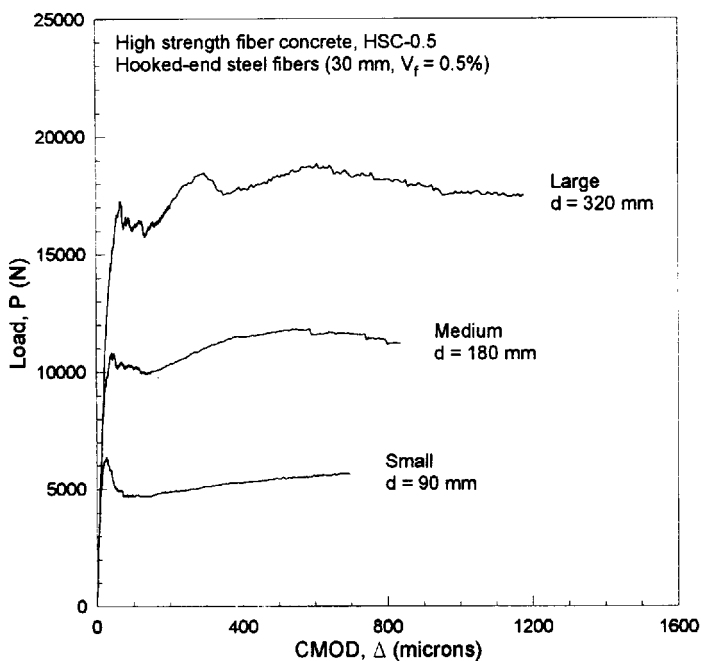


Fig. 4—Typical load-CMOD curves for concrete with fiber $V_f = 0.5$ percent

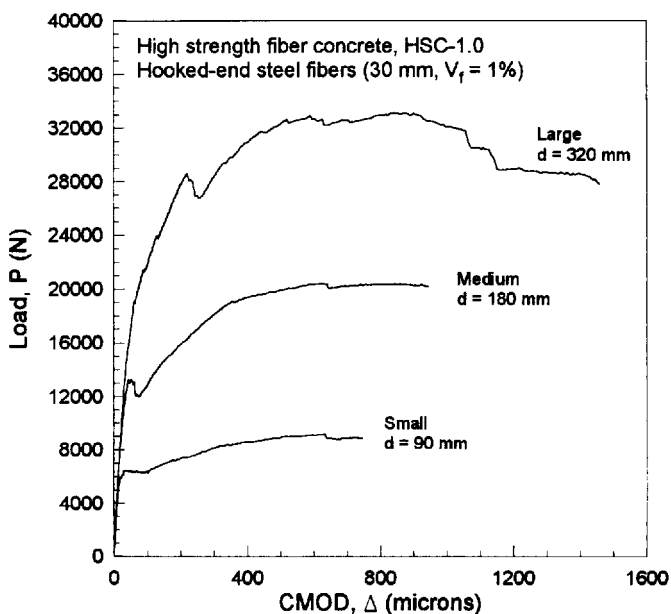


Fig. 5—Typical load-CMOD curves for concrete with fiber $V_f = 1.0$ percent

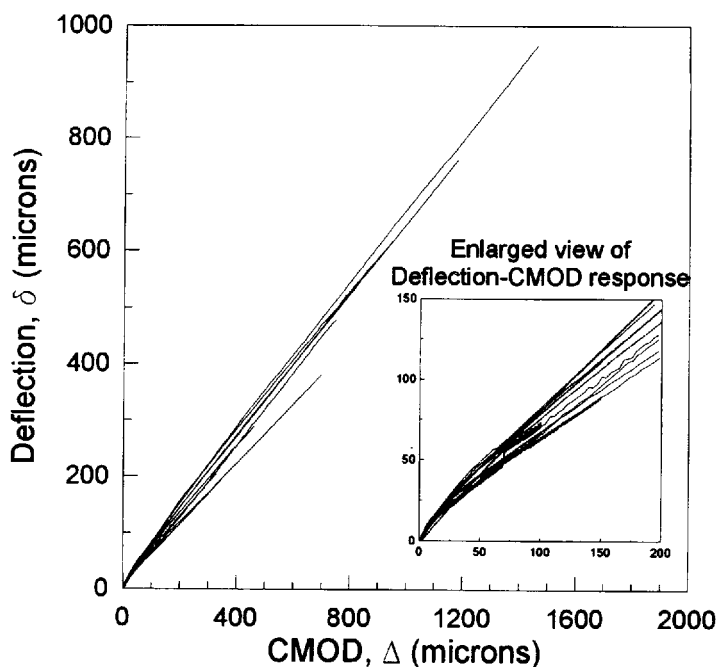


Fig. 6—Typical deflection-CMOD curves for all concretes and all specimen sizes

