

Structural Applications of Fiber Reinforced Concrete

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SP-182



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PREFACE

Over the last 10 years, there has been a dramatic increase in our interest in fiber reinforced concrete, and much progress has been made not only towards developing a clearer understanding of the material itself, but also of developing new and innovative applications. With over 140,000 metric tons of fibers produced annually for concrete reinforcement, fiber reinforced concrete has finally emerged as a mainstream construction material of great industry-wide interest. Current major applications include floors, pavements, shotcrete, overlays, and precast products. Other rapidly developing applications include machine foundations, residential concrete, pile and pile caps, and impact buffers.

One area, often considered by many as the "last frontier," is the use of fiber reinforced concrete in truly structural applications. It has been clearly shown that a combination of fibers and reinforcing bar in structural elements produces a synergistic effect between the two forms of reinforcement, and fibers may in fact permit a more efficient use of the continuous reinforcement. The combination may not only improve the performance and durability, but may also enhance the constructability and reduce costs. Seismic applications are a good example. Fiber reinforcement in such cases is shown to result in a significantly enhanced structural ductility, joint integrity, and energy dissipation under repeated ground movements. Also, some of the congested shear reinforcement in structures with full seismic detailing may be replaced by fibers in concrete, resulting in a greater ease of construction and a reduction in cost.

Much further work remains to be done, however. There are still many areas where additional information is necessary. Test methods have to be developed to clearly and rationally characterize not only fiber reinforced concrete itself, but also the complex interaction between fibers and reinforcing bar. Finally, appropriate code provisions have to be developed. Without these codes, use of fibers in structural applications will remain only an activity of the research labs and academia.

Two sessions were held in 1997 during the ACI spring convention in Seattle, Washington, where papers on topics related to the use of fiber reinforcement in structural applications were presented. This special publication contains some of the papers, and assembles the thoughts of some leading researchers in this field. Authors have also identified areas where additional efforts are necessary for an efficient and rational use of the material, and for an accelerated acceptance of this material in real-life applications. Collectively, these writings are a snapshot of contemporary thinking in this field, and provide a clear direction for future activity.

It is hoped that this Special Publication will stir an active debate, and better still, result in novel applications. On our part, we would like to express our deep appreciation to each one of the authors for their very useful and timely contribution. Finally, our sincere thanks to the reviewers for their assistance, comments, and valuable advice. Our sincere thanks also to the American Concrete Institute for giving us this opportunity and making this Special Publication a reality.

Editors

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SP 182-1

High-Strength Fiber Reinforced Concrete Utilizing Closely Spaced Reinforcing Bars

by B. Aarup

Synopsis:

CRC (Compact Reinforced Composite) is the designation for a special type of fiber reinforced concrete with high strength (150-400 MPa) and closely spaced reinforcing bars. The dense matrix - with water/powder ratios of typically 0.16 - provide a good bond to fibers and reinforcing bars, and the large content of steel fibers provide the ductility necessary for utilizing reinforcement effectively. The steel fiber content is typically 2-6% by volume and the content of reinforcing steel is 2-10% by volume. The improved durability of the matrix - due to a high micro silica content - makes it possible to use a concrete cover to the reinforcement of only 10 mm in aggressive environments, improving the effectiveness of the reinforcement.

The CRC concept was developed in 1986 and aimed specifically for use in structures such as beams, columns and joints, but most of the applications so far have been in the security industry, for corrosion protection and in heavily loaded floors. However, in the last few years CRC has also been applied in structures.

One of these applications, production of 40,000 drain covers for a tunnel as a replacement for cast-iron covers, is described as an example of a project where the properties of high performance fiber reinforced concrete were utilised.

Keywords: fiber reinforced concretes; high-strength; precast; structural applications

Bendt Aarup (M.Sc., Civil Engineer) has been involved in cement and concrete research since 1988 at the Cement & Concrete Laboratory of Aalborg Portland A/S, PO Box 165, DK-9100 Aalborg. He has mainly worked with High Performance Concrete and Fiber Reinforced Concrete and is now manager of the CRC Marketing Department.

INTRODUCTION

High Strength or High Performance Concretes (HSC or HPC) are used increasingly for a range of structural applications and standards in a number of countries are being revised to accommodate these improved materials. As HSC are often more brittle than conventional concretes this can lead to the use of larger factors of safety for these materials and in order to provide these materials with improved ductility it is in some cases advantageous to incorporate fibers in the materials.

Fiber Reinforced Concretes (FRC) have mostly been used in non-structural applications such as slabs, floors, roads and architectural concrete, and typically less than 1% by volume of fibers are used. It is often prohibitive - for reasons of cost as well as in order to ensure acceptable workability and homogeneity - to include larger contents of fibers, but it has been tried with success in a few cases, usually by combining the advantages of fiber reinforcement and HPC, and in these cases other properties besides ductility can be improved as well (1). The high fiber contents have even been used in connection with considerable main reinforcement - as for Slurry Infiltrated Fiber CONcrete (SIFCON)(2) - or with prestressing - as for Reactive Powder Concretes (RPC)(3).

The improved ductility which can be obtained by using fiber reinforcement is necessary to utilize the high compressive strength which can be achieved in HSC in structures where tensile or bending loads are applied. If large contents of main reinforcement are used in brittle concretes, structures will exhibit large cracks at relatively small loads, whereas they will be able to show highly improved behavior if the necessary ductility is achieved. This has been the design philosophy behind Compact Reinforced Composite (CRC), which was developed at the Cement & Concrete Laboratory of Aalborg Portland in 1986 based on experience with DSP-concretes, a type of concrete developed in 1978 (4).

COMPACT REINFORCED COMPOSITE

CRC is the designation for a special type of Fiber Reinforced High Performance Concrete (FRHPC) with high strength (150-400 MPa). The matrix has a very large content of micro silica and water/powder ratios of typically 0.16. A typical composition is shown in table 1.

The binder contains cement, micro silica and a dry superplasticizer. As the cement and micro silica content is quite large there is considerable strength development after 28 days. The maturity curve also differs from that observed for conventional concrete as heat curing is more effective with CRC. One day at 80 °C corresponds to one year of curing at 20 °C.

The type of fibers used have a length of 12.5 mm, a diameter of 0.4 mm and a tensile strength of 1600 MPa. Other types of fibers have been used in earlier research projects with good results - especially brass coated fibers produced by Bekaert with 0.15 mm diameter, 6 mm length and a tensile strength of 2950 MPa. However, while a compressive strength of 180 MPa can easily be achieved with these fibers the price is so high that it would only be realistic to use these fibers for very special applications.

The combination of micro silica, a very low water content and steel fiber contents of 2-6% by volume provides CRC with a good bond as well as with a large ductility and has the effect of ensuring crack-free behavior in e.g. beams at tensile strains of up to 3 mm/m, more than 10 times the strain capacity of conventional concrete.

As an example of the behavior in bending a load-deformation curve for a CRC beam with 10% by volume of main reinforcement is shown in fig. 1. In this case prestressing wires with a yield strength of 1800 MPa have been used as main reinforcement. The wires have not been prestressed, but have been used as conventional reinforcement in order to utilize the high quality of the steel. Even at the very large deformations - 60 mm center deflection on a span of less than 2 meters - only very small cracks were observed. At this stage of loading the bending stress was 320 MPa.

One key factor in utilizing the high quality matrix in FRHPC is to be able to use large contents of reinforcement - and also reinforcement of higher quality - without experiencing large cracks. Another important factor is to be able to place the reinforcement to full advantage. In CRC the size of the fibers and the largest grains of the CRC matrix dictate the distance between reinforcing bars and the cover layer to the reinforcement. This is the reason for typically using a mortar composition for CRC. Often a cover layer of

10-15 mm and a similar distance between individual bars are used. Especially the durability has been the subject of a number of research projects because of these small cover layers, but it has been demonstrated on a number of occasions that 10 mm of cover layer will provide sufficient protection of the reinforcement in an aggressive environment (5).

With the high fiber contents CRC is most suitable for pre-cast applications, but in-situ cast concrete with 6% by volume of fibers has also been produced - for joints between slabs in conventional concrete - using a poker vibrator.

As the properties of FRHPC are not taken into account in existing standards and recommendations, it has been necessary to provide extensive documentation on the properties of CRC, and the material has been the subject of a number of research projects. These have often been carried out in cooperation with universities in Denmark, but CRC has also been the subject of international research projects, such as COMPRESIT, a EUREKA project with partners from Denmark and England and sponsored by the two national Departments of Trade and Industry (6), and MINISTRUCT, a BRITE/EURAM project with partners from Denmark, France and Spain and sponsored by the European Commission (7).

APPLICATIONS

CRC was developed with applications in bridges and tall structures in mind, but some of the first applications of this combination of large fiber contents and closely spaced reinforcement were non-structural, as the concept was used in the security industry, for machine components and in heavily loaded floors where limited space was available.

It was anticipated that CRC could be used as a replacement for steel or cast-iron due to the favorable strength/weight ratio of the material, and this had also been the subject of the first international research project on CRC - COMPRESIT - in which design, production and testing was made of segmental tunnel-linings to be used as replacement for cast-iron linings. The project was concluded in 1993, at the same time as a major infrastructure project was carried out in Denmark - a joint bridge-tunnel project linking the two major islands of Denmark under the name of the Great Belt Link. As problems were anticipated with availability of the cast-iron linings, an inquiry was made to Aalborg Portland as to the availability of CRC-linings. This contact did not lead to the use of CRC-linings as the problems with availability of cast-iron were solved and the designers could maintain the original design, but the contact lead to the application of CRC in another

part of the two tunnels, a part which may seem of limited significance in the larger scale, but a part which was nevertheless well suited as a demonstration of the advantages of FRHPC, and an application which will be used as an example in the following in an attempt to describe the properties which can be of interest to the designers.

DRAIN COVERS FOR THE GREAT BELT LINK

The covers placed over the drain channels in the Great Belt tunnels were originally designed in cast-iron, but it was discovered that this could cause problems with stray currents for the electrical installations in the tunnel. The drain channels were an integrated part of the lining design, so the dimensions of the covers were fixed with a thickness of 40 mm and a span of 500 mm. As the covers were to be placed under the rails, they had to sustain an ultimate static load of 47 kN/m which would necessitate the use of reinforcement. The small thickness, however, was not compatible with the requirement for a cover of at least 45 mm to ensure a design life of 100 years in the aggressive environment.

A cross-section of the design made to suit these criteria is shown in fig. 2. The covers have the dimensions 590x412x40 mm. The reinforcement - 8 mm bars with a specified yield strength of 550 MPa - is closely spaced and the cover to the reinforcement is 10 mm. Compressive strength of the matrix was 150 MPa with 6% of fibers with dimensions 12.5x0.4 mm.

