WORKABILITY AND STRENGTH PROPERTIES OF STEEL FIBER REINFORCED CONCRETE

By

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ABSTRACT

The strength properties of fiber concrete have received much attention in the past with little emphasis being placed on the workability of the composite in its fresh state. Both field and laboratory studies have identified workability of the composite as a potential problem due to fiber addition resulting in field strengths lower than those obtained in the laboratory. Therefore, in addition to studying strength properties such as fatigue, the workability of fiber concrete was considered in detail.

To obtain an optimum mix in terms of strength and workability, fiber length and concentration are of considerable importance. In the initial stages of the study, direct pull-out tests were completed to determine the optimum fiber length i.e. the fiber length just sufficient to allow full bond development under ideal conditions. The theoretical critical fiber length (1.8 cm) was found to correspond closely with the experimental optimum length of 1.27 to 1.91 cm.

Emphasis was placed on the static and fatigue flexural strengths which are important properties required for rigid pavement design. Compression, split cylinder and impact resistance strengths were also considered. The tests

indicated the importance of adequate workability with respect to obtaining improved strength performance with fiber addition. An increase in fiber length and/or concentration, decreased the workability of the composite resulting in an increased number of flaws decreased compaction and decreased uniformity of fiber distribution. The relative strength increase or decrease was dependent on the predominant influence; i.e. strength loss due to increased flaw generation or strength increase due to the fiber addition. The only significant factor influencing workability besides adjusting the fiber length and/or content was with the use of superplasticizers.

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CHAPTER 1

INTRODUCTION

1.1 FIBER CONCRETE

Fiber concrete is a composite material consisting of a hydraulic cement binder, with or without aggregates of various sizes, and a dispersion of natural or synthetic fibers. To ensure volumetric stability most concrete composites contain some aggregate with the possible exception of asbestos cement products [Rilem, 1977]. In this study, the term fiber concrete is restricted to a composite incorporating a modified concrete mix with randomly dispersed fibers.

Until recently, practically all fiber reinforced cement materials were produced with asbestos for cladding, water tanks, pipes and fire resistant components. However, since the 1950's there has been a dramatic increase in the use of fiber reinforced plastics that has also lead to intensive research in the development of fiber reinforced concrete [McCurrich and Adams, 1973]. A summary of some typical fibers and fiber properties which have been considered for reinforcing concrete is given in Table 1.1. Steel fiber, given much emphasis in the past few years, is investigated herein due

TABLE 1.1 TYPICAL FIBRE PROPERTIES [Rilem, 1977]

Fibre	Diameter (μm)	Density ($10^3 kg/m^3$)	Young's modulus (kN/mm^2)[1]	Tensile strength (kN/mm^2)	Elongation at break (%)
Asbestos:					
(a) Chrysotile ...	0.02-20	2.55	164	3.1	2-3
(b) Crocidolite ..	0.1 -20	3.37	196	3.5	2-3
Carbon:					
(a) Type I	3	1.90	380	1.8	~0.5
(b) Type II	9	1.90	230	2.6	~1
Polypropylene	20-200	0.9	5	0.5	~20
Nylon (Type 242)..	>4	1.14	4	0.9	~15
Kevlar:					
(a) PRD 49	~10	1.45	133	2.9	2.6
(b) PRD 29	12	1.44	69	2.9	4.0
Sisal	10-50	1.5	-	0.8	~3
Glass	9-15	~2.6	80	2-4	2-3.5
Steel	5-500	7.8	200	1-3	3-4

[1] $1 kN/mm^2 = 10^3 MPa$

to the many potential applications that have been identified, some of which are discussed in the next section.

1.2 APPLICATIONS OF STEEL FIBER REINFORCED CONCRETE

Since the studies completed by Romauldi and Baston (1963), and Romauldi and Mandel (1964), steel fiber reinforced concrete has been transformed from a mere laboratory development into a proven construction material. The composite has not only performed well in the laboratory, but also in the field showing superior mechanical properties; i.e. improved static and fatigue flexural strengths, wear and impact resistance, thermal and shock resistance, and good serviceability. Unlike plain concrete, steel fiber reinforced concrete can carry loads after cracking.

Much emphasis has been placed on using steel fiber reinforced concrete as a substitute for conventional plain concrete and asphalt pavements. Several field, experiments have been designed to study comparatively such variables as concrete mix design, fiber type and quality, joint spacing and design overlay thickness, and type of bonding to old surfaces (pavement overlays). Thinner slab and overlay placement has been possible with the use of fiber concrete than previously with plain concrete due to increased first crack strength, spall resistance and the ability to carry loads after cracking. Field placement studies have also shown that relatively large masses of fiber concrete can be produced and placed using current technology without much difficulty [see, e.g., Lankard and Walker, 1975].

Besides pavement applications, steel fiber reinforced concrete has also been used in other areas including:

1. repair of high velocity water passages such as sluices and spillways for dams when wear and formation of cracks must be controlled [see, e.g., Schrader and Munch, 1976];
2. tunnel liners and slope stabilization - although there are various means of placement, shotcretting of fiber concrete onto rock surfaces either for tunnel liners or slope stabilization has received much attention. Fiberous shotcrete has been shown to be two times as strong as the non-reinforced material [see, e.g., Kaden, 1974];
3. concrete pipes - large concrete pipes using steel fibers have not performed as well as concrete pipes using conventional wire reinforcement with regards to dollar value or volume of steel used. However, load requirements can be met with sufficient fiber content and decreased wall thickness to allow the structural element to behave as a flexible culvert which would permit soil arching

to be developed [see, e.g., Gray, 1972; and Henry, 1973];

4. factory and warehouse floor slabs - due to higher first crack strength and ductility, the slab thickness can be reduced. In addition, the material's ability to resist considerable strain while retaining strength and structural integrity can reduce maintenance and prolong service life [see, e.g., Battelle, 1975]; and
5. refactory linings - the fibers help overcome the cracking strains induced by thermal gradients [Gray, 1972].

1.3 ECONOMICS

It is difficult to access the economic value of the composite on the basis of material costs alone. Improved performance and reduction in size (e.g., thinner pavement overlay thickness) must also taken into account. In addition the current cost of fibers produced in small quantities is not necessarily indicative of long term production costs [Swamy and Kent, 1973]. However, any gain in engineering properties has, of yet, not been sufficient to offset economic considerations to a degree where large scale use of such a product is feaseable except for a few exceptional cases such as repair of sluiceways [see, e.g., Schrader and Munch, 1976].

1.4 PROBLEMS WITH STEEL FIBER REINFORCED CONCRETE

Although steel fiber reinforced concrete has been shown to be an alternative construction material with much potential, various problems have been encountered in the field due to poor control of concrete quality and reduced workability. The lack of workability which causes inadequate concrete compaction and non-uniform fiber dispersion has been attributed to the fiber geometry, i.e., length and diameter. The fiber geometry has also been responsible for the formation of fiber clumps (balling) which do not allow the matrix to flow between the fibers. Fiber balling, inadequate compaction and non-uniform fiber dispersion have lead to lower mechanical properties in the field than observed in the laboratory.

1.5 SUMMARY OF OBJECTIVES

The trend in the past has been to use fibers 2.54 cm (1 inch) long or longer to develop sufficient fiber-matrix bond strength in order to fully utilize the fibers' tensile strength. However, longer fiber lengths decrease concrete compaction and uniformity of fiber distribution due to decreased workability, resulting in lower strength properties in the field than initially anticipated from laboratory data. Therefore, this study has been directed towards investigating the influence of steel fiber addition on the workability and mechanical properties of concrete. More specifically, the objective of this study is to investigate the following factors influencing fiber concrete performance:

1. Fiber-matrix bond strength - Direct fiber pull-out tests were completed to determine an optimum fiber length where fibers just begin to break rather than pull-out. A parameter study to investigate curing, matrix and fiber characteristics was also completed to determine the sensitivity of bond strength.
2. Static and fatigue flexural strengths - Since the use of steel fiber reinforced concrete has great potential for pavement structures, the static and fatigue flexural strengths were the prime mechanical properties considered. This study tried to determine whether a sufficient increase in fatigue endurance would be obtained to result in the design of thinner overlay thicknesses than is presently possible using PCA pavement design procedures for plain concrete. Additional mechanical properties studied included: compression, split cylinder and impact strengths.
3. Workability - The workability of fiber concrete, which has received little attention in the past, has been covered in detail by investigating the

influence of fiber length and volume, workability additives and minor changes in free paste on the flow properties of the composite. The workability not only affects the mechanical properties as discussed previously, but also influences the amount of labour required for production and placement which is reflected in the cost.

Fiber-matrix bond strength, mechanical properties and workability are covered separately in Chapters two, three and four, respectively. Each of these chapters contain a brief introduction and literature review in addition to the details of the experimental programs and discussion of experimental results. A summary of the experimental findings and topics requiring further study or extension is given in Chapter 5.

CHAPTER 2

FIBER MATRIX BOND STRENGTH

2.1 INTRODUCTION

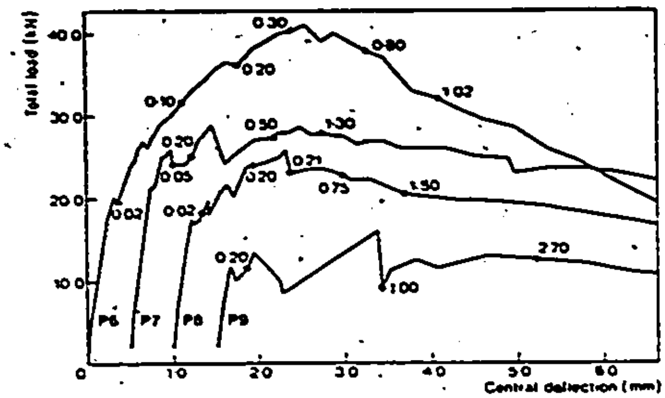
The effect that fibers have on improving a concrete's mechanical properties is related to the efficiency of the fiber in sharing the load induced tensile stress in terms of the fiber's own tensile strength. The observed difference between theoretical and experimental strengths is generally attributed to inability to fully utilize fiber tensile strength due to low bond strength between fibers and matrix [Tattersall and Urbanowicz, 1974; Naaman and Shah, 1975; and Maage, 1977]. Therefore, it is apparent that the fiber-matrix interface is of prime concern as it governs the actual stress transfer.