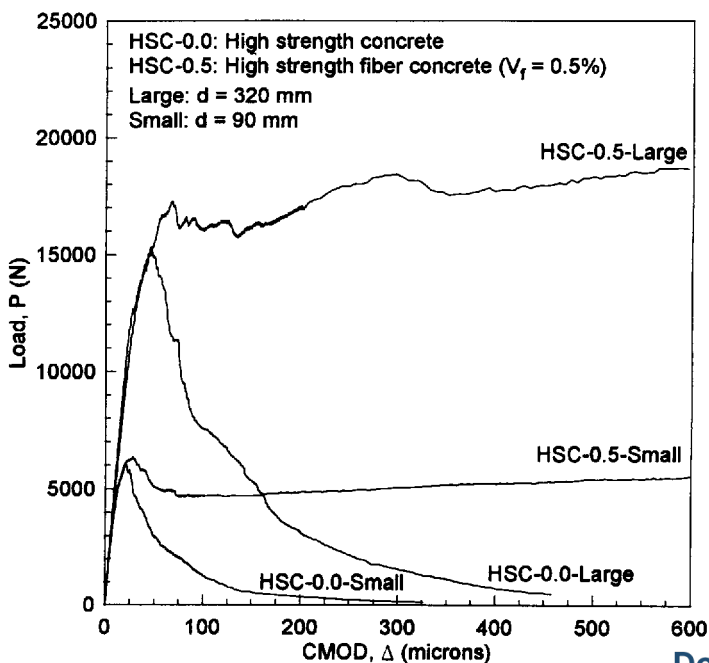


Fig. 7—Comparison of typical load-CMOD curves of the small and large specimens of unreinforced concrete and FRC with $V_f = 0.5$ percent

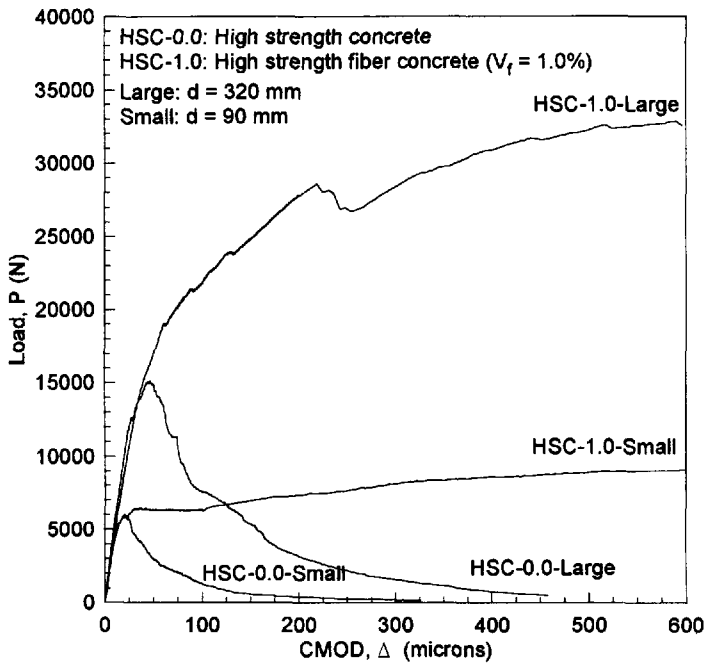


Fig. 8—Comparison of typical load-CMOD curves of the small and large specimens of unreinforced concrete and FRC with $V_f = 1.0$ percent