With regard to resistance to intrusion of chloride ions, measurements had been made on specimens under load exposed to chlorides, and these investigations show that the diffusion coefficient for chloride ions measured after two years of exposure under accelerated conditions was only $8 \times 10^{-15} \text{ m}^2/\text{s}$, in which case corrosion should not be a problem. Tests performed later indicate that the chloride threshold value is also considerably higher for these very dense concretes.

Production of a total of 40,000 covers were carried out by a pre-cast manufacturer and the covers were installed in the tunnel in the spring of 1995. As the design was not exactly according to the current standards, it was decided to adhere to a rather strict program of quality control, which included testing of 1% of the covers produced in three-point bending after 7 days, after which a control was made of the cover to the reinforcement. Also compressive strength was measured on 100x200 cylinders cured at 80 °C for one day. Average compressive strength of the reference specimens tested during the production was slightly above 150 MPa, while average strength of the more than 400 drain-covers tested was 216 kN/m. This

strength was considerably higher than necessary for the static case, but the high strength was used to ensure the capacity in fatigue as well, as 3.2×10^8 load cycles had to be considered in the design life of 100 years.

DISCUSSION

The application of CRC for the drain-covers of the Great Belt Link is an example of an application of FRHPC where properties of durability and fatigue as well as bending strength have been utilized.

Even though the properties of FRHPC are well documented for a number of compositions, the first applications are probably going to be similar to this case, where CRC was considered due to a problem encountered after the design was made - for trouble-shooting. Another solution for the owner could have been to electrically isolate the cast-iron covers. However, the risk of using a new material in this application was moderate, as the design allowed for full-scale testing of the individual covers, a strict quality control was maintained and failure of the covers would - while very costly - not lead to dangerous situations. Finally, the cost of the CRC covers was only about 60% of the cost for the cast-iron covers which could also have been of some significance in the considerations.

In the last few years CRC has been used for other structural applications, mostly as pre-cast components such as balcony slabs, manhole covers and facade elements, and as the components are increasing in size the results from these applications - combined with results from the joints which are cast on-site and results from applications of other types of FRHPC such as SIFCON, SIMCON and RPC - should provide designers with the references necessary for considering the use in columns and large beams where the properties of FRHPC can also be utilised.

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TABLE 1—TYPICAL CRC COMPOSITION AND PROPERTIES MEASURED AT 28 DAYS.

binder	940 kg/m ³
quartz sand 0-0.25 mm	170 kg/m ³
0.25-1 mm	340 kg/m ³
1-4 mm	680 kg/m ³
water	150 kg/m ³
steel fibers	475 kg/m ³
compressive strength [MPa]	140
bending strength [MPa]	20
Young's modulus [GPa]	47
density [kg/m ³]	2750

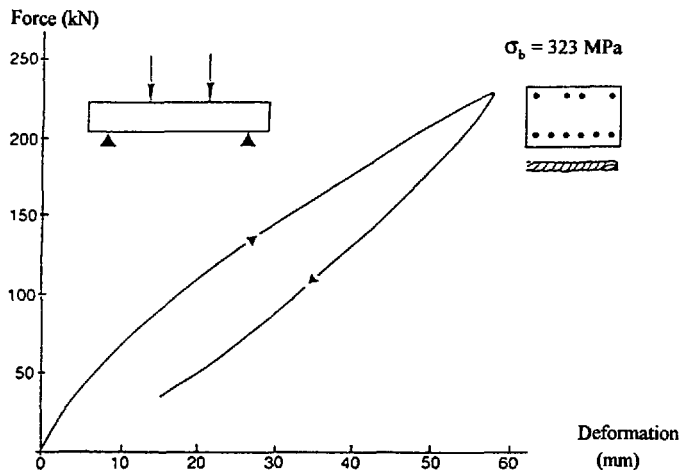


Fig. 1—Load-deformation behavior of CRC beam with prestressing wires as main reinforcement.

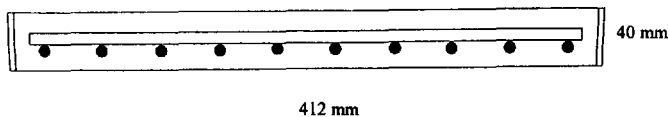


Fig.2—Cross section of CRC drain covers used for the Great Belt Link.

SP 182-2

Structural Behavior of Steel Fiber Reinforced Concrete Beams in Shear

by B. Oh, D. Lim, K. Hong, S. Yoo and S. Chae

Synopsis: The structural behavior of steel fiber reinforced concrete beams in shear is studied. A comprehensive experimental program has been set up and several series of reinforced concrete beams with steel fibers have been tested. The test variables include the volume contents of steel fibers and stirrups. The fiber contents varies from 0% to 2% by volume. It is seen from these tests that the cracking and ultimate shear strengths increase as fiber contents increase. The present study indicates that fiber reinforcement can reduce the amount of shear stirrups to obtain same strength. The combination of fibers and stirrups may accomplish strength requirements as well as ductility requirements.

A theoretical approach is proposed to predict the shear strength of reinforced concrete beams containing steel fibers and good correlation is obtained with test data. The present study allows more efficient structural application of steel fibers for shear reinforcement in reinforced concrete structures.

Keywords: cracking shear strength; fiber reinforced concrete; shear analysis; shear reinforcements; steel fibers; ultimate shear strength

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INTRODUCTION

The brittle nature of concrete causes collapse in unreinforced beams shortly after the formation of the first crack. The addition of steel fibers aids in converting the brittle characteristics to a ductile one. The principal role of fibers is to resist the formation and growth of cracks by providing pinching force at the crack tip. In addition, a marginal improvement in tensile strength also results and fiber reinforced concrete have higher ultimate strain than plain concrete (1-12).

Many studies have been conducted to determine the flexural behavior of steel fiber reinforced concrete beams. The authors recently published a paper concerning the flexural analysis of reinforced concrete beams containing steel fibers as Ref.1. However, only a few studies (5,10,11) are available on the shear behavior of reinforced concrete beams with steel fibers and the shear behavior of those beams is not well established yet. The purpose of the present study is therefore to explore the mechanical behavior of steel fiber reinforced concrete beams in shear for structural

application.

The tests were carried out on concrete beams reinforced with stirrups and steel fibers. The purpose of the present study is (1) to examine the effect of fiber addition on the shear behavior of reinforced concrete beams, (2) to study the potential use of fibers to replace the stirrups, and (3) to investigate effective combinations of stirrups and steel fibers for improvement of ultimate and cracking shear strengths as well as ductility. Based on the present test results and earlier published studies, a method of predicting ultimate shear strength for beams reinforced with both stirrups and steel fibers is proposed.

RESEARCH SIGNIFICANCE

The addition of steel fibers in reinforced concrete beams results in a substantial increase in strength and ductility. These phenomena have been verified for fiber reinforced concrete beams in flexure(1). The behavior of reinforced concrete beams containing steel fibers under shear loading is not relatively well established. Moreover, quantitative amounts of shear resistance due to steel fibers must be clarified in order to determine the shear capacity of fiber reinforced concrete beams.

The purpose of the present study is to investigate a possibility of optimum combination of steel fibers and shear stirrups to achieve the strength requirement as well as ductility requirement. To accomplish this goal, a comprehensive experimental study was planned and conducted in the present study.

DIMENSION OF TEST MEMBERS

The test program here consists of testing several series of reinforced concrete beams having identical rectangular cross sections of 100mm×180mm. The span length of the members is 1300mm and the shear span length is 400mm. The effective depth of the beam is 150mm. A total of 9 beams were tested to investigate the influence of fiber reinforcement on the mechanical behavior of reinforced concrete beams in shear. The major test variables are the amount of steel fibers and the volume of shear stirrups. The fiber contents were varied from 0% to 2% of the concrete total volume and the ratios stirrups from 0% to 100% of the code-required values. Table 1 describes the details of the beams and the major test variables.

The present tests are to examine the influence of fiber addition on the shear behavior of reinforced concrete beams and to determine the effectiveness of steel fibers on the shear resistance.

MATERIALS AND FABRICATION OF TEST MEMBERS

The ordinary portland cement(TYPE I) was used and the pea gravel with maximum aggregate size of 10mm was used. The sand fineness modulus is about 2.21. The round straight steel fibers with 0.7mm diameter, 42mm length and ultimate strength of 1784MPa are used. The aspect ratio of steel fiber is 60. The material properties of steel fibers are summarized in Table 2. The longitudinal deformed steel, 16mm diameter(tensile steel), 10mm diameter(compressive steel) with yield strength of 420MPa and 6mm diameter steel for stirrups were used. The actual yield stress of stirrups was 359 MPa.

The mix proportion of concrete of test members is listed in Table 3. The mix was designed to obtain the compressive strength of 35MPa at 28days. The aggregates, cement, and water were batched by weight and mixed in a drum mixer. The fibers were introduced last and dispersed uniformly. Although the fibrous mixes were less workable than plain concrete, the mix procedures proved satisfactory in that the dispersion of fibers was found to be uniform and there was no significant fiber balling. The slump value was about 12cm. The $100\phi \times 200$ mm concrete cylinder specimens for compression and split-tensile test and $100 \times 100 \times 400$ beam specimens mold for flexural tests were also cast from the same mix as were the corresponding.

All test beams were vibrated by a 25 mm diameter internal poker vibrator until satisfactory compaction was achieved. Following casting, the concrete beams and control specimens were covered with wet burlap. All specimens were stripped from their molds after 24 hours and then stored in a curing water tub.

TEST PROCEDURES AND MEASUREMENTS

The four-point loading method has been applied to the test beam as two equal concentrated loads by means of a steel spread beam (see Fig.1). A calibrated load cell was placed between the jack and spread beam. Three dial gages with 0.01mm accuracy were used to measure the deflections under increasing loads at several locations. Steel strain gages were attached on longitudinal steel and stirrups, and concrete strain gages were attached for searching concrete strains and principal stress direction. Locations of measuring sensors are shown in Fig.1. The loads were applied step by step in small increments up to the ultimate load. The amount of incremental load at each step was 5 kN(500kg). All the measuring values were automatically scanned and stored in the computer at each loading step. These test results were analyzed later by the computer.

TEST RESULTS AND DISCUSSIONS

Strength Characteristics and Modes of Failure

Compressive strength--The compressive strengths of test cylinders were measured for various cases of fiber contents. Fig. 2 shows the relative strength of concrete in compression due to the addition of steel fibers. The strength increase was about 25% when fibers are introduced into concrete up to 2% by volume. The actual strength values are summarized in Table 4.

Flexural strength--The present test indicate that the flexural strength of fiber reinforced concrete is greatly enhanced due to the addition of steel fibers. The increase of flexural strength was about 55% when the fiber content was increased to two percent (see Fig. 3). One more important feature in flexural behavior is that the fiber concrete showed great ductility and energy absorption capacity.