Although the importance of a strong interface fiber-matrix bond strength cannot be overemphasized, no direct correlation has been shown to exist between increased bond strength as measured by single pull-out tests and improved composite behaviour [Swamy, 1973; Tattersall and Urbanowicz, 1974; and Pinchin and Tabor, 1978B]. In view of this, the minimum fiber length required to develop sufficient bond strength to fully utilize the tensile strength of the fiber, i.e. critical fiber length, was the main parameter of concern in the study. The importance of minimizing the fiber length is reflected in fiber handling costs and workability of the composite which will be discussed in following chapters. Additional factors influencing bond strength which have been covered include: basic matrix properties (sand type and water-cement ratio); fiber tensile strength; and curing.

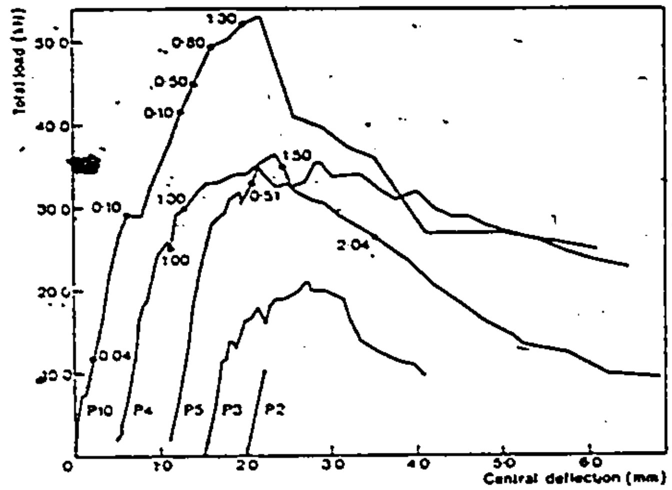
2.2 EFFECT OF FIBERS ON CONCRETE

Unlike plain concrete, the first crack and ultimate do not necessarily correspond to the same point on a load-deflection curve for fiber reinforced concrete, due to the randomly distributed fibers. For low fiber concentrations, first crack and ultimate occur simultaneously as anticipated. Figures 2.1(a) and (b) illustrate the influence of fibers on static flexural load-deflection properties of fiber reinforced paste beams.

In addition to factors such as fiber volume, stiffness and orientation discussed in the next chapter, the load-deflection behaviour of the fiber composite is strongly dependent on the bond strength and frictional stress transfer at the fiber-matrix interface [see, e.g., Swamy et al., 1973; and Pinchin and Tabor, 1978C]. The increased ductility due to the frictional strength can be of considerable importance with regard to impact resistance and factors of safety for the static flexural strength. However, fatigue resistance is not significantly influenced by the post cracking strength



(a) *Load-central deflection curves for Beams P6-P9, showing maximum widths—in millimetres—at different stages of loading.*



(b) *Load-central deflection curves for Beams P2-P5 and P10 showing maximum crack widths—in millimetres—at different stages of loading.*

Beam	Volume Fraction	Fibre (diameter × length) mm
P1	0	—
P2	0	—
P3	1·0%	Hooked 0·36 × 41
P4	2·0%	Ditto
P5	3·0%	Ditto
P6	2·0%	Duoform 0·64 × 59
P7	2·0%	Crimped 0·59 × 49
P8	2·0%	Crimped 0·79 × 97
P9	2·0%	Polypropylene 12 000 denier, 1·50 (bulk) × 51
P10	2·13%	14 No. 6·2 mm dia. bars arranged in two columns each of 7 bars, and uniformly spaced by two U-stirrups.

FIGURE 2.1 Flexural Load-deflection Curves for Cement Paste Beams (Hughes and Fattuhi, 1976)

as indicated by proposed mechanisms for fatigue failure given in the next chapter. A key part of the study was a comparison of the theoretical critical fiber length to the critical fiber length obtained experimentally in pull-out tests.

2.3 FACTORS INFLUENCING THE FIBER-MATRIX INTERFACE

2.3.1 INTERFACE PROPERTIES

A common consensus in the technical literature with respect to the fiber-matrix interface is that surface treatments have a limited influence on the pull-out load of fibers. This lack of influence is largely due to the nature of the surface which is very rough, and the process of pulling which is very damaging to the fiber surface [see, e.g., Maage, 1977; and Pinchin and Tabor, 1978C]. Tattersall and Urbanowicz [1974] have also indicated that surface treatments showing considerable increases in bond strength at 7 days do not seem to show the same promise at 28 days. In spite of the large amount of experimental work showing the ineffectiveness of surface treatments, some technical reports do indicate improvements with the application of the treatments [see, e.g., Mayfield and Zellys, 1973; and Naaman and Shah, 1975].

Pinchin and Tabor [1978C] have shown that increased fiber roughness has a tendency to increase the debonding load, i.e. bond strength, but has no real influence on the frictional stress transfer after debonding. Therefore, for improved composite properties with regard to post cracking behaviour, any increase in bond strength only slightly affects the frictional stress transfer. This is believed to be related to the breakdown of the matrix structure immediately adjacent to the fibers that decreases the normal applied stress and consequent friction development. To improve the frictional stress transfer, the fiber "misfit" needs to be considered. This concept of misfit is best illustrated through the use of the diagram in Figure 2.2. An increase in fiber misfit, i.e. an increase in confining pressure, increases both the debonding load and subequent frictional stress transfer. Pinchin and Tabor also showed that the frictional stress transfer decreased considerably with an increase in wire movement as shown in Figure 2.3.

It has been shown that fibers with positive anchorages along, or at the end of the fiber, show superior bond strength compared to plain straight fibers [see, e.g., Tattersall and Urbanowicz, 1974; Hughes and Fattahi, 1975; and Hughes and Fattuhi, 1976]. The anchorages are particularly beneficial for short fibers where the development length would otherwise be too short to obtain full use of fiber tensile strength. However, this study is restricted to plain steel fibers for which most of the available theory has been developed, and there has been most Canadian interest (Stelco has considered manufacture, for instance).

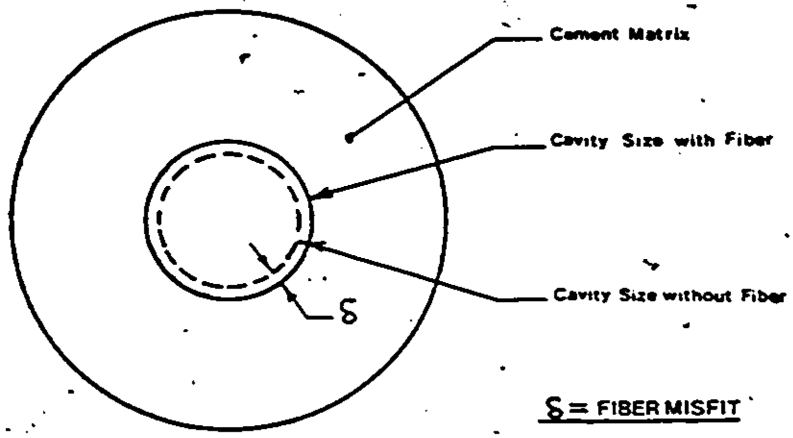


FIGURE 2.2 The Concept of Fiber-misfit

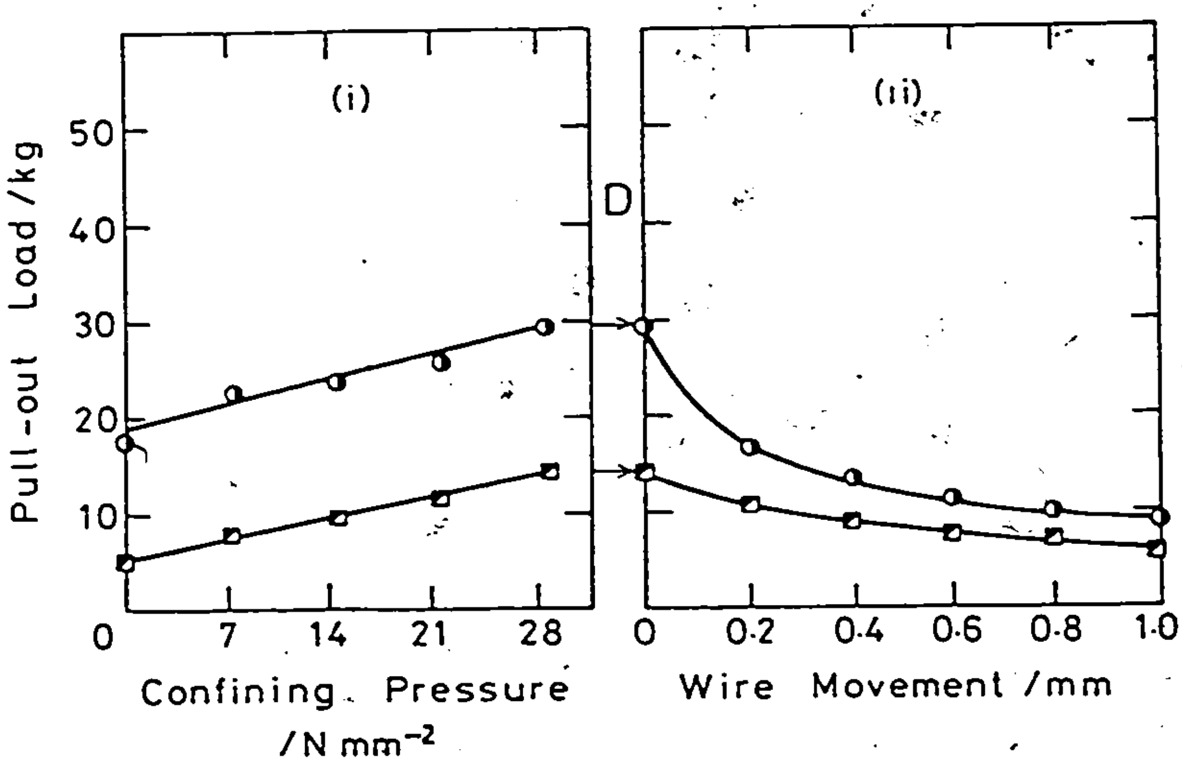


FIGURE 2.3 The Influence of Confining Pressure and Wire Movement on Pull-out Load (Pinchin and Tabor, 1978 C)

2.3.2 INFLUENCE OF CURING

The technical literature gives conflicting results concerning the dependence of pull-out load on duration of curing. Pinchin and Tabor [1978A] in their experimental work have shown that the fiber-matrix interface changes very little for curing periods greater than one week due to rapid enrichment of the interface with $Ca(OH)_2$ with Portland cement pastes. However, earlier investigations indicated that the effect of curing duration on fiber pull-out is also influenced by chemical surface treatments [Tattersall and Urbanowicz, 1974]. Therefore, it is apparent that the influence of curing duration on pull-out load is also dependent in other factors accounting for the observed differences reported by the various investigators.