SP 155-3

Comparative Toughness Testing of Fiber Reinforced Concrete

by L. Chen, S. Mindess, D. R. Morgan, S. P. Shah,
C. D. Johnston, and M. Pigeon

Synopsis: Round-robin tests of the flexural toughness of fibre reinforced concrete were carried out using six different testing machines in five different laboratories. Six groups of beams, including a plain concrete control, two different volumes of polypropylene fibres, and three different volumes of steel fibres were tested in accordance with ASTM C1018, with special care taken to exclude the “extraneous” deflections due to deformations at the specimen supports. The results from each laboratory were used to compute the ASTM C1018 toughness indices I_5 , I_{10} , I_{20} , I_{30} and I_{50} , and the corresponding residual strength factors $R_{5,10}$, $R_{10,20}$, $R_{20,30}$ and $R_{30,50}$. In addition, the JSCE Toughness and Toughness Factor were also computed. It was found that, although the load vs deflection curves were inherently quite variable, in most cases there was no significant difference amongst the participating laboratories, except for those mixes with a very low toughness.

It was found that the ASTM C1018 toughness indices, particularly I_5 and I_{10} , did not discriminate very well amongst the different fibre contents or different fibre types; the JSCE parameters were rather more successful in this regard.

Keywords: Fiber reinforced concretes; fibers; flexural strength; polypropylene fibers; steels; strength; tests; toughness

BIOGRAPHIES

Mr. Lianrong Chen is a Ph.D. candidate in the Department of Civil Engineering at The University of British Columbia. He obtained his BS and MS degrees in Materials Science and Engineering from Tongji University and Wuhan University of Technology, respectively, in China. His research interests include mechanical properties and durability of concrete and fibre reinforced concrete.

ACI Fellow, Sidney Mindess, is a professor in the Department of Civil Engineering at The University of British Columbia. He is a member of ACI Committee 370, Short-Duration Dynamic and Vibratory Load; and ACI Committee 446, Fracture Mechanics. He is also a member of the Co-ordinating Committee of RILEM.

ACI Fellow, Dudley R. Morgan, is chief materials engineer for the AGRA Earth & Environmental Group in North America. He is a member of ACI Committee 544, Fibre Reinforced Concrete, ACI Committee 506, Shotcrete and ACI Committee 234, Silica Fume, and has been extensively involved in specifying and testing toughness for fibre reinforced concrete and shotcrete projects.

ACI Fellow, Surendra P. Shah, is a Walter P. Murphy professor of Civil Engineering and the Director of the National Science Foundation's Science and Technology Centre for Advanced Cement-Based Materials. He is also the editor-in-chief of the recently established Advanced Cement Based Materials Journal, published by Elsevier in affiliation with ACI. He is post chairman of ACI Committee 544, Fibre Reinforced Concrete; and ACI Committee 215, Fatigue of Concrete; and is a current member of several ACI Committees. He received the ACI Anderson Award in 1989.

ACI Fellow, Colin D. Johnston, is a professor of Civil Engineering at the University of Calgary. He was awarded the ACI Wason Medal for materials research in 1976, and received a Government of Alberta Achievement Award for excellence in concrete engineering the same year. He was chairman of ASTM Subcommittee C09.03.04 on Fibre Reinforced Concrete from 1980 to 1990, and is currently a member of ACI Committee 544, Fibre Reinforced Concrete.

ACI Fellow, Michel Pigeon, is a professor of Civil Engineering at Laval University. He is a member of ACI Committee 201, Durability of Concrete; and of RILEM Committee TC115 on High-Strength Concrete and TC117 on Frost Resistance. He was president of the Quebec and Eastern Ontario Chapter of ACI in 1986-1987.