Splitting tensile strength--Figure 4 shows the increase of tensile strength due to steel fibers. It can be seen that splitting tensile strength was increased more than twice at 2-percent fiber volume. Figure 5 shows a comparison of various relative strengths for various fiber contents. The relative strength indicates the normalized value to the strength of plain concrete without fibers. It can be seen from this figure that the rate of strength increase due to fiber addition is greatest in splitting tensile strength. This means that the steel fiber greatly enhances the tensile properties of concrete and improves the resistance to cracking.

Modes of failure--All the beams exhibited similar linear behavior from initial loading up to the occurrence of first hair-line crack. The beam S0V0, which is without shear reinforcement, failed soon after the formation of the diagonal crack. The cracks initiated and progressed along the compressive stress trajectories in the shear span and led very rapidly to a catastrophic type of failure. On the other hand, the beams with fiber reinforcement (beams S0V1 and S0V2) continued to resist higher shear stress and a considerable ductility was observed. As shown at the failure modes of the S0-beam series, the mode failure changed from shear to flexural types as fibers contents increase. The ductility is also enhanced greatly with the addition of fibers. It can be also seen that the beams with 50% of conventional code-required stirrups and 1% of steel fiber contents had a flexural type of failure. The modes of failure are shown at Fig. 6. It can be seen from this figure that fiber contents of 1% is critical point of transferring failure modes (from shear mode to flexural mode). In beams with stirrups(S0.5V0, S0.75V0, S1V0), some spalling occurred at ultimate stage. But the inclusion of steel fibers eliminated the spalling of concrete : steel fibers enhanced the capacity to hold parts of concrete together in post cracking stage, thus preventing the spalling even at ultimate failure.

Load-Deflection Characteristics

The load-deflection curves for the test beams are shown at Fig. 7-9. No displacements were observed at supports since the supports were fixed. From these figures, it can be seen that all the beams exhibit linear behavior from initial loading up to the occurrence of first diagonal crack. After formation of cracks, all the beams reveal nonlinear load-deflection characteristics.

Comparison of peak deflections at ultimate loads reveals that there was great improvement in the ductility due to use of fibers. Beams without stirrups exhibit much improvement in the ultimate strength (Fig. 7), but beams with stirrups (Fig. 8,9) exhibit small improvement in ultimate strength due to fiber addition compared with the beam without stirrups. These phenomena can be clearly seen from Fig. 7 - Fig. 9.

Cracking Shear Strength

One of predominant effects due to steel fibers is the increase of cracking shear strength. Generally, cracking shear strength of fiber reinforced concrete is higher than that of conventional reinforced concrete.

Fig. 10 shows cracking shear strength as it varies with fiber contents. From these test results, it can be seen that fiber inclusion increases cracking shear strength greatly. This means that, for the increase of cracking shear strength and ductility, the addition of steel fibers is better than an increase of stirrups.

Ultimate Shear Strength

Fig. 11 shows ultimate shear strength according to fiber contents for each beam. From this figure, it can be seen that fiber reinforced concrete beams exhibit much improvements in ultimate shear strength compared with the companion beams without fibers. The increase of shear strength due to fibers becomes larger for the beams with lower shear stirrups. This means that the steel fibers play a much more role for lightly reinforced conventional beams.

It is found from Fig. 11 that the combination of 75% of full conventional stirrups with 1% steel fiber volume contents (S0.75V1) gives the same ultimate shear strength as the conventional beam (S1V0), and that the combination of 50% of full conventional stirrups with 2% fiber contents (S0.5V2) also gives almost similar shear resistance to the beam with full stirrups without fibers (S1V0).

From these test results, it can be concluded that fiber reinforcement can reduce the needed amount of shear stirrups and

that the combination of fibers and stirrups may accomplish the strength requirement as well as the ductility requirement.

THEORETICAL PREDICTION OF SHEAR STRENGTH FOR STEEL FIBER REINFORCED CONCRETE BEAMS

Fig. 12 shows the free body diagram of parts of the shear span of a simply supported fiber reinforced concrete beam. From this figure, the total shear forces, V_u , can be written as follows,

$$V_u = V_c + V_{ay} + V_d + V_{fy} \quad (1)$$

Where, V_c : shearing force across the compression zone resisted by concrete,
 V_{ay} : aggregate interlocking force (vertical component),
 V_d : dowel action force,
 V_{fy} : vertical component of fiber pull out force along the inclined crack

Considering the shear resistance of concrete without web reinforcement, V_{uc} , the equation of ultimate shear strength can be written as follows (13),

$$V_{uc} = (10 \rho f_c' \frac{d}{a_s})^{\frac{1}{3}} b d \quad \text{for } \frac{a_s}{d} \geq 2.5 \quad (2)$$

$$V_{uc} = (160 \rho f_c')^{\frac{1}{3}} (\frac{d}{a_s})^{\frac{4}{3}} b d \quad \text{for } \frac{a_s}{d} \leq 2.5 \quad (3)$$

Where, ρ =reinforcement ratio, f_c' =compressive strength of concrete(MPa), d =effective depth(m), b =width of the beam(m), a_s =shear span(m).

However, as our tests conducted on reinforced concrete beams with web reinforcements, one must consider the contribution of the stirrups, $V_{us}=A_v f_y d/s$. Thus the equation of ultimate shear strength must be written as $V_u=V_{uc}+V_{us}$, i. e.,

$$V_u = (10 \rho f_c' \frac{d}{a_s})^{\frac{1}{3}} b d + \frac{A_v f_y}{s} d \quad \text{for } \frac{a_s}{d} \geq 2.5 \quad (4)$$

$$V_u = (160 \rho f_c')^{\frac{1}{3}} (\frac{d}{a_s})^{\frac{4}{3}} b d + \frac{A_v f_y}{s} d \quad \text{for } \frac{a_s}{d} \leq 2.5 \quad (5)$$

Where, A_v =area of stirrups(m²), f_y =yield strength of stirrups(MPa), s =spacing of stirrups(m).

To estimate the contribution of the steel fibers to the shear resistance, the neutral axis depth (c) has to be calculated first. The external moment ($V_u \cdot a_s$) must be equal to internal ultimate resisting moment (M_u).

$$M_u = 0.85f_c' ab(d-a/2) + A_s' f_s' (d-d') \quad (6)$$

Where, A_s' =area of compressive steel, f_s' =compressive steel stress, and d' =cover of compressive steel.

From Eq.(6), one can determine rectangular stress block depth "a", and the neutral axis depth is calculated as $c=a/\beta$.

Referring to Fig. 12, the length of the inclined shear crack is equal to $(h-c)/\sin\alpha$, and the area through which the steel fibers contribute to shear resistance of the beam is $b(h-c)/\sin\alpha$. The number of fibers, n , crossing a unit area of the crack may be taken as (5,10)

$$n = 0.5 \frac{V_f}{\pi r_f^2} \quad (7)$$

Where, V_f =volume fraction of fiber, r_f =radius of fiber.

At failure, fiber pull-out invariably occurs, since the fiber length(l_f) is usually less than the critical length necessary to develop the ultimate tensile strength of fiber, and also due to the relative displacement of two faces of the crack. Since failure is by fiber pull-out, it has been shown (4,5) that the mean fiber pull-out length is $l_f/4$. The average pull-out force F_0 for a fiber is then given by ;

$$F_0 = \tau \pi d_f \frac{l_f}{4} \quad (8)$$

Where, τ =average bond strength which is defined as half the ultimate bond strength, i. e., $\tau_u/2$, where $\tau_u=4.15\text{N/mm}^2$ given by Swamy et. al. (3,4).

The ultimate force sustained by steel fibers per unit area of crack at failure is therefore given by ;

$$F_1 = nF_0 = n \tau \pi d_f \frac{l_f}{4} \quad (9)$$

Substituting Eq.(7) into Eq.(9), the following equation is obtained for F_1 ,

$$F_1 = 0.5 \tau V_f \frac{l_f}{d_f} \quad (10)$$

The total force(F_t) perpendicular to the crack is therefore ;

$$F_t = F_1 \{ b(h-c) / \sin\alpha \} \quad (11)$$

The vertical component of this force is equal to the increase in shear resistance of the beam due to the presence of steel fibers and

is equal to ;

$$V_{fy} = F_t \sin \alpha = F_1 b(h - c) \quad (12)$$

Finally, the total shear strength of fiber reinforced concrete beam can be calculated as the sum of Eq.(4) and Eq.(12). [or Eq.(5) and Eq.(12)]

$$V_u = V_{uc} + V_{us} + V_{fy} \quad (13)$$

or Eq.(13) can be written in stress form as

$$v_u = \frac{V_u}{bd} \quad (14)$$

COMPARISONS OF THE THEORY WITH TEST RESULTS

Table 4 and Fig.13 represent the results of the comparison of the observed ultimate shear strength v_{uo} with the predicted ultimate shear strength v_{up} given by this analysis method. The predicted values using this method give acceptable results.

The mean value of the ratio of the observed ultimate shear strength to predicted ultimate shear strength for test beams was about 0.92 with a standard deviation of 0.08 and coefficient of variation of 8.7 percent.

CONCLUSIONS

The structural behavior of steel fiber reinforced concrete beams subjected to shear is studied. A series of beams have been tested to investigate the influence of fiber reinforcement on the mechanical behavior of reinforced concrete beams in shear. The major test variables are the amount of steel fibers and the volume of shear stirrups. The fiber contents were varied from 0% to 2% by volume and the amount of stirrups varied from nothing to full reinforcement of code-required values.

From the present experimental and analytical study on the shear behavior of steel fiber reinforced concrete beams, the following conclusions may be drawn.

1.The addition of steel fibers increases compression strength, flexural strength and splitting tensile strength, with the rate of strength increase greatest in splitting tensile strength. This means that the steel fibers greatly enhance the tensile properties of concrete and improve the resistance to cracking.

2. The cracking shear strength increased significantly due to the

addition of fibers, and the increase of the strength was about 100% when the fiber content was increased from 0% to 2%. Thus, the addition of fibers is effective in controlling shear cracks.

3. The mode of failure of conventional reinforced concrete beam underreinforced in shear changed from shear failure to flexural failure types when the volume fraction of fibers exceeds 1%. This means that fiber reinforcement increases shear capacity greatly.

4. The use of steel fibers can reduce the amount of shear stirrups needed, and a combination of fibers and stirrups may accomplish strength requirement as well as ductility requirement. It is found that either the combinations of 75% of full conventional stirrups and 1% fiber volume content or 50% of full conventional stirrups and 2% fiber content may be good optimized combinations for shear ductility and strength.

5. A theoretical method to predict the shear strength of reinforced concrete beams containing steel fibers is presented and good comparisons are made with test results. The method will allow more realistic shear analysis of reinforced concrete structural members containing steel fibers.