It is generally agreed that the type of curing has a significant influence on bond strength at the fiber-matrix interface. It has been shown that lack of curing (i.e. a dry atmosphere) can reduce the pull-out load by 50 percent, however, curing under water or with high temperature treatment can increase the pull-out load by 70 percent. The influence of curing method has been attributed to the shrinkage or swelling of the cement paste or mortar which affects the fiber misfit discussed previously and the extent of paste hydration adjacent to the fiber. Therefore, in addition to the minimum time required for $Ca(OH)_2$ build-up adjacent to the fiber, water and aggregate to cement ratios would also contribute to the influence of curing on bond strength [see, e.g., Tattersall and Urbanowicz, 1974; Hughes and Fattuhi, 1975; and Pinchin and Tabor, 1978B].

It should be noted that for polymeric matrices in fiber reinforced polymers, large fiber-matrix contact pressures occur during shrinkage of the matrix resulting in large increases in stress transfer at the interface. Although it has been suggested that the shrinkage of a cement matrix on drying will also result in improved stress transfer, Pinchin and Tabor [1978B] indicated that specimens do not respond uniformly to shrinkage or expansion due to curing conditions. Based on their results (i.e. decreased stress transfer upon shrinkage) they expressed doubt about the elastic behaviour of the specimens. They suggested that contractive stresses due to matrix shrinkage caused microcracking adjacent to the fibers, resulting in disruption of the fiber-matrix stress transfer and inelastic behaviour.

2.4 CRITICAL FIBER LENGTH

2.4.1 CONCEPT

As indicated previously, it is desirable that the fiber length be such that the fiber bond strength is just equal to the ultimate tensile strength of the fiber. The idealized bond stress distribution for a fiber having a length greater than the critical length is shown in Figure 2.4. The bond stress is a maximum at the end of the fiber and tends to zero at $\ell_c/2$, where ℓ_c is the critical fiber length.

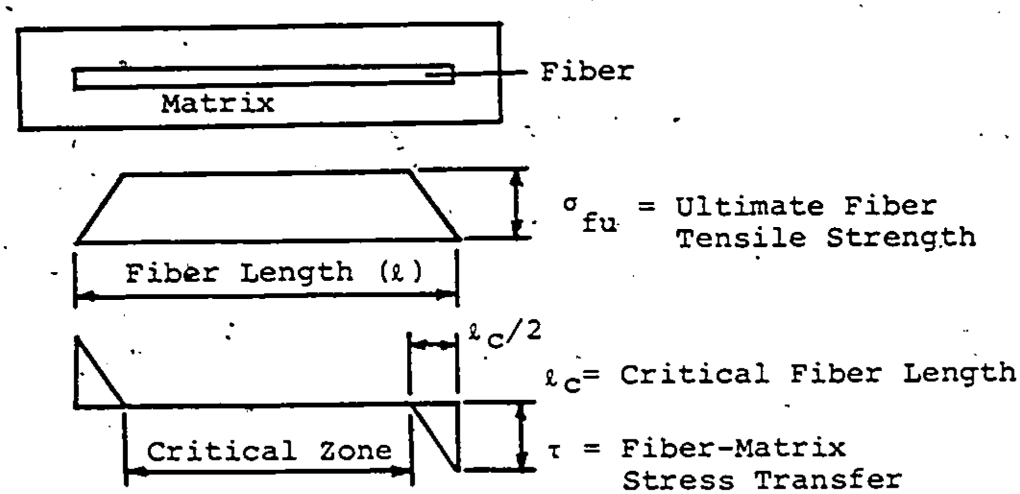


FIGURE 2.4 FIBER-MATRIX STRESS DISTRIBUTIONS

For very long fibers, the critical zone where assumed cracking of the matrix occurs is free of shear forces. In the following discussion, the equilibrium state of interest occurs when the critical zone reduces to a point. If a crack forms in the region where the bond stress transfer is developed, a modification to the derivation that follows is required such that no bond stress transfer occurs across the crack [Swamy, et al., 1973].

2.4.2 THEORETICAL CRITICAL FIBER LENGTH

A length of fiber is considered as shown in Figure 2.5:

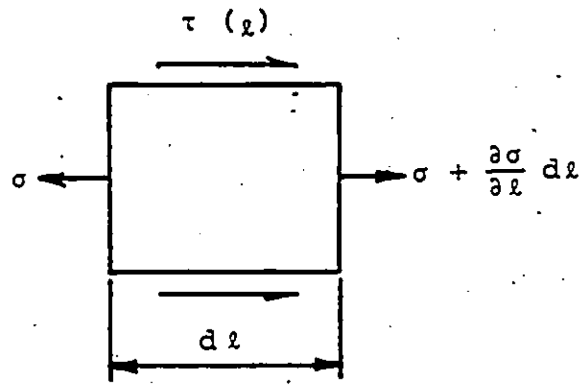


FIGURE 2.5 STRESSES ON INCREMENTAL LENGTH OF FIBER

where τ is the fiber-matrix bond strength over the length $d\ell$, and σ is the tensile stress in the fiber. Force equilibrium is then applied to the element yielding:

$$A_f \frac{\partial\sigma}{\partial\ell} = S\tau(\ell) \qquad (2\text{-}1)$$

where A_f is the fiber cross-sectional area and S is the fiber perimeter. It is assumed that the fiber length ℓ is the only varying parameter, therefore Equation 2-1 becomes:

$$\frac{d\ell}{d\sigma} = \frac{A_f}{S\tau(\ell)} \qquad (2\text{-}2)$$

after rearranging the terms. At the critical fiber length, the idealized stress distributions required to solve Equation 2-2 are shown in Figure 2.6.

The integral obtained from Equation 2-2, in which the fiber length, bond stress distribution and ultimate tensile stress of the fiber are considered, is

$$\int_0^{\sigma_{fu}} d\sigma = \frac{S}{A_f} \int_0^{\ell_c/2} (\ell)\,d\ell \qquad (2\text{-}3)$$

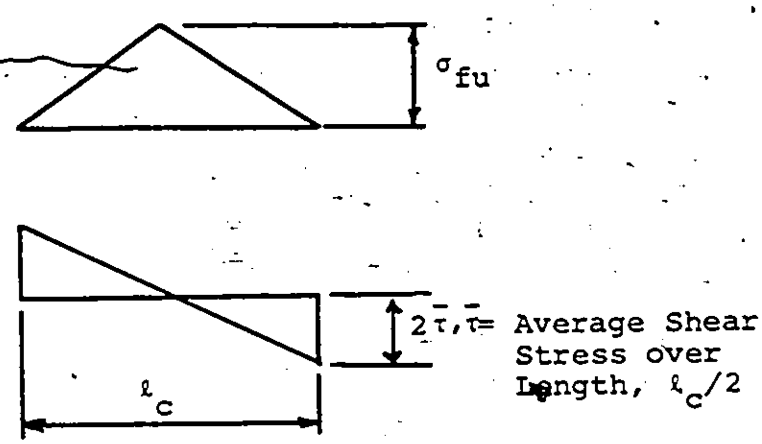


FIGURE 2.6 IDEALIZED STRESS DISTRIBUTIONS FOR $\ell = \ell_c$

Integrating Equation 2-3 using the stress distributions given in Figure 2.6 yields:

$$A_f \sigma_{fu} = S\bar{\tau}\left(\frac{\ell_c}{2}\right) \tag{2-4}$$

where $\bar{\tau}$ is the average shear stress along half the fiber. For a round fiber with diameter d, this leads to the equation often given in the technical literature [Swamy et al., 1973]:

$$\frac{\ell_c}{d} = \frac{\sigma_{fu}}{2\bar{\tau}} \tag{2-5}$$

In this study, cut fibers of roughly rectangular section provided by the Steel Company of Canada Limited (Stelco) were used. These fibers were idealized as true rectangular sections as shown in Figure 2.7.

For the rectangular fiber, Equation 2-4 becomes:

$$\frac{\ell_c}{a} = \frac{\alpha}{1+\alpha} \frac{\sigma_{fu}}{\bar{\tau}} \tag{2-6}$$

where α is the ratio between the largest and smallest lateral dimensions of the assumed fiber shape and a, the smallest dimension. For the experimental work completed in this study, α = 2 and therefore:

$$\frac{\ell_c}{a} = \frac{2}{3} \frac{\sigma_{fu}}{\bar{\tau}} \qquad (2\text{-}7)$$

A comparison of theoretical and critical fiber lengths will be presented later.

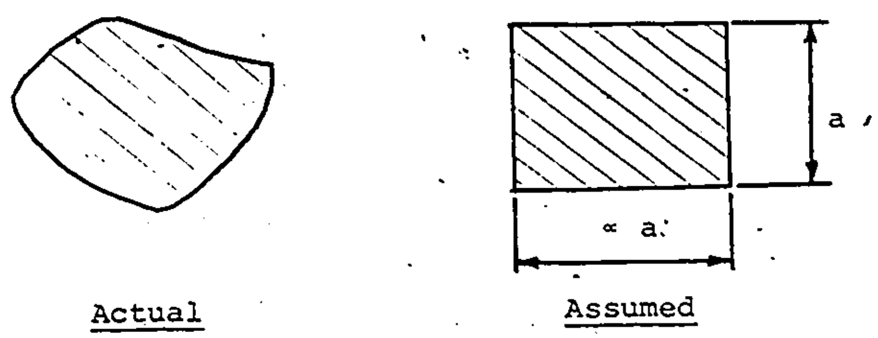


FIGURE 2.7 ACTUAL AND ASSUMED FIBER SHAPES

2.5 PULL-OUT TEST

The simplest method to determine the critical fiber length is the direct pull-out test. In many previous studies concerned with fiber-matrix stress transfer, the bond strength was found by pulling a single fiber or wire. However, since the bond strength of the fiber-matrix interface tends to be low, a more efficient manner of testing bond strengths developed by Hughes and Fattuhi [1975] was adopted for the study. This apparatus not only allows measurement of single fiber strength but also group fiber strengths and related fiber efficiency.