INTRODUCTION

The primary reasons for adding discontinuous fibres to a plain concrete matrix are to improve the post-cracking response of the concrete, i.e., to improve its energy absorption capacity and apparent ductility, and to provide crack resistance and crack control. For fibre reinforced concrete (FRC), the concept of flexural toughness (which is a measure of the energy absorption) is often used to characterize its performance. However, it has long been known that the flexural toughness of FRC is sensitive to the way in which it is measured (1-4). In addition, there are a number of uncertainties regarding the way in which the flexural toughness should be determined, interpreted or used.

Probably the most common method of trying to quantify the flexural toughness of FRC is that prescribed in ASTM C1018 (5). In this test, the energy absorbed up to certain specified deflections is normalized by the energy absorbed up to the point of first cracking; the resulting toughness indices and residual strength factors are defined in Fig. 1. However, it has been shown (1,4) that, for this test to be reproducible between laboratories, great care must be taken to eliminate any "extraneous" deformations caused by deflections at the specimen supports or machine deformations. Indeed, in the round-robin test series reported in (1), it was found necessary to divide the tests into two groups: those which excluded the extraneous deformations, and those which included them.

In addition, it has recently been shown that the ASTM C1018 toughness indices are not independent of specimen geometry (3), unless the specimens are in geometrical similarity. The specimen size was found to influence not only the toughness, but also the stress and deflection at first crack, and the ultimate flexural strength. The toughness parameters decreased with an increase in the span-to-depth ratio. They also decreased with a decrease in the width of the specimen. The ASTM toughness indices also have been shown to depend upon the way in which the point of "first crack" is defined (4); when the load vs. deflection curves are examined carefully, it becomes clear that there is no unique definition of first-crack. First crack may be defined as the point at which the load reaches its first maximum point; it may be defined (as per ASTM C1018) as the point at which the load-deflection curve first becomes obviously non-linear; or it may be defined (4) arbitrarily as the point from which a series of 20 consecutive data points (over a total deformation of 0.01 mm or more) have a slope at least 5% less than the average slope of the load vs. deflection curve between 45% and 70 % of the peak load. These different definitions can lead to differences in computed ASTM C1018 toughness indices ranging from 10% at low fibre contents to about 35% at higher fibre contents. Thus, there are concerns about the

applicability of this test method for specifying FRC, or for carrying out quality control in the field.

A second commonly used method of evaluating FRC is that proposed in JSCE-SF4 (6). This test method uses the area under the load-deflection curve out to some specified deflection, i.e., the point at which the deflection reaches $1/150$ of the beam span (Fig. 2); a toughness and a toughness factor are defined. Unlike ASTM C1018, which is supposed to be independent of specimen dimensions, JSCE-SF4 clearly is dependent on the specimen geometry. This method has the considerable advantage that the precise shape of the load-deflection curve in the region surrounding the point of first crack is not very important in defining the toughness. (This should not be taken as an invitation to include extraneous deflections; these are specifically excluded in the test specifications.) However, this method is less sensitive than that prescribed in ASTM C1018 as to whether the extraneous deformations have been properly eliminated (and completely insensitive as to whether the first crack deflection has been properly chosen). For instance, Fig. 3 shows the load vs. deflection curves for three different FRC mixes, both including and excluding extraneous deformations. These three sets of curves show quite different behaviours. The calculated toughness parameters for these curves are shown in Table 1, for the mixes described in Table 2. It may be seen that T_{JSCE} and F_{JSCE} are essentially independent of how the deflections were measured; the various ASTM C1018 parameters, on the other hand, can vary considerably.

It has been suggested (1,3) that JSCE-SF4 is better able than ASTM C1018 to distinguish amongst FRC's containing different fibre types and different fibre volume fractions. This view, however, remains a controversial one (2,7).

In the round-robin tests reported here, six different machines in five different laboratories were used to carry out toughness tests on FRC, from which both the ASTM toughness parameters and the Japanese toughness and toughness factor could be determined. In all six test arrangements, great care was taken to eliminate extraneous deflections, so that at least some of the problems that arose in the previous round-robin tests (1) could be avoided.

EXPERIMENTAL PROCEDURES

Specimen Preparation

Unnotched specimens of dimensions $100 \times 100 \times 350$ mm were cast in perspex moulds and moist cured for about 30 days. Six different concrete