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TABLE 1—TEST BEAM DETAILS AND TEST VARIABLES.

Identification of beams	Stirrup		Fibers		f'_c (MPa)	f_{sp} (MPa)
	Spacing (mm)	ρ_v (%)	Aspect ratio (ℓ/d)	V_f (%)		
S0V0	-	0.000(0%)*	60	0	34.0	2.5
S0.5V0	80	0.007(50%)*	60	0	34.0	2.5
S0.75V0	60	0.009(75%)*	60	0	34.0	2.5
S1V0	40	0.014(100%)*	60	0	34.0	2.5
S0V1	-	0.000(0%)*	60	1	38.7	4.0
S0.5V1	80	0.007(50%)*	60	1	38.7	4.0
S0.75V1	60	0.009(75%)*	60	1	38.7	4.0
S0V2	-	0.000(0%)*	60	2	42.4	5.1
S0.5V2	80	0.007(50%)*	60	2	42.4	5.1

NOTE : $\rho_v = A_v/b_s =$ shear reinforcement ratio. * The values in parenthesis represent the percent ratio of shear stirrups to the code-required full shear reinforcements.

TABLE 2—MATERIAL PROPERTIES OF STEEL BARS AND STEEL FIBERS.

	Yield strength (MPa)	Tensile strength (MPa)	Modulus of elasticity ($\times 10^5$ MPa)
Longitudinal Steel	420	545	2.0
Stirrup	359	534	2.0
Steel fiber	1,303	1,784	2.0

TABLE 3—PROPORTIONS OF MIX DESIGN.

	Cement	Water	Sand	Aggregate
Contents	450	171	707	1,060

(kg/m³)

TABLE 4—COMPARISON OF THE OBSERVED ULTIMATE SHEAR STRENGTH V_{uo} WITH THE PREDICTED ULTIMATE SHEAR STRENGTH V_{up} FOR VARIOUS TEST SERIES.

Identification of beams	Stirrup		Fiber		f'_c	f'_{sp}	V_{cr}	V_{uo}	V_{up}
	Spacing (mm)	$\rho_v(\%)$	ℓ/d	$V_f(\%)$	(MPa)				
S0V0	—	0.000(0%)*	60	0	34.0	2.5	0.78	3.56	3.38
S0.5V0	80	0.007(50%)*	60	0	34.0	2.5	1.09	5.35	5.34
S0.75V0	60	0.009(75%)*	60	0	34.0	2.5	1.18	6.36	6.30
S1V0	40	0.014(100%)*	60	0	34.0	2.5	1.25	7.00	7.77
S0V1	—	0.000(0%)*	60	1	38.7	4.0	1.49	4.49	4.98
S0.5V1	80	0.007(50%)*	60	1	38.7	4.0	1.69	5.73	7.00
S0.75V1	60	0.009(75%)*	60	1	38.7	4.0	1.84	7.00	7.70
S0V2	—	0.000(s 0%)*	60	2	42.4	5.1	1.81	5.73	6.36
S0.5V2	80	0.007(50%)*	60	2	42.4	5.1	2.60	6.80	8.50

Note : f'_c =compressive strength, f'_{sp} =splitting tensile strength, $v_{cr}=V_{cr}/bd$ =cracking shear strength, $v_{uo}=V_{uo}/bd$ =observed ultimate strength, $v_{up}=V_{up}/bd$ =predicted ultimate shear strength, $\rho_v=A_v/bs$ =shear reinforcement ratio. * The values in parenthesis represent the percent ratio of shear stirrups to the code-required full shear reinforcements.

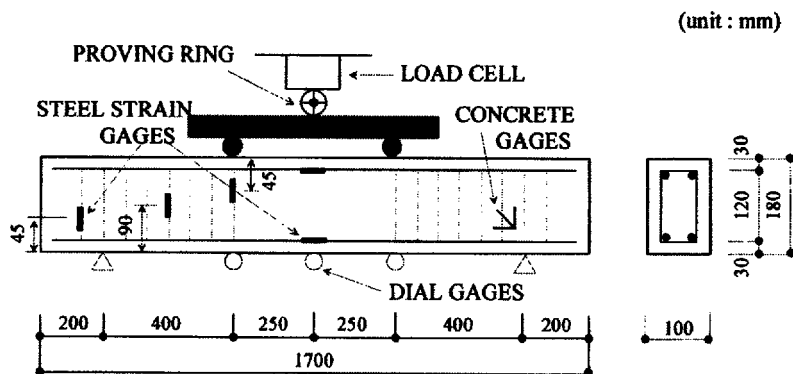


Fig. 1—Test setup and the locations of strain gages installed.

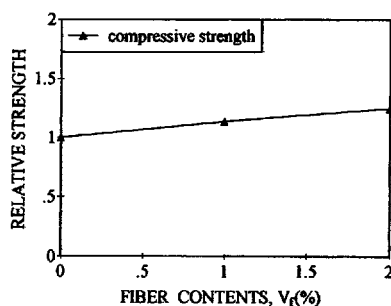


Fig. 2—Relative compressive strength of concrete due to the addition of steel fibers.

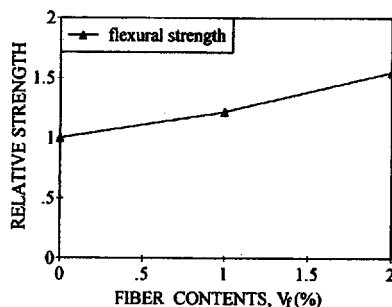


Fig. 3—Relative flexural strength of concrete due to the addition of steel fibers.

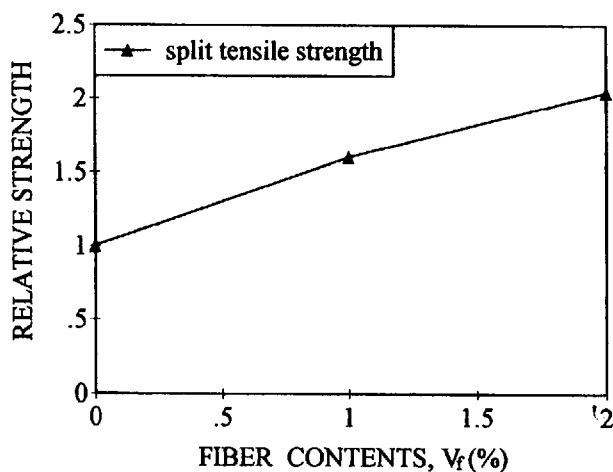


Fig. 4—Relative splitting tensile strength of concrete due to the addition of steel fibers.

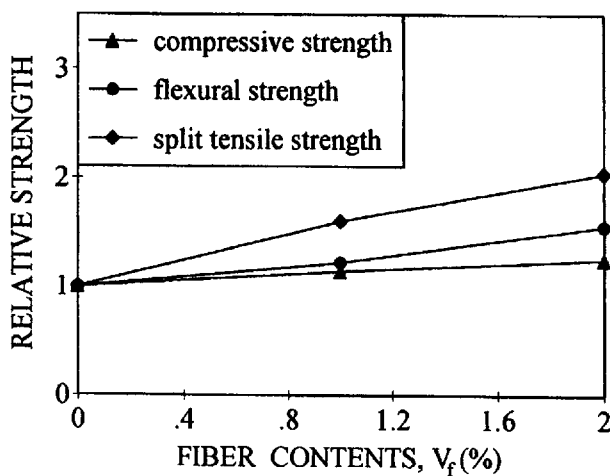


Fig. 5—Comparison of various relative strength of concrete due to the addition of steel fibers.

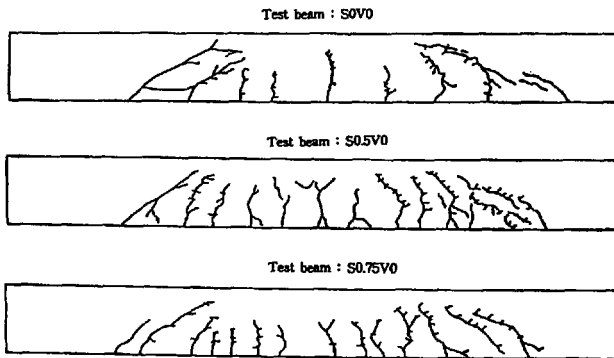


Fig. 6a—Failure modes (cracking pattern) of test beams.

Test beam: S0V0—Test beam: S0.5V0—Test beam: S0.75V0

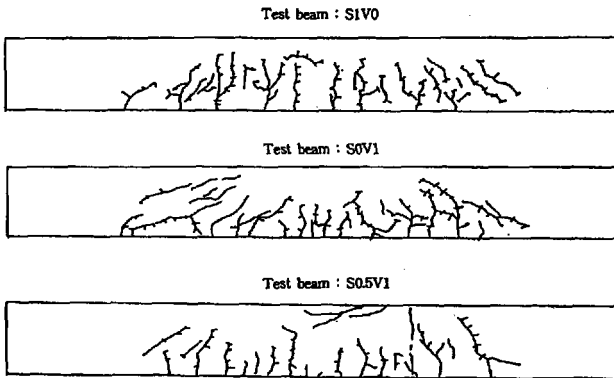


Fig. 6b—Test beam: S1V0—Test beam: S0V1—Test beam: S0.5V1

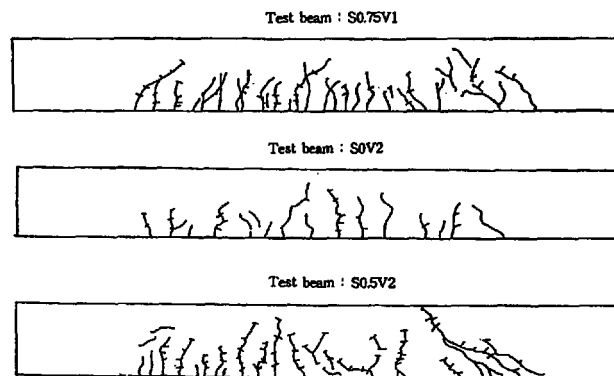


Fig. 6c—Test beam: S0.75V1—Test beam: S0V2—Test beam: S0.5V2

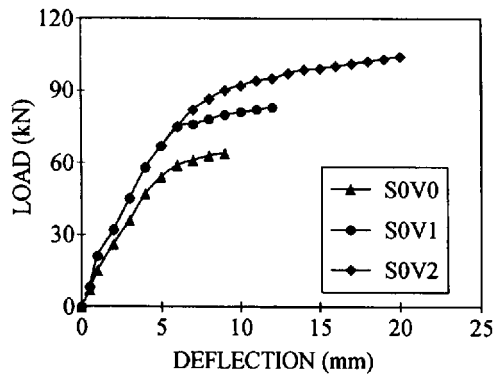


Fig. 7—Load-deflection curves for the beams without stirrups.