Although much information can be gained from single fiber pull-out tests, the test has been criticized on the grounds that it cannot reproduce actual stress field encountered in most composite applications, and the lack of ability to adequately simulate fiber-fiber interactions which are also very important to the behaviour of steel fiber reinforced concrete. Naaman and Shah [1975] also indicated that the efficiency of fiber groups decreases significantly, particularly when the fibers are inclined with respect to the applied stress field. Therefore, an increase in the fiber-matrix bond strength has not necessarily given the anticipated increase in composite strength behaviour [see, e.g., Naaman and Shah, 1975; and Pinchin and Tabor, 1978B]. Fiber-matrix bond strengths from various investigations are summarized in Table 2.1.

2.6 EXPERIMENTAL PROCEDURE AND TESTING

It has been indicated previously that the development of sufficient bond strength is of prime concern with regard to the performance of steel fiber reinforced concrete. Although many factors influence the fiber-matrix stress transfer, practical applications of the composite have shown that in addition to fiber concentration, the fiber length is of considerable importance due to its effect on both the mechanical

TABLE 2.1

REVIEW OF FIBER-MATRIX PULLOUT LOADS

Matrix	Fiber (mm)	Curing	Pull-Out Load (N)	Pullout Load Per Length (N/mm)	Reference
Mortar W/C=0.4	0.3x0.3x30	Water	68.32	2.28	Takazuka et al. [1947]
Mortar W/C=0.4	0.8 x 30	Water	141.16	4.71	Takazuka et al. [1977]
Mortar W/C=0.55	0.4 x 25.4	-	40[1]	3.20	Naaman & Shah [1975]
Mortar W/C=0.5	0.38 x 10.0	-	39.7	3.97	Maage [1977]
Cement Paste W/C=0.30	Duoform[2] .64 x 60	Fogroom	487	8.12	Hughes & Fattuhi [1975]
Cement Paste W/C=0.30	Wire .69 x 50	Fogroom	80	1.60	Hughes & Fattuhi [1975]

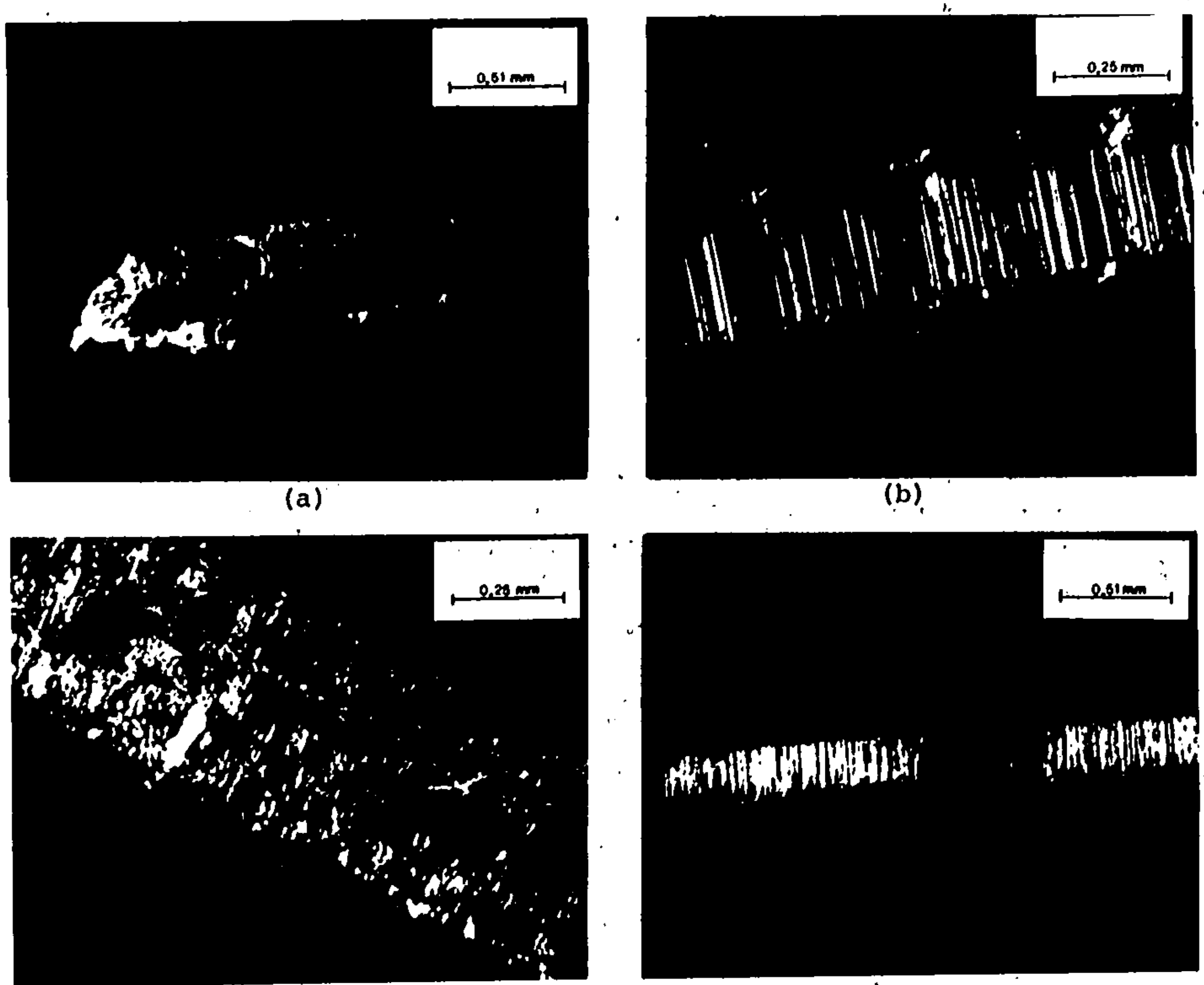
[1] Pullout Load Per Fiber

[2] Fiber with Positive Anchorages.

and rheological properties which are discussed in the following chapter. To increase the efficiency of the fibers with respect to the concrete's strength properties and workability, the prime factor considered throughout was the optimum fiber length; believed to be the critical fiber length. The influences of fiber strength, paste properties, aggregate gradation and curing duration and method were also of concern.

2.6.1 MIXES AND MATERIALS

Two types of fibers were provided by Stelco for the fiber pull-out tests. These fibers varied with respect to their strength properties: one having isotropic properties with a tensile strength of 380 MN/m^2 (55ksi); and the other having anisotropic properties with a tensile strength of 690 MN/m^2 (100ksi). The fibers were supplied in 15.24 cm lengths produced by a prototype cut sheet process, which could then be trimmed to vary the embedment length, i.e. half the fiber length. The cross-sectional areas of both fiber types were similar; the dimensions being 0.25 mm x 0.51 mm before shearing deformation during the manufacturing process as shown in Figure 2.7. Micrographs of the 380 MN/m^2 fiber are given in Figure 2.8 where variation of roughness along the fiber shows up clearly. The cut face in Figures 2.8 (b) and (d) can be easily distinguished from the sheet face in Figures 2.8 (a) and (c). The higher strength fibers were similar in appearance



(a) (b) (c) (d)

FIGURE 2.8 Various Micrographs of Fibers After Pull-out except that the cut face did not appear as rough.

Symbol 10 normal Portland cement supplied by the St. Lawrence Cement Company was used throughout. Two bags were sealed in plastic in order to retard the hydration of the cement due to moisture in the air, and it is considered that the cement was of consistent chemistry and quality for the program. The testing was done within a six month period of which up to three months were required for curing.

Two graduations of silica sand aggregate as given in Table 2.2 were used: Barnes 40 sand and ASTM C109 silica sand.

TABLE 2.2

GRADATION OF BARNES 40 AND ASTM C109 SILICA SANDS

US SEIVE	Grain Size (mm)	Percent Passing Barnes 40	Percent Passing ASTM C109
16	1.2	100	100
30	0.50	89	98 ± 2
40	0.41	20	70 ± 5
50	0.29	9	25 ± 5
100	0.15	2	2 ± 2

The Barnes 40 silica sand was used in order to compare the current test results with tests completed the previous year by W. Scott. The only difference between the two test series was the curing method adopted; moist curing by Scott and curing under water in the current study. The ASTM C109 silica sand was used for the remaining tests in order to comply with the usual test methods given in the technical literature.

The water-cement ratio (0.45) and the sand-cement ratio (2.0) were kept constant throughout the testing program except for Series A (testing the influence of the curing method using Barnes 40 silica sand), Series G (testing the influence of paste) and Series H (testing the influence of water-cement ratio). The mixes and curing processes adopted for each mix are summarized in Table 2.3.

2.6.2 PREPARATION AND TESTING OF SPECIMENS

The test sequence for determining the optimum fiber length is shown pictorially in Figure 2.9 and a more detailed schematic flow chart is given in Figure 2.10. The initial tests conducted in Series A consisted of specimens using Barnes 40 silica sand as aggregate and a water-cement ratio of 0.50. The specimens were cured for 28 days in a lime water bath. The variable considered in Series A was embedment length, however, comparisons were also made with Scott's results, given in Appendix 1, to determine the influence of curing method on embedment length. Details for test Series B through I can be developed in a similar way from the descriptions given in Tables 2.3 and 2.4.

TABLE 2.3

DETAILED DESCRIPTION OF THE VARIOUS TEST SERIES

Series	Number of Specimens	Fiber	Range of Embedment Lengths (mm)	Water Cement	Sand Cement	Sand	Curing
A	40	Low Strength[1]	6.4-12.7	0.50	2.0	Barnes 40	Water Bath
B1	6	Low Strength	6.4	0.45	2.0	C109	Oven
B2	59	Low Strength	6.4-16.0	0.45	2.0	C109	Oven
D	6	Low Strength	16.0	0.45	2.0	C109	Autoclave
E	72	High Strength[2]	6.4-28.6	0.45	2.0	C109	Autoclave
F	24	Low Strength	6.4-16.0	0.45	2.0	C109	Moist Room
G1	6	Low Strength	6.4	0.45	2.0	Barnes 40	Autoclave
G2	3	Low Strength	6.4	0.45	2.0	C109	Autoclave
G3	3	Low Strength	6.4	0.45	1.8	C109	Autoclave
G4	3	Low Strength	6.4	0.45	2.2	C109	Autoclave
G5	3	Low Strength	6.4	0.50	2.0	C109	Autoclave
H	6	Low Strength	9.5	0.45	4/3	Barnes 40	Water Bath
I[3]	52	Low Strength	6.4-12.7	0.45	2.0	C109	Moist Room

[1]Low strength fiber has tensile strength of 380 MN/m^2.

[2]High strength fiber has tensile strength of 690 MN/m^2.

[3]Duration of curing is the variable.

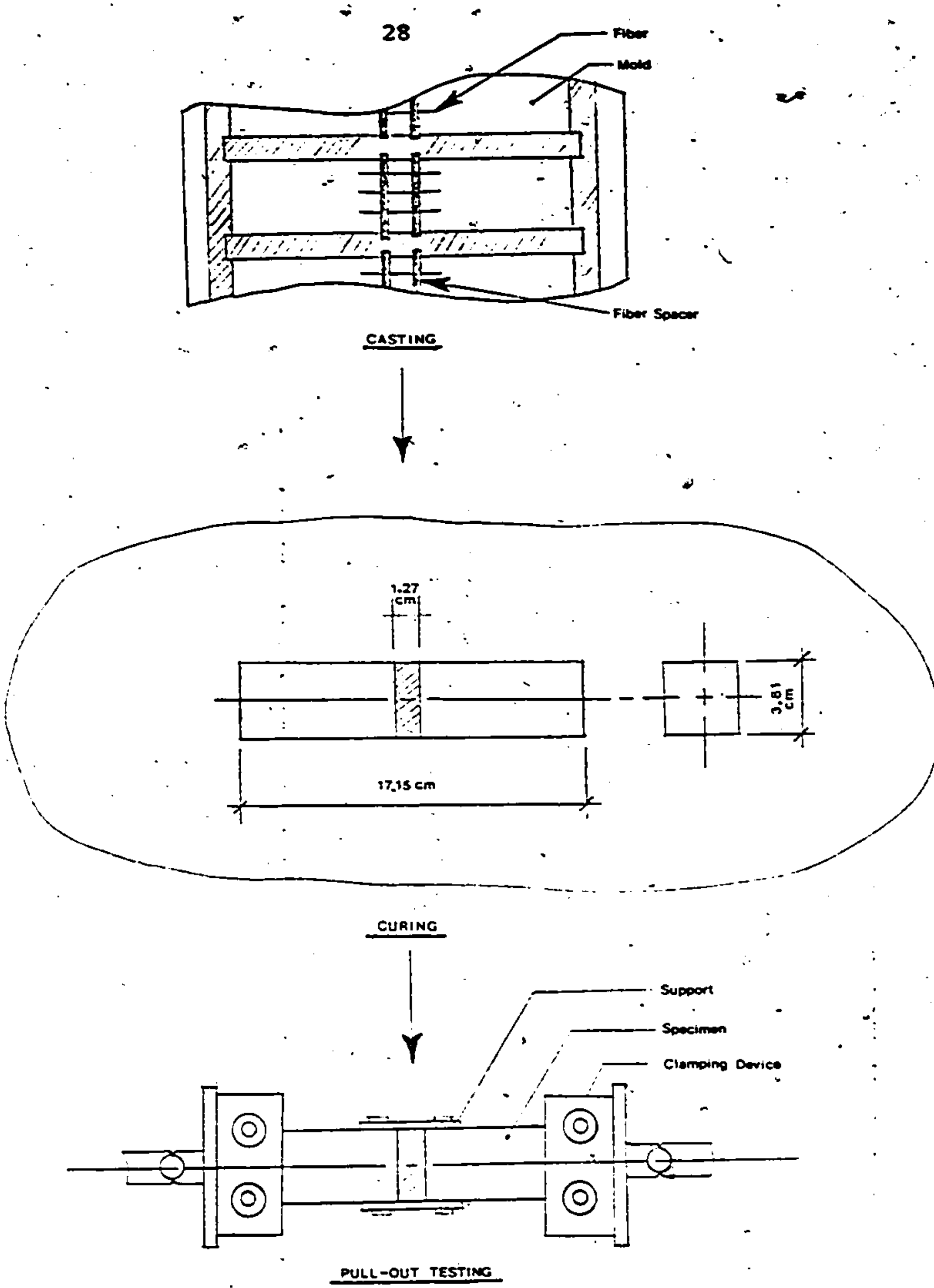


FIGURE 2.9 Pictorial Representation of Testing Procedure

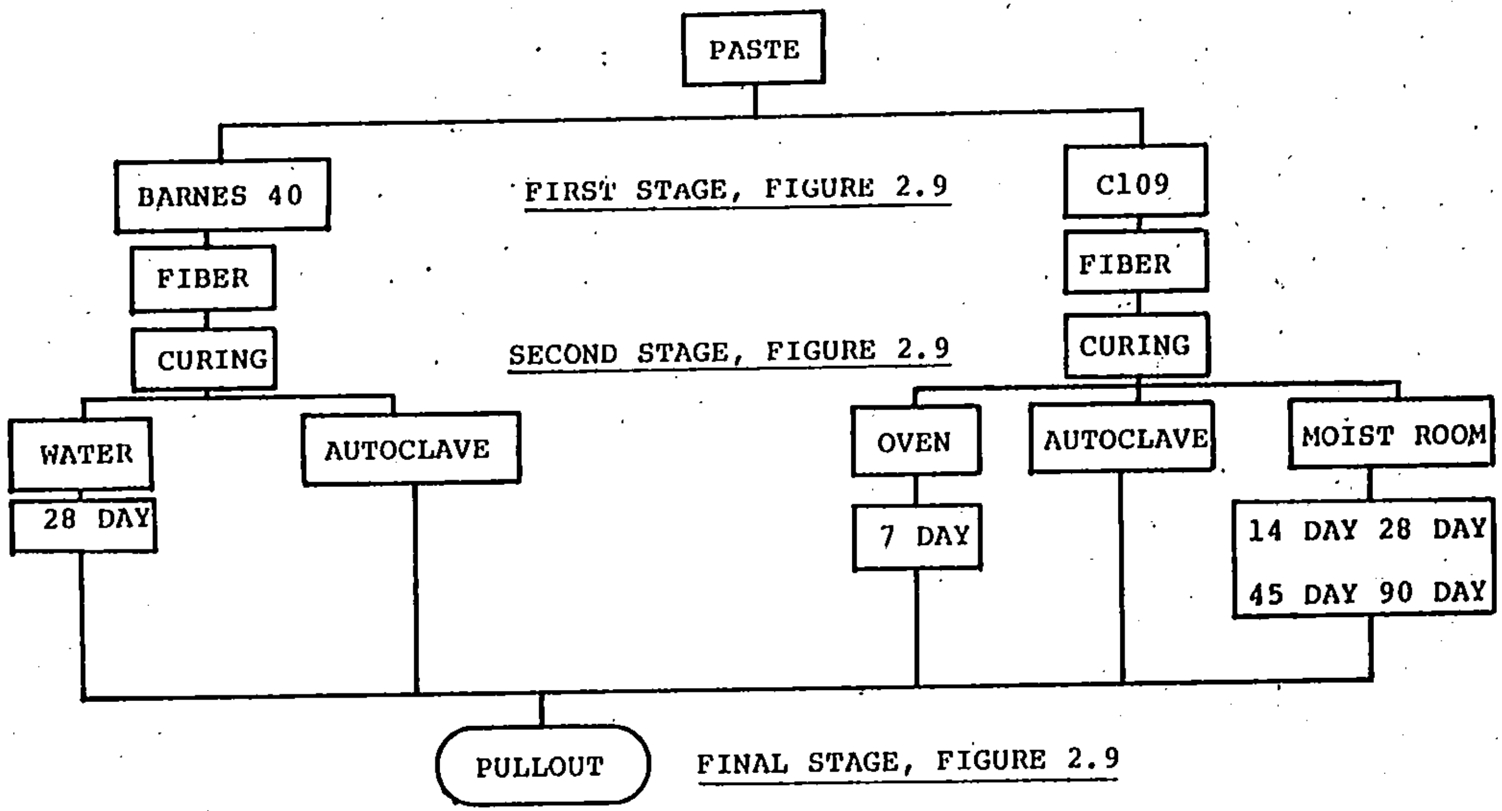


FIGURE 2.10 FLOW CHART ILLUSTRATING WORK DONE FOR PULL-OUT TESTS

TABLE 2.4

DETAIL OF CURING

Curing	Curing Environment	Temperature (^{o}C)	Curing Period
Water Bath	Hydrated lime solution	21	28 days
Oven	Specimens sealed in plastic bag containing some water to saturate environment	41	7 days
Moist Room	Saturated water vapour environment	21	28 days
Autoclave	Saturated water vapour environment pressurized at 1.0 MN/m^2 (150psi)	182	4 hours

While each of the various curing processes have been mentioned, further details of each curing process should be given here. All specimens were initially placed in the moist room for one day after casting of the matrix. This allowed strength gain adequate for each of the specimens to be removed from their molds and curing was then completed in the various environments described in Table 2.4.

All mixing operations were in accordance with ASTM C109. The initial stage shown in Figure 2.9 represents the casting of the specimens which involved the following steps:

1. Casting fibers cut to a predetermined embedment length into a plaster plug consisting of sand and hydrostone. The plug held 9 fibers all aligned parallel to one another and spaced at 7.94 mm centers on a 3 x 3 grid. Nine fibers were used as previous research indicated that this number gives a representitive quantity of pull-outs or breakages along with a reasonable strength level which could be recorded by the Hounsfield Tensometer;
2. Placing a film of paper adjacent to the hydrostone plug after the plug hardened in order that no bonding would occur between the plug and matrix; and
3. Casting the matrix around the fibers. The plugs holding the fibers were centered and orientated along the axis of the mold. The mortar mixed in Hobart mixer was then placed in two layers and compacted manually. The specimens were then cured as described above.