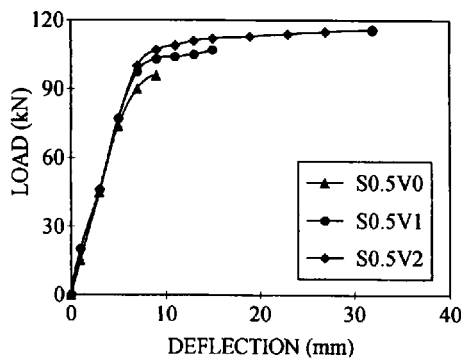


Fig. 8—Load-deflection curves for the beams with 50 percent full conventional stirrups according to fiber contents.

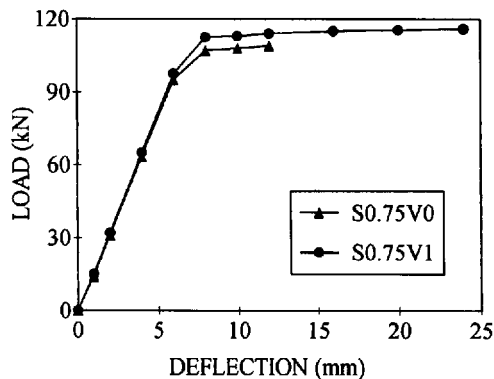


Fig. 9—Load-deflection curves for the beams with 75 percent full conventional stirrups according to fiber contents.

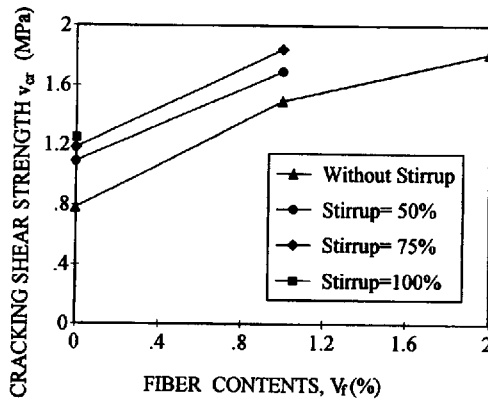


Fig. 10—Cracking shear strength according to fiber contents.

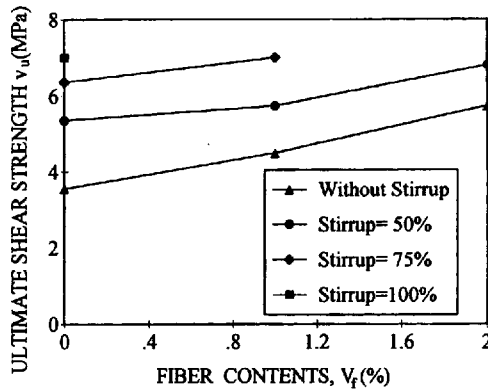


Fig. 11—Ultimate shear strength according to the fiber contents.

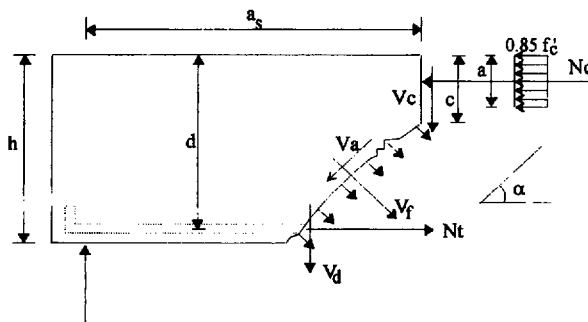


Fig. 12—Free body diagram of a part of shear span of a simply supported fiber reinforced concrete beam.

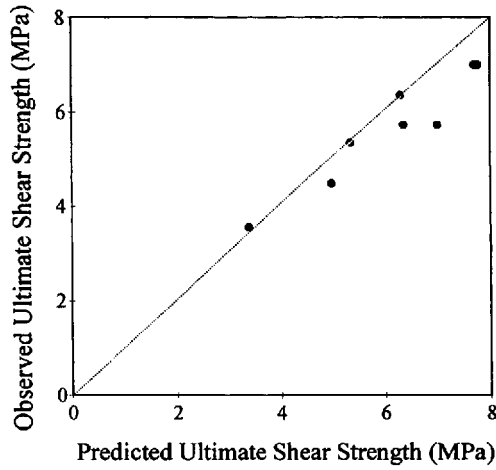


Fig. 13—Comparison between the observed ultimate shear strength V_{uo} with predicted ultimate shear strength V_{up} .

SP 182-3

Fiber Reinforced High-Performance Concrete Beams: Material and Structural Behavior

by M. Marazzini and G. Rosati

Synopsis: The mechanical behavior of a few plain and fiber-reinforced high-performance concretes ($f_c=80\text{-}130$ MPa) is studied here by means of direct tensile tests and three-point bending tests, and a special “identification” procedure is adopted in order to cleanse the stress-strain and stress-displacement curves of any undesired structural effect. Then the overall behavior of two P/C beams typifying the sub-elements of a hollow-core slab is examined, with and without fibers, to study crack formation and propagation (by optical interferometry) and structural ductility.

Keywords: bond; concretes; cracking; hollow-core slabs; steel fibers; structural ductility; tensile behavior; test methods (concrete)

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INTRODUCTION

In many engineering fields, the study of composite materials is now an important topic [1,2]. Among these materials, high performance cementitious composites are specifically suited to Civil Engineering, owing to their durability, abrasion resistance, early strength and volume stability [3]. Compared to normal concretes, high-performance concretes are more expensive and may increase the initial cost of a structure, but the extra cost is generally more than offset by their better long-term behavior (i.e. better durability with lower maintenance costs and longer service life), more affordable environmental impact and lower structural weight (since the greater strength leads to a generalized size reduction) [4].

Unfortunately, high-performance concretes containing silica fume (high-strength and ultra high-strength concretes and microconcretes) are very brittle compared to normal concretes, but this drawback can be overcome by introducing medium-to-high fiber contents into the mix [6] ($v_f > 1\%$ by volume, and typically $v_f \geq 4\%$, steel fibers).

The added toughness of the material increases the structural ductility, which is instrumental in preserving the overall performances of a structure, especially where and when unrepairable and irreplaceable members are in issue (e.g. bridge pylons and underwater tunnel liners).

To obtain long-term durability, concretes having very low permeability are required, and since concrete permeability is related to pore size and distribution, diffusion and transport processes must be fully understood.

In this paper the mechanical behavior of a few plain and fiber-reinforced high-performance concretes ($f_c = 80\text{--}130$ MPa) is studied by means of direct tensile tests and three-point bending tests, and a special "identification" procedure is adopted in order to cleanse the stress-strain and stress-displacement curves of any undesired structural effect. Then the overall behavior of two P/C beams is examined, with and without fibers, to study crack formation and propagation (by optical interferometry) and structural ductility. These precast, prestressed beams typify the single sub-elements of the hollow-core P/C slabs that are often found in medium-span bridge decks, where the durability and fatigue resistance provided by HPCs and FRCs are of great importance.

MATERIALS, INSTRUMENTATION AND TEST SETUP

The scope of the tensile and bending tests carried out by the authors on ultra high-strength concretes ($f_c = 130\text{--}150$ MPa) was the characterization of the material properties for different fiber contents. The mix-design is reported in Table 1. Carbon steel microfibers, diameter 0.15 mm, length 6 mm, were used.

The mechanical properties refer to a curing period in water of 28 days.

The mix-design of the two prestressed beams, built in a precast-concrete factory, are given in Table 2. The first beam was made of a typical normal concrete for prestressed elements ($f_c = 46.9$ MPa). The second beam was made of a fiber-reinforced concrete with a fiber content of 1% by volume ($f_c = 81$ MPa, steel fibers). The tendons were tensioned before casting at a stress level of 1250 MPa. The tests were carried out one week after casting.

To carry out the tensile and bending tests, a suitably-modified 100 kN Instron machine was used. The machine is provided with an electromechanical actuator, with a minimum speed of 2 μm per hour; among the three control channels, one gives the possibility of choosing the feedback signal, in order to allow a stable control of the test; the closed-loop control with integral and derivative gain, makes it possible to remove the effects of the finite axial stiffness of the machine, so as to avoid unstable failures.

In the tensile tests, the displacements were measured using two LVDT's, having a sensitivity of 0.2 μm , set across the notch, with a 50 mm base and arranged at 180° around the specimen axis (Fig.1: the average value was used as feedback signal for machine control). Furthermore, the relative displacement of the machine heads was measured at four points, by means of four LVDT's. In the part outside the notched zone, two electrical strain gauges were set at two diametrically-opposite points.

The original machine was stiffened by adding four adjustable instrumented rods, connecting the lower platen of the actuator with the fixed cross head. The high-sensitivity loading cell was connected to the cross head. In this way the cell directly dimensions the load acting on the specimen, without any need for laborious and imprecise corrections. The bars make it possible to correct any relative rotation between the specimen end-sections, and to assure the uniformity of the displacement distribution within a 1 μm tolerance. In this way it was possible to impose a step-by-step uniform crack opening through the notched cross-section of the specimens (Fig.2).

In the bending tests, the feedback signal was the crack opening displacement, which was measured by a clip-gauge placed astride the midspan section (Figs 3 and 4), along the bottom surface.

To test the prestressed beams representing a portion of the hollow slabs, the load (F, Fig.5) was applied by means of a hydraulic actuator, and the test was force-controlled.

The moiré grid had 40 lines/mm, and was glued to the central part of the specimens, in order to monitor crack formation and propagation.

The moiré fringe pattern was continuously recorded during the whole test, by means of a video-camera, which was connected to a video-recorder. All the recorded images were analyzed by means of a software package, in order to

valuate the displacement field and crack opening. The outputs of the transducers, as well as those of the load cell, were automatically stored in a data logger. Fig. 5 shows the test setup and the instrumentation.

TENSILE TESTS

The direct tensile tests were performed on cylindrical specimens, cast in special Plexiglas moulds. All of the specimens had an external circumferential notch 7 mm deep, obtained by inserting a stiff rubber ring into the mould (Fig.1). The fresh concrete was poured into the moulds in several layers in order to favor concrete compaction. The tests were controlled with two linear voltage differential transformers (LVDT) placed parallel to the longitudinal axis of the specimen and diametrically opposed.

The complete load-displacement curves for each specimen and for each material are shown in Fig.6. The specimens with no steel micro-fibers showed typical brittle behavior. The response was linear up to roughly 60% of the peak load, and became always very steep in the post-peak phase.

The fiber-reinforced specimens showed a much gentler softening response, owing to the cohesive behavior of multiple cracking, as well as to the bridging action of the fibers across the localized cracks, up to fiber pull-out.

From the mechanical point of view, it is interesting to observe that the fiber-reinforced specimens showed a critical crack opening (= crack width with no stress transfer at the interface), about forty times larger than in unreinforced specimens.

With reference to Young's modulus (which was measured with strain-gauges having a 20 mm base length), the fiber content has no practical influence (Fig.7).

BENDING TESTS

Three-point bending tests were performed on 100x100x400 mm specimens, with a simply- supported span of 360 mm.

The response of the unreinforced specimens was very brittle and the post-peak behavior was characterized by a "snap-back". This phenomenon was fully controlled since the feedback signal to the servo-controller was the crack mouth opening displacement (CMOD), which is a monotonically-increasing function in this type of tests (Figs 8,9,10). The fibers also increased the peak load, besides inducing a more ductile kind of behavior.

In Fig.11 a comparison between the nominal tensile and bending strengths is shown, for different fiber contents.

The fiber reinforcement contributes markedly to the nominal strength since the fibers bridge the cracks, allowing significant tensile stresses to be transmitted in the damaged regions. As a result, larger stresses can develop in the compressive region, with a better exploitation of the improved compressive strength of the fiber-reinforced concrete.

INTERPRETATION OF THE EXPERIMENTAL CURVES IN TENSION

To assess the material properties in tension from the experimental curves, the identification procedure presented in [8] is applied here.

The load-elongation curves (see Fig.12) exhibit an initial linear ascending branch up to more than 50% of the peak load, a second increasingly non-linear branch up to the peak load, a first steep descending branch down to about one third of the peak load and, finally, a very flat branch down to total unloading. A close interrelation of these four branches with the increase in the notched-section rotation allows us to attribute a clear physical meaning to each branch.

As far as the non-linear elastic stress - strain curve is concerned, a three-parameter (E_c , $\epsilon_{\max}$ and σ^*) relation is adopted:

$$E_c \epsilon = \sigma \left(1 - \left(\sigma / \sigma^* \right)^2 \right)^{-1/2} \quad \text{where } \sigma(\epsilon_{\max}) = f_{ct} \quad (1)$$

where f_{ct} is the effective material tensile strength, E_c is Young's modulus, $\epsilon_{\max}$ is the maximum tensile strain before the onset of cracking, and σ^* is the asymptotic value of the stress corresponding to an infinite strain.

Firstly, the Young's modulus E_c is evaluated referring to the initial ascending branch, where the material response is linear elastic. The load P_M at the limit of linearity (point A in Fig.12) corresponds to a sudden increase in the rotation increments in the notched section. This increase is due to the non-linear material response at the onset of microcracking. The Young's modulus can, therefore, be calculated :

$$E_c = P_M L_b \delta_M^{-1} A_i^{-1} \quad (2)$$

where A_i is the notched-section area, δ_M is the average elongation at the limit of linearity and L_b is the base length of the transducers.

Secondly, the maximum longitudinal strain $\epsilon_{\max}$ is evaluated at peak load $P_{\max}$ (point B in Fig.12). Being affected by notch effects, $\epsilon_{\max}$ will be greater than the measured nominal strain ϵ_{nM} at peak load. If $2\rho_e$ is the longitudinal length, astride the notch, where strain concentration occurs, and ϵ_{aM} is the corresponding strain in the region unaffected by the notch, the effective value of $\epsilon_{\max}$ can be formulated as follows (Fig.13):

$$\epsilon_{\max} = \epsilon_{\max}(\epsilon_{nM}, \epsilon_{aM}, \rho_e, L_b) \quad (3)$$

Thirdly, the value of σ^* can be calculated with the following theoretical relationship:

$$\sigma^* = \left(\alpha_k'^2 / \left(2\sqrt{\alpha_e'^2 - \alpha_k'^2} \right) \right) \sigma_{nM} \quad (4)$$

where α_k' is the stress concentration factor at the tip of the notch for an ideal linear-elastic material, and α_e' is the non-linear strain concentration factor:

$$\alpha_\varepsilon = E_c \varepsilon_{\max} / \sigma_{nM} \quad (5)$$

and σ_{nM} is the nominal tensile strength, corresponding to the limit of linearity (point A in Fig.12 and point M in Fig.14):

$$\sigma_{nM} = P_M / A_i + M(\varphi_M) / W_i = 4P_M / (\pi D_i^2) + E_c D_i / (2L) \varphi_M \quad (6)$$

where D_i is the diameter of the notched section, L is the specimen length, W_i is the resistant modulus, M is the secondary bending moment provoked by non-intentional eccentricities and φ_M is the rotation in the notched section (corresponding to point A in Fig.12 and point M in Fig.14).

Finally, the material tensile strength f_{ct} is obtained:

$$f_{ct} = E_c \varepsilon_{\max} \sigma^* / \sqrt{\sigma^{*2} + (E_c \varepsilon_{\max})^2} \quad (7)$$

The transformation of the nominal into the effective σ - ε relation (Eq.1) is shown in Fig14.

As far as the cohesive stress - crack opening curve is concerned, a two-parameter (w_c and κ) hyperbolic relation [9] is adopted.

$$\begin{aligned} \sigma_w / f_{ct} &= (d_{\max} (w_c - w)) / (w_c (d_{\max} + \kappa w)) \quad \text{for } 0 \leq w \leq w_c; \\ \sigma_w / f_{ct} &= 0 \quad \text{for } w \geq w_c \end{aligned} \quad (8)$$

where σ_w are the stresses transferred along the crack surfaces, w_c is the critical crack width beyond which no cohesive stress σ_w is transferred, κ is a coefficient related to the slope of the hyperbolic curve and $d_{\max}$ is the maximum aggregate size. Note that -for a given fracture energy G_F - the value of κ can be calculated by matching the definite integral of Eq.8 (evaluated from zero to w_c) with G_F , whose value represents the area underneath the σ_w - w curve.

Let us now make a few remarks: (a) across a fully propagated crack in the notched section no stress and strain concentration occurs; (b) assuming a linear distribution for the local crack width $w(\rho, \vartheta)$ over the cracked section, the four measured elongations δ (see Fig.1) make it possible to assess the crack-width distribution at each loading step; (c) the value of both κ and w_c can be predicted by means of Eq.8, so that a possible σ_w - w relation can be identified at each loading step, where $w = \max w(\rho, \vartheta)$; and (d) by minimizing the standard deviation between the theoretical and experimental σ_w - w curve, the most probable cohesive relation is identified.

BENDING TESTS TO EVALUATE THE TENSILE PROPERTIES OF CONCRETE

The most spontaneous way to determine the behavior of concrete in tension is to carry out direct tensile tests. Nevertheless, the specimen response is highly influenced by its geometry and size, material heterogeneity, boundary and loading

conditions. Furthermore, the collapse can develop in a sudden and unstable way, with a localization of the strain which may be incompatible with the base length of the usual instruments. As a consequence, the test results may not represent the actual material properties, unless the above-mentioned disturbances are minimized. Thus, indirect tensile tests (such as three-point bending tests and splitting tests) are often preferred since their results are more reliable and less dispersed. However, at least in principle indirect tests lead to less exact results, owing to their more complex stress and strain state, which makes these tests a sort of "structural" tests, with solid, microcracked, plasticized, fracturing and fractured material, and step-by-step variable loading and geometrical conditions. As a matter of fact, it is not possible to identify exactly each contribution to the global response of each specimen. To analyze the structural effects correctly, it is necessary to know: (a) the local constitutive laws, (b) their governing parameters; (c) the structural effects and the consequent stress and strain redistribution; and (d) the interaction between the fractured and solid regions.

According to the sketch in Fig.15, the overall beam response is indicated by the experimental load-displacement curve; the local response of the mid-span notched-part is represented by the experimental bending moment-rotation curve; the notch effects are described by the experimental bending moment-CMOD curve; and the fracture process by the experimental CMOD-crack penetration curve.

By fitting the experimental load-displacement curves (obtained via three-point bending tests) by means of a specific theoretical approach based on the material properties measured in direct tension [10], it is possible to identify the "objective" or "true" values of the parameters which govern the stress-strain and stress-crack opening laws of the concrete in tension.

ELIMINATION OF THE STRUCTURAL EFFECTS

In the present analysis the following scheme was adopted (Fig.16):

- all the experimental load-displacement curves (tensile and bending tests) were treated according to the above-mentioned procedure, for each mix design, curing and age;
- many structural effects were filtered out both by placing suitable instruments in the most significant zones of the specimen, and by adopting sophisticated devices to reduce any disturbing interaction between the testing equipment and the specimen;
- for both the tensile and bending behavior, the above procedure was adopted, making it possible to compensate for the lack of information about the actual stress and strain distribution, and fracture process occurring during the tests;
- by adopting the tensile-test response as input data and the bending-test response as a reference, it was possible to develop an algorithm for a more reliable and accurate formulation of the constitutive law in tension [11].

For $v_f = 0$ and 4% by volume, the "objective" concrete properties are given in Table 3.

STRUCTURAL RESULTS

Two tests were carried out on pretensioned beams subjected to three-point bending (see the test set-up, Fig.5). The first P/C beam was made of fiber-reinforced concrete (1% by volume of smooth microfibers $\varnothing$ 150 μ m/ L 13 mm, beam size 8x20x300 cm, f_c = 81 MPa, Fig.18), while the second P/C beam had no fibers (beam size 16x20x300 cm, f_c = 46.9 MPa, Fig.19).

Both beams had a simply-supported span of 290 cm and were designed for the same ultimate capacity in bending (P_u =18 kN, reinforcement: one single straight 9.5 mm tendon). Furthermore, the FRC beam (Fig.18) was devised to simulate in some way a portion of a typical hollow-core slab (Fig.17).

The response curves are presented in Fig. 20. The moiré fringe patterns (fiber-reinforced specimen) are shown in Figs.21 and 22. With reference to point A of the load-displacement curve, the first crack appeared close to the middle cross-section. The behavior of the beam changed from linear to non linear, as can be seen from the load-deflection curve. With reference to point B, a second crack appeared. The crack widths were 0.075 and 0.05 mm at point C, and 0.1 and 0.075 mm at point D, respectively. Finally, at point E, (deflection = 4 cm) the crack widths were 0.4 and 0.341 mm, respectively (load level = 75% of the ultimate load). The P/C no-fiber beam showed the first crack at point A (Fig.23). Afterwards, no more cracks appeared. Beyond the peak load, the beam showed a sudden drop in the load carrying capacity. At point B, the crack opening was 0.54 mm, (load level = 40%, falling branch). All cracks formed at right angles to the principal tensile stress; in the FRC beam, the cracks originating in the central part of the beam propagated toward one of the loading points.