In the final stage, the pullout tests were completed using the modified Hounsfield Tensometer shown in Figure 2.11. The damping device shown in Figure 2.12 was advanced until

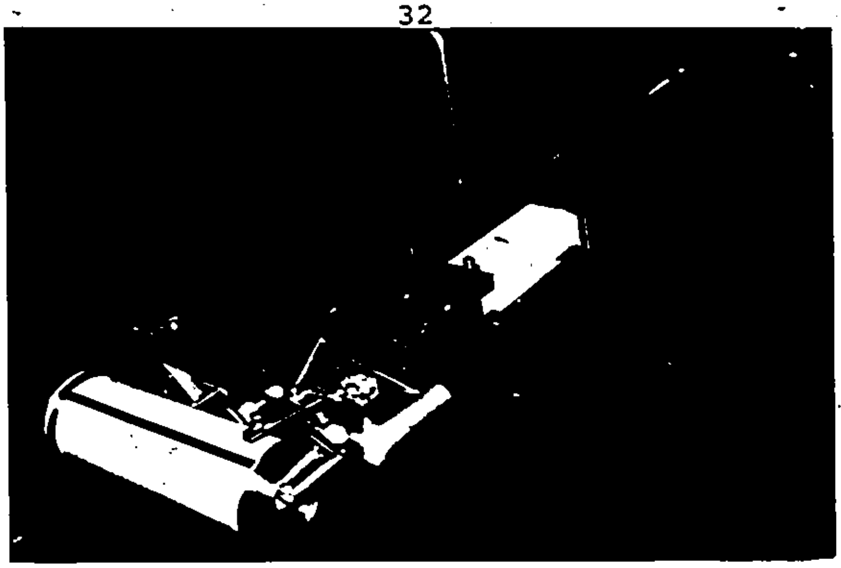


FIGURE 2.11 Modified Hounsfield Tensometer Used for Pull-out Tests

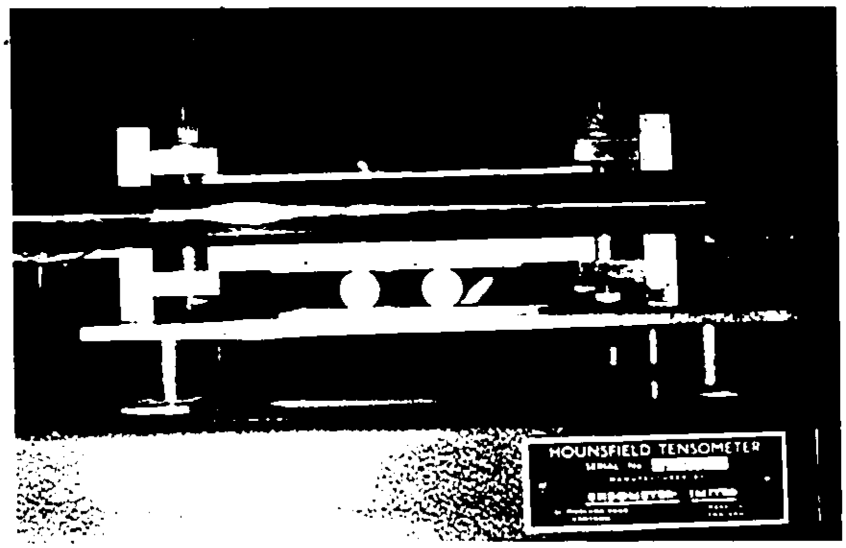


FIGURE 2.12 Closeup of Specimen Clamping Device and Vertical Supports

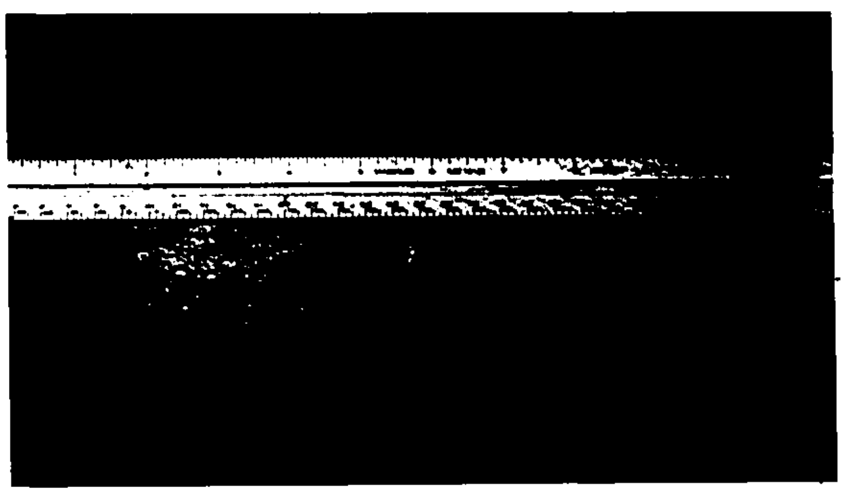


FIGURE 2.13 Parts of a Typical Specimen After Pull-out Failure

each jaw could grip 19 mm of the specimen which ensured that the specimen length during testing remained constant throughout. The Tensometer was operated manually at a displacement rate of approximately 1.0 mm/min. After failing the specimen the peak load was recorded and the fibers were checked for signs of corrosion which might have taken place during the curing process. The parts of a typical specimen after complete pull-out failure are shown in Figure 2.13.

2.7 DISCUSSION OF RESULTS

2.7.1 GENERAL

Several variables were of interest in this study, but due largely to corrosion of fibers between the hydrostone plug and matrix interface a number of test results could not be used. Those test results of interest are summarized in Appendix 2 and discussed in the following sections. Rusting of fibers occured primarily in the groups which were cured in the moist room, an atmosphere very favourable to such a process. It was also found that other difficulties arose in controlling the uniformity of the embedment length which is very important with respect to the actual mode of failure of the pull-out specimens. Embedment length and rusting of fibers were considered to be the prime factors influencing experimental scatter.

Several terms which are used in the discussion will now be defined to avoid confusion. As indicated previously, for most of the testing program, the variable of interest was embedment length which allows determination of the critical embedment at which pull-out failure changes to failure of the fiber in tension. In determining the strength characteristics of the fiber-matrix interface, load was measured directly with the tensometer. The load measured before failure reflected the development of bond shear strength. At failure, full bond shear strength can only be developed if all the fibers pull out rather than fracture, and the load at which this occurs is termed the pull-out load. The "residual strength" measured by the tensometer after failure reflects frictional shear strength.

2.7.2 INFLUENCE OF FIBER LENGTH

The effect of altering the fiber length on pull-out load was investigated under the conditions given in Table 2.5 for some of the series summarized in Table 2.3.

The fiber pull-out loads for Series A were lower than those recorded in the other series using the low strength fibers. The change from fiber pull-out to fiber fracture was not sudden as may be noted from the gradual transition in the number of breakages with increased embedment length shown in Figure 2.14. The optimum fiber length for Series A was between

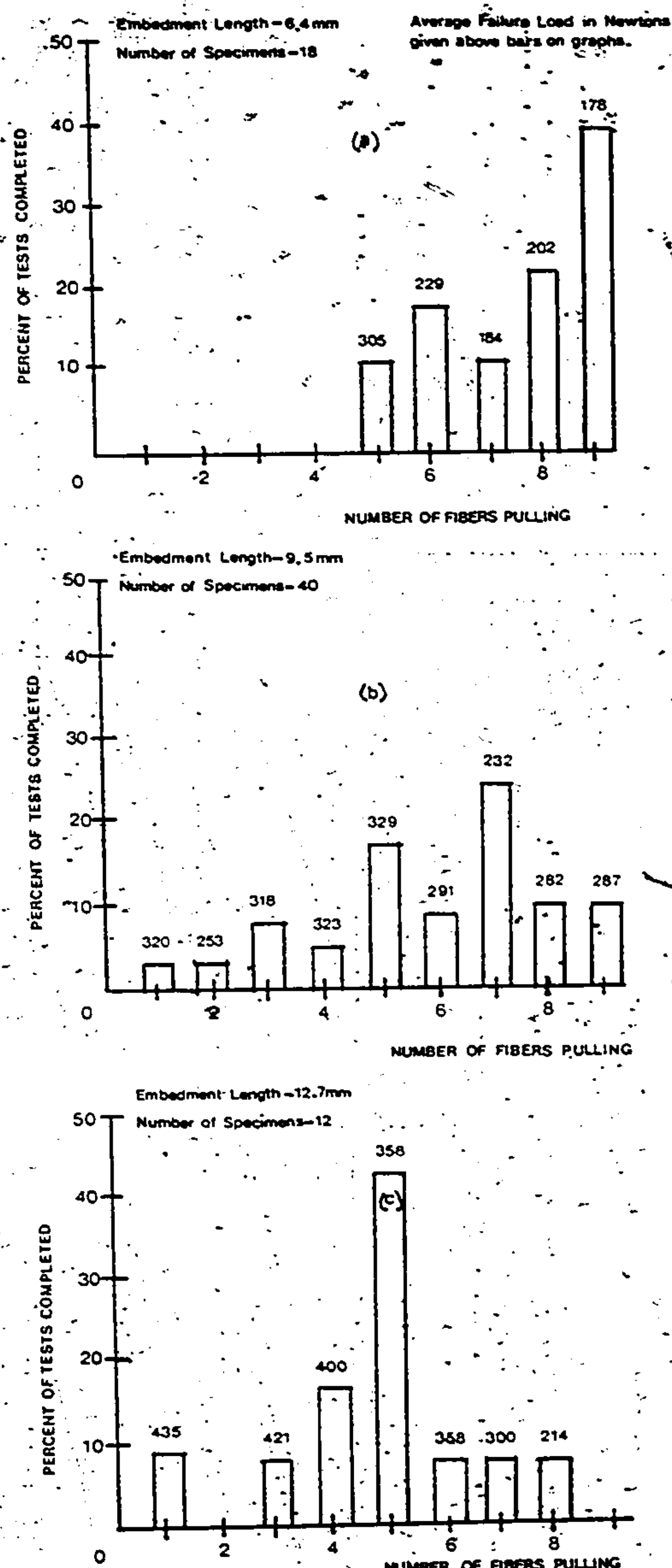


FIGURE 2.14 Results of Pull-out Tests for Series A

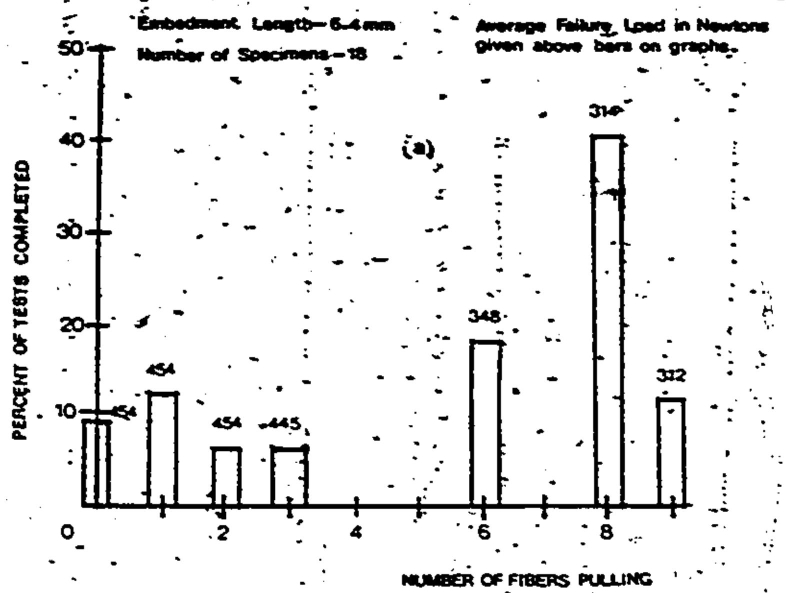
TABLE 2.5

DESCRIPTION OF SERIES FOR INVESTIGATION OF OPTIMUM EMBEDMENT LENGTH

Series	Aggregate	Fiber	Curing
Scott	Barnes 40	Low Strength	Moist Room, 7 days
A	Barnes 40	Low Strength	Water Bath
B	C109	Low Strength	Oven
E	C109	High Strength	Autoclave
F	C109	Low Strength	Moist Room

1.27 and 1.91 cm. Although results obtained by Scott were similar, the pull-out loads recorded by Scott were almost two times higher than those recorded in the current study. This indicated that the fibers were either damaged during casting or not of the same quality as specified by the manufacturer, since close examination of fractured fibers with suitable optical magnification showed limited corrosion.

The oven-cured specimens from Series B also indicated that the optimum fiber length ranges between 1.27 and 1.91 cm. Bond shear strengths were higher than the strengths recorded in Series A, and the change from fiber pull-out to fiber fracture was more pronounced as shown in Figure 2.15. The fibers developed tensile strengths close to the specified ultimate fiber strength (380 MN/m^2) as anticipated. A



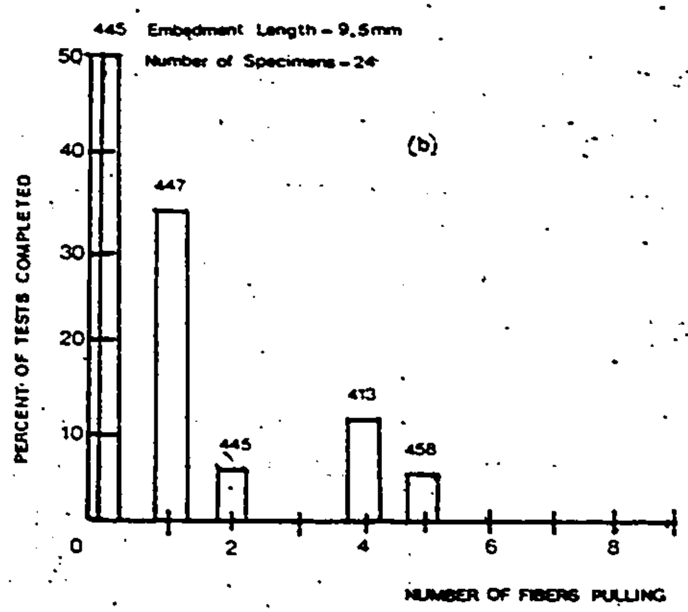


FIGURE 2.15 Results of Pull-out Tests for Series B

micrograph of a typical low strength fiber fracture from Series B indicating limited corrosion and ductile fiber failure is shown in Figure 2.8(a). Close examination of the results from both Series A and B indicated that although not monotonic, the number of fiber breakages increases with increased failure load.

The remaining two test series for the study of critical fiber length, i.e. Series E and F, were of limited value with regards to determining an optimum fiber length due to the high degree of fiber corrosion at the hydrostone plug-matrix interface. This corrosion resulted in premature tensile failures. Additional problems for Series E also developed due to the fibers' inherent anisotropic properties and increased non-uniform compaction with increased fiber length. If a force component was applied transversely, premature failure could occur as the fibers were weak in bending. With increased fiber length, the mortar could not be properly compacted between the fibers without causing fiber damage which resulted in lower pull-out loads than obtained with shorter fiber lengths. A comparison between Scott's and the autoclaved results (Series E) indicates the influence of anisotropic properties and non-uniform compaction on specimen failure as shown in Figure 2.16.

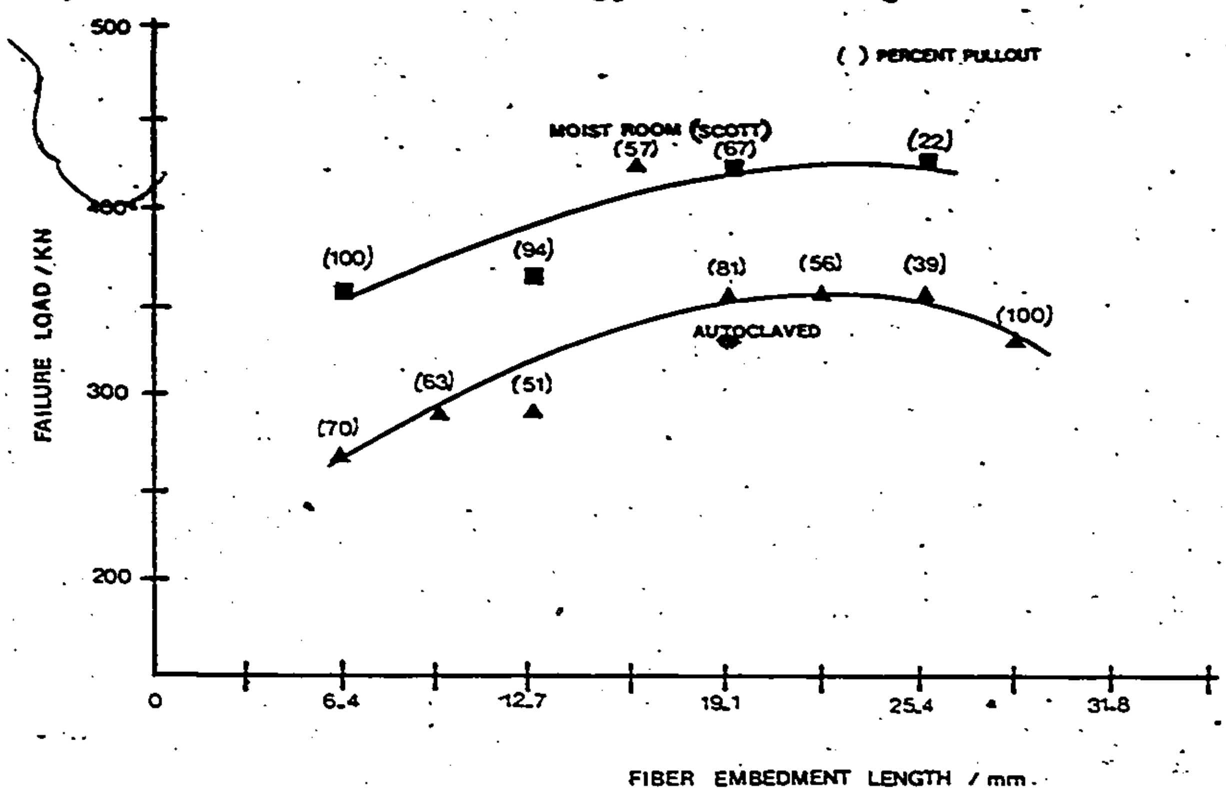


FIGURE 2.16 A Comparison of Scott's Results and Results Using Autoclaved Specimens from Current Study

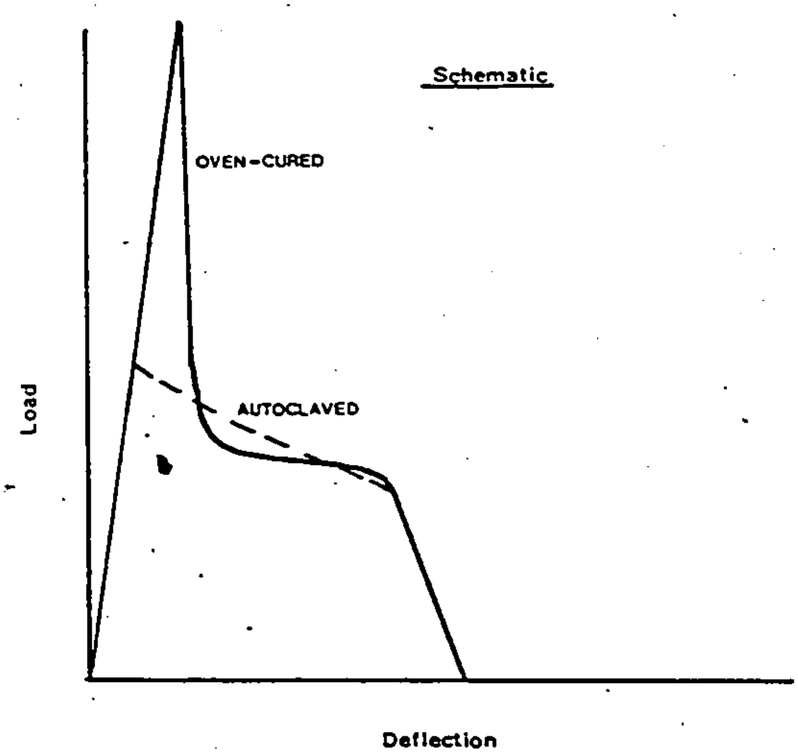


FIGURE 2.17 Comparison of Oven-cured and Autoclaved Load-deflection Curves

2.7.3 INFLUENCE OF CURING ON PULL-OUT STRENGTH

To determine the effect of curing on pull-out strength, short embedment lengths were used to reduce bond development in order that the bond shear strength could be compared. Scatter in test results was believed to stem from two causes: the fibers were not all equally efficient due to non-uniform embedment length; and the fibers themselves contained imperfections due to corrosion, damage while casting or non-uniform quality as manufactured. Due to the experimental scatter, many of the test results were difficult to interpret.

A comparison of different curing environments indicated that the autoclaved specimens had the lowest pull-out loads (bond shear strengths) no matter which aggregate was used as shown in Table 2.6. In addition to having lower pull-out loads the autoclaved specimens did not respond as dramatically to the transition from bond shear strength to frictional shear strength as shown in Figure 2.17. Although the curve for autoclaved specimens shows a decrease in load upon "bond failure", calculations indicated that the frictional shear strength subsequent to debonding continued to increase with decreased fiber embedment. This would suggest that severe surface disruptions occurred at the fiber-matrix interface during curing and the initial load build-up was due to friction rather than bond strength.

TABLE 2.6

EFFECT OF DIFFERENT CURING ENVIRONMENTS

Series	Embedment Length (cm)	Number of Tests	Number of Fibers Pulled[1]	Curing	PULL-OUT LOAD (N) Min.	Max.	Average
ASTM C109 SAND							
B2	0.64	2	9	Oven	302	320	311
G2	0.64	3	9	Autoclave	89	116	98
I	0.64	2	9	Moist Room (90 days)	209	303	256[2]
BARNES 40 SAND							
A	0.64	6	9	Water Bath	85	254	171
A	0.95	5	9	Water Bath	62	454	299
Scott	0.64	2	9	Moist Room (7 days)	153	405	281
Scott	1.27	2	9	Moist Room (7 days)	248	428	338
G1[3]	0.64	6	9	Autoclaved	-	-	66.75

[1] Only the specimens in which all fibers pulled were selected for comparison.

[2] Moist curing at 28 and 45 days had much higher bond strengths, i.e., 431 N.

[3] Autoclaved specimens have a W/C = 0.45.

A comparison of oven and moist curved specimens indicated that for specimens in which 9 fibers pulled out, the oven cured specimens had higher bond shear strengths. However, it should be noted that the results given in Table 2.6 only show pull-out loads for 100 percent pull-out; other specimens in which fiber failure occured showed similar bond strengths for the two curing environments. One further comparison between water and moist room cured specimens (Scott) indicated that water curing tended to reduce the pull-out load. This would suggest that curing extremes, i.e. autoclaving or water curing, decreases bond shear strength.

The influence of curing time on pull-out load shown in Table 2.7 indicates that the bond shear strength initially developed by 28 days decreased between 28 and 90 days for the 0.64 cm fiber embedment length. However, for the longer fibers (0.95 cm embedment length) the bond shear strength appeared to increse slightly with curing time. This would suggest that the curing length does influence the bond strength, however, the relationship between bond strength increase or decrease cannot be determined due to incomplete data resulting from the corrosion and high bond strength developed over the 0.64 cm embedment length.

TABLE 2.7

EFFECT OF CURING TIME

Series	Embedment Length (cm)	Number of Pull	Number of Specimens	Length of Curing (Days)	Average Pull-out Load (N)
I1	0.64	9	2	90	255.88
		7	3	90	358.85
		6	1	90	320.40
		5	2	90	340.43
		4	1	90	369.35
I2	0.64	4	1	45	391.60
		3	1	45	413.85
		2	1	45	431.65
		1	1	45	471.70
		0	1	45	431.65
I3	0.64	2	2	28	449.41
		1	5	28	445.5
		0	6	28	441.41
I4	0.64	1	1	14	350.97
		0	5	14	398.67
I5	0.95	8	1	90	324.85
		0	5	90	436.00
I6	0.95	5	1	45	378.25
		2	1	45	409.75
		0	4	45	422.75

2.7.4 INFLUENCE OF ALTERING MATRIX COMPONENTS

One of the necessary requirements to optimize bond shear strength is that sufficient paste be present for complete coating of the fibers. Series G and H were completed to examine the influence of changing the aggregate gradation, sand-cement ratio and water-cement ratio on pull-out load. Each of these variables influences the amount of paste present which coats the fibers. Table 2.8 summarizes the results from Appendix 2 which are directly related to changes in these variables.

The change in gradation was achieved by using the two different silica sands, Barnes 40 and ASTM C109, for the pull-out tests. Barnes 40 sand is coarser as indicated in Table 2.2, therefore it was expected that more free paste would be present for the coating of fibers than with ASTM C109 silica sand. Since the flow of mortar is also dependent on the amount of free paste, flow table tests were made in accordance with ASTM C230. The flow for Barnes 40 sand ranged between 115 and 118, whereas the ASTM C109 sand ranged between 112 and 116 which confirmed the paste availability trend. By comparing Series G1 and G2, the results show that ASTM C109 specimens had higher pull-out loads, thus indicating higher bond shear strength. This is the opposite of the expected behaviour, which would suggest that although sufficient free paste is required to coat fibers, too much free paste will reduce the bond shear strength. Similar observations were made by Hughes and Fattuhi [1975] when they showed that mortar specimens had higher pull-out loads than cement paste specimens.

A decrease in the sand-cement ratio also increases the free paste available to coat the fibers. From Series A and H, the results indicate that a sharp decrease in sand-cement ratio, i.e. 2/3 of the control mix, reduces the pull-out load by 50 percent. However, small decreases or increases in sand content, i.e. by 10 percent, result in small increases or decreases respectively in the pull-out load of the fiber as recorded in Series G2, G3 and G4. With regards to pull-out load, the optimum mix for pull-out tests occurred with a sand-cement ratio of 1.8.

A further comparison made between Series G2 and G5 in which the water-cement ratio was varied, indicated that an increase in water content reduces the bond shear strength. The water-sand ratio in Series G5 was the same as in Series G3 where optimum mix design occurred, therefore suggesting that the strength decrease was due to decreased paste quality. From the tests conducted in Series G, it is apparent that the pull-out strength is not significantly influenced by minor changes in the sand-cement ratio, however, the paste quality and aggregate gradation are important with regards to obtaining adequate bond shear strength.

TABLE 2.8

RESULTS FOR TESTS IN WHICH THE MATRIX CHARACTERISTICS WERE ALTERED

Series	Embedment Length (cm)	Number of Tests	Number of Pull-Outs	Aggregate	S/C	W/C	Curing	PULLOUT LOADS (N) Max.	Min.	Average
A	0.95	6[1]	9	Barnes 40	2.0	0.50	Water Bath	453.9	62.3	249.2
H	0.95	5	9	Barnes 40	1.33	0.50	Water Bath	205.19	11.13	127.7
G1	0.64	6	9	Barnes 40	2.0	0.45	Auto-clave	-	-	66.75
G2	0.64	3	9	C 109	2.0	0.45	Auto-clave	115.75	89.0	97.9
G3	0.64	3	9	C 109	1.8	0.45	Auto-clave	124.6	66.75	102.4
G4	0.64	2[2]	9	C 109	2.2	0.45	Auto-clave	89.00	81.6	84.5
G5	0.64	3	9	C 109	2.0	0.50	Auto-clave	120.15	53.4	75.9

[1] For test Series A only specimens in which 9 fiber debonded during the pullout test were referred to for comparison with results obtained in Series H.

[2] An additional specimen was failed at 253.65 N in which 8 fibers pulled out.

2.7.5 EFFECT OF HIGH AND LOW STRENGTH FIBERS ON PULLOUT

A minimal number of tests were completed to determine the influence of different fibers on the pull-out load. The results given in Table 2.9 indicate that the high strength fibers had greater bond strengths. This observation is contrary to what was expected since the lower strength fibers had rougher surfaces. This fiber roughness occurs at the microscopic level, i.e., a magnification of 4X. However, during the manufacturer's shearing of the 15.25 cm long fibers, they are twisted. This twist along the fiber may have a more pronounced influence on the pull-out load for the high strength fibers than the low strength fibers. In addition the lateral deformation of the fiber due to anisotropy may not be as great for the higher strength fiber which would also account for the higher pull-out loads, i.e. fiber misfit not reduced as much.

TABLE 2.9

EFFECT OF FIBER STRENGTH ON PULL-OUT LOAD

Series	Embedment Length (cm)	Number of Tests	Fiber	Number of Fiber Pulled	Pull-out Load (N)
E	0.64	3	High Strength	9	232.88
G2	0.64	3	Low Strength	9	97.67

With the higher strength fibers, a larger selection of embedment lengths could be tested. The results for the pull-out tests with these fibers are shown in Figure 2.18. The strength of specimens increase up to the 1.59 cm fiber embedment length, and then fall off. This decrease in specimen strength is believed to result from insufficient compactibility of the matrix around the fibers. The specimen strength here refers to failure by pull-out or fracture of the fibers. It is also believed that as the length of the fibers goes up, more fiber damage occurs which also reduces the specimen strength. Scott also experienced the same trend of decreased specimen strength with increasing fiber length, although he did observe a critical length.

2.7.6 FIBER BOND STRENGTHS

Previous discussion focused on the relative bond shear strengths, i.e. pull-out load, of the various specimens for a number of variables. Emphasis in this section is on the variation of shear strength at the fiber-matrix interface. The two mechanisms reponsible for the development of shear strength have already been defined in Section 2.7.1. They are the bond shear strength, i.e. pull-out load, and the frictional shear strength.

A study of the fiber-matrix stress transfer for Barnes 40 sand specimens indicated that the stress transfer