DISCUSSION OF THE RESULTS

Figs. 21 and 22 show the moiré fringe patterns at five load levels (no load = initial mismatch and points A, B, D, E of Fig.20, left) for the FRC beam, and Fig. 23 shows the moiré fringe patterns at three load levels (no load = initial mismatch, point A and point B of Fig.20, right) for the beam with no fibers. The beams, at the load level corresponding to first cracking, showed a uniform deformation along the bottom face, with a deformation of 0.0005 m/m in the fiber-reinforced beam, 2.5 larger than the deformation observed in the plain concrete beam.

Fiber-reinforced concrete seems to ensure a tendon-to-concrete bond about twice as large as that of plain concrete, as can be observed in Fig. 24, where the applied load is plotted against the draw-in of the tendons. Thus the beams failed because of bond reaching its ultimate limit state.

Finally, it is worth noting that the two beams exhibit roughly the same load at the end of the mostly linear ascending branch (9.2 kN and 11.2 kN respectively), but the subsequent behavior is entirely different (Fig.20).

CONCLUDING REMARKS

The results can be summarized as follows:

- Ultra High-Strength microconcretes ($f_c \geq 130$ MPa) with small-diameter quartzitic aggregates are very brittle, in terms of both tensile strength ($f_{ct}/f_c = 1/30$) and toughness; however, by adding 2 to 4% of smooth metallic fibers (by volume) the material becomes stronger in tension ($f_{ct}/f_c \cong 1/20$) and definitely tougher (the fracture energy increases by two orders of magnitude, from 130 to 8000 N/m).
- In the UHS microconcretes studied here, the Young's modulus is not significantly affected by fiber amount, since fiber contents as large as 2-4% by volume increase E_c by less than 10%.
- The difference between the values of the Young's modulus in tension and compression is negligible ($\sigma_t \leq f_{ct} / 3$, $\sigma_c \leq f_c / 30$).
- By controlling the Crack-Mouth Opening Displacement (CMOD), the notched specimens can be tested successfully in direct tension even in the case of ultra high-strength concretes (with or without fibers, $v_f = 0-4\%$).
- The nominal tensile strength in bending varies from less than 2 to more than 3 times the tensile strength in direct tension, by adding up to 4% of fibers.
- By adopting a special procedure the results in direct tension can be cleansed of any undesirable structural effect (lack of concrete uniformity in the notched section, rotation of machine platens), and the constitutive laws (stress-strain and stress-crack opening) can be identified by means of an algorithm capable of combining the curves in direct tension and those in bending.
- The post-elastic behavior of typical prestressed, pretensioned beams made of a high-performance Concrete (same nominal ultimate capacity in bending) becomes completely different by adding smooth steel fibers (up to 1% by volume): in the case of plain concrete (no fibers) there is a steep falling branch followed by an extended plateau (- 30% in bending capacity); with 1% of steel fibers, there is a plastic plateau, followed by a further increase in bending capacity (+ 30%).
- The better performance of pretensioned FRC beams seems to be mostly due to the better concrete-to-tendon bond, which reduces definitely the tendon draw-in and contributes markedly to structural ductility.

Finally, it may be observed that the same nominal material may behave differently in a test specimen and in a real structure, both cast and tested in lab conditions. As a matter of fact, the structural behavior of the specimen often overshadows the "true" mechanical properties of the material, making specimen response different from material response. Consequently, in order to achieve "objectivity", all structural effects in specimen behavior have to be identified and removed via a suitable procedure based on a thorough analysis of both stress and strain concentrations in the region where crack and damage localize.

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TABLE 1—MIX DESIGN OF TEST SPECIMENS.

MIX-DESIGN	NO FIBERS	FIBERS (2/4% by volume)
cement 52.5	696 kg/m ³	667 kg/m ³
silica fume (SF/C)	174 kg/m ³ (25%)	167 kg/m ³ (25%)
quartz 0.06-3.2 mm	1300 kg/m ³	1246 kg/m ³
superplasticizer (SP/C)	14.4 kg/m ³ (2.1%)	13.8 kg/m ³ (2.1%)
water (W/C+SF)	196 kg/m ³ (22.5%)	187 kg/m ³ (22.4%)
steel fibers (v _f %)	//	162/324 kg/m ³ (2.1/4.2%)

TABLE 2—MIX DESIGN OF THE PRESTRESSED BEAMS.

MIX-DESIGN	NO FIBERS	FIBERS (1% by volume)
cement 52.5	352 kg/m ³	695 kg/m ³
silica fume (SF/C)	//	174 kg/m ³ (25%)
sand	813 kg/m ³	652 kg/m ³
gravel 3-6 mm	527 kg/m ³	652 kg/m ³
gravel 5-10 mm	592 kg/m ³	//
superplasticizer (SP/C)	11.7 kg/m ³ (3.3%)	46.4 kg/m ³ (6.6%)
water (W/C+SF)	97 kg/m ³ (27.5%)	200 kg/m ³ (23%)
steel fibers (v _f %)	//	78 kg/m ³ (1%)

TABLE 3—CONCRETE PROPERTIES IN TENSION FROM THE IDENTIFICATION PROCEDURE.

FIBER CONTENT	E_c (MPa)	f_{ct} (MPa)	G_F (N/m)	w_c (mm)
0 % by volume	50000	9.6	130	0.15
4 % by volume	51.000	14.7	8000	3

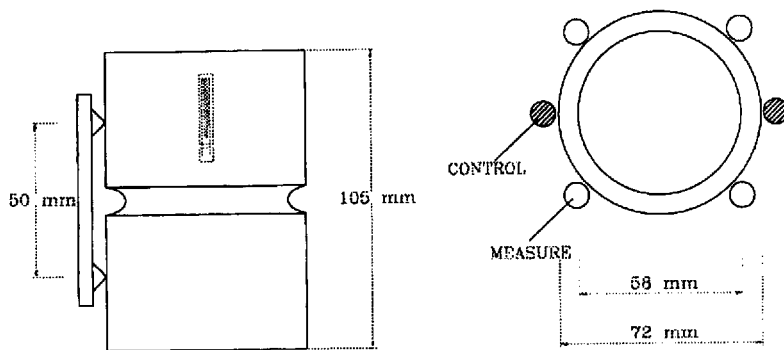


Fig. 1—Testing in tension: notched specimens and instrumentation (see also [7]).

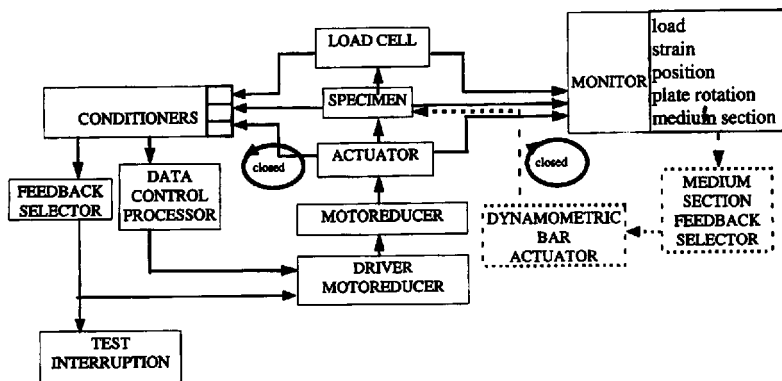


Fig. 2—Flow chart of the tensile testing procedure, with the feedback control.

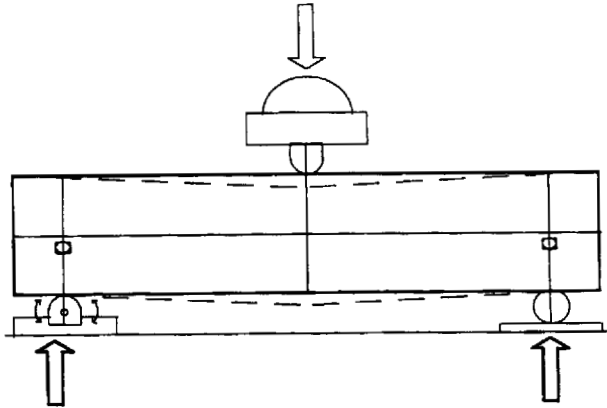


Fig. 3—Prismatic specimens for three-point bending tests.

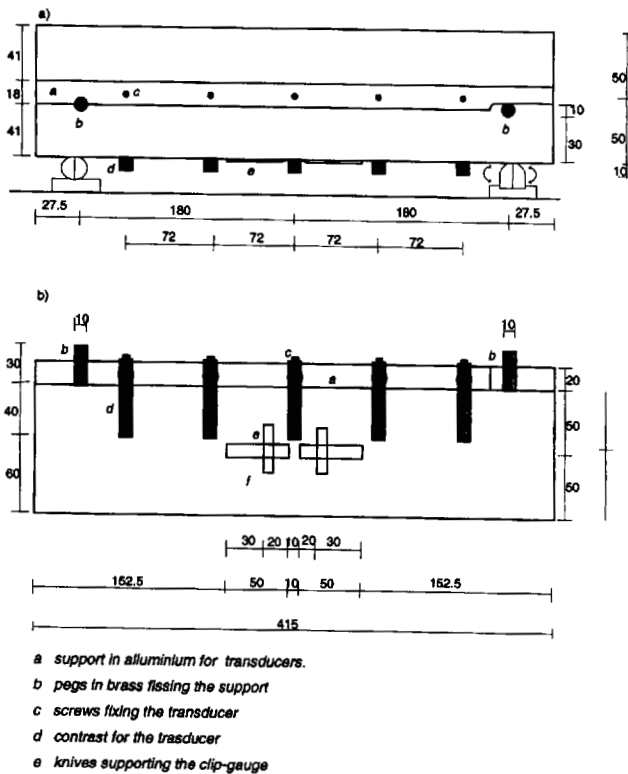


Fig. 4—Instrumentation of the three-point bending tests (dimensions in mm).

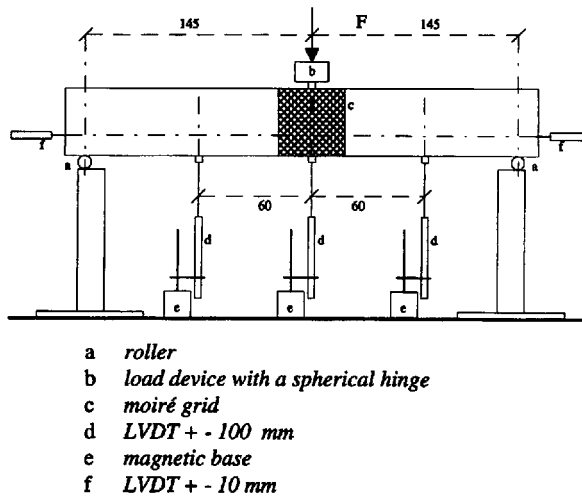


Fig. 5—Instrumentation and loading equipment for the bending tests on prestressed beams (dimensions in cm).

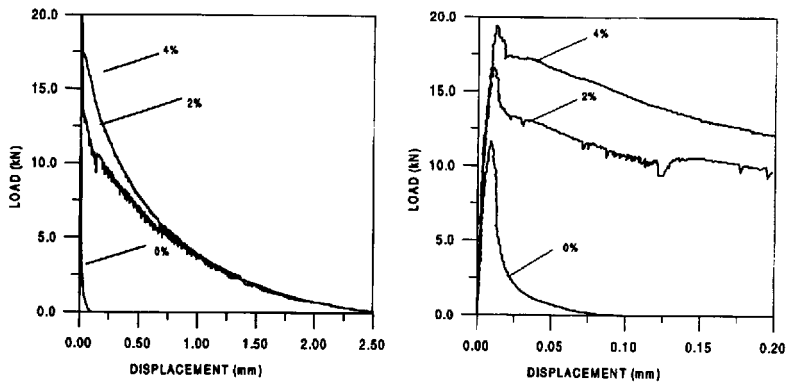


Fig. 6—Load displacement curves in direct tension (notched specimens).

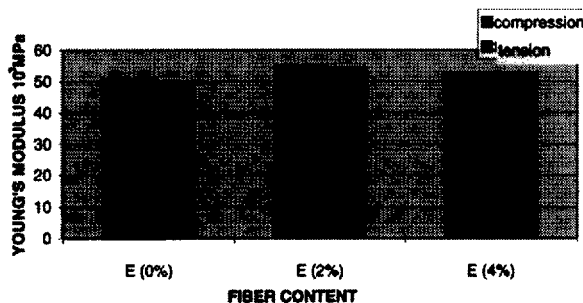


Fig. 7—Young's modulus in tension and compression: effects of fiber content.

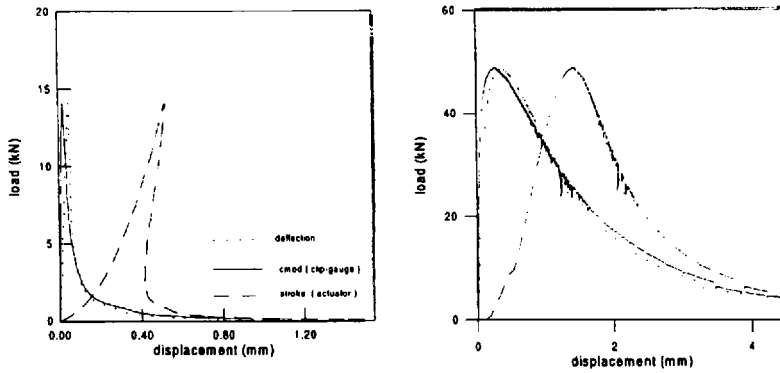


Fig. 8—Test specimens in bending, with no fibers (left) and with fibers (right, 4 percent by volume).

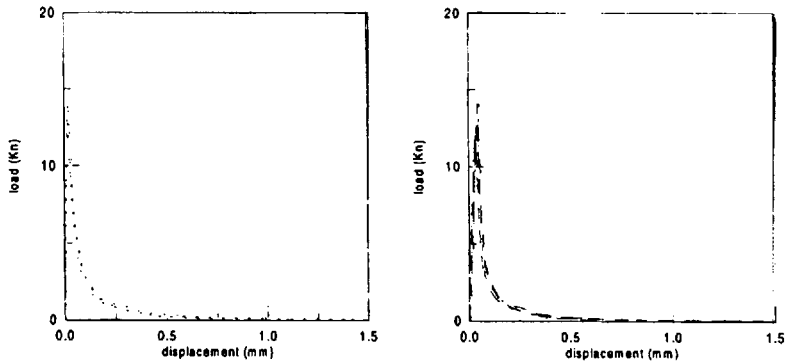


Fig. 9—Test specimens in bending without fibers: load CMOD and load deflection curves.

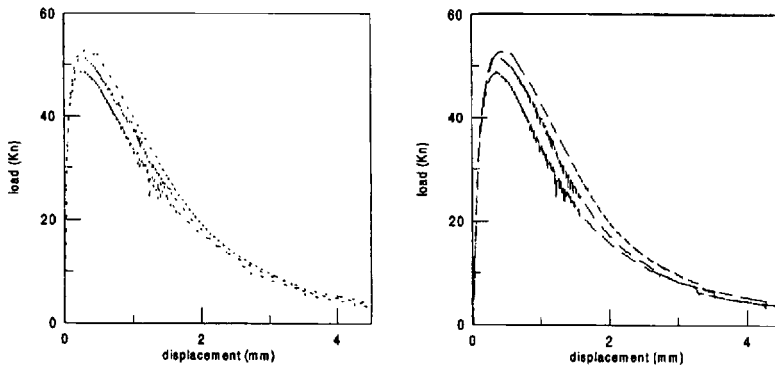


Fig. 10—Test specimens in bending with fibers (4 percent by volume): load CMOD and load deflection curves.

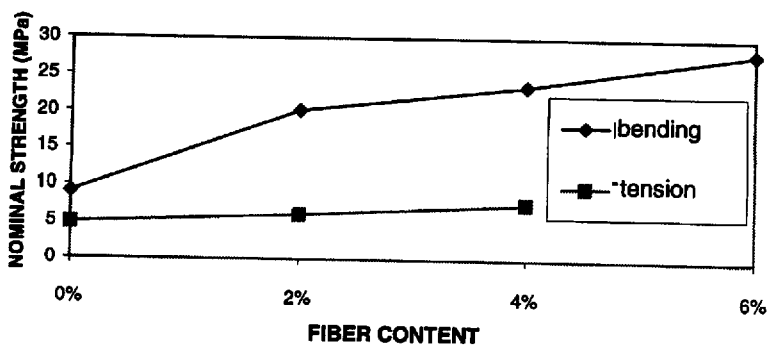


Fig. 11—Nominal tensile and bending strength: influence of fiber content.

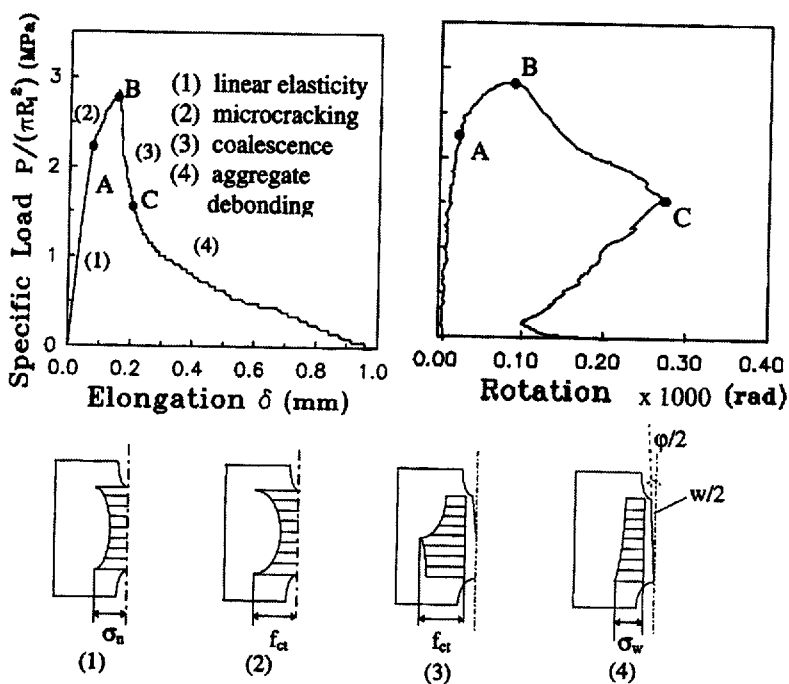


Fig. 12—Load displacement and load rotation curves; stress and c.o.d. distributions.

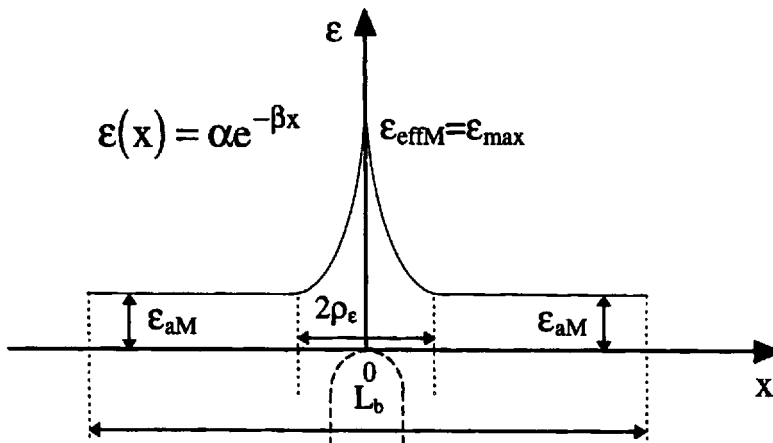


Fig. 13—Longitudinal strain distribution astride the notch in a tension test.

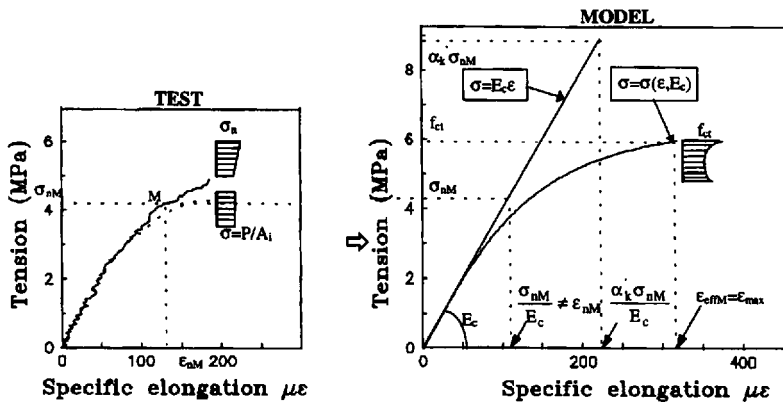


Fig. 14—Typical stress-strain curves cleansed of the bending effects (left) and notch effects (right).

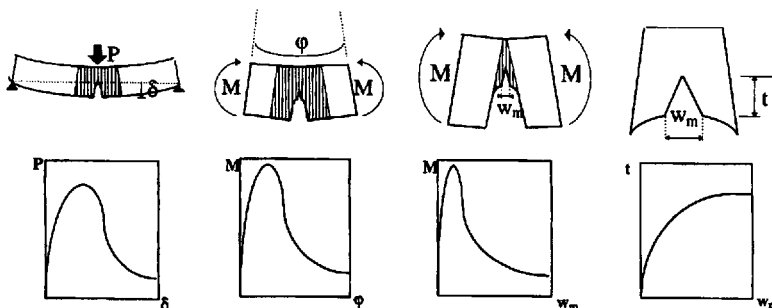


Fig. 15—Physical phenomena (top) and response curves (bottom) in a bending test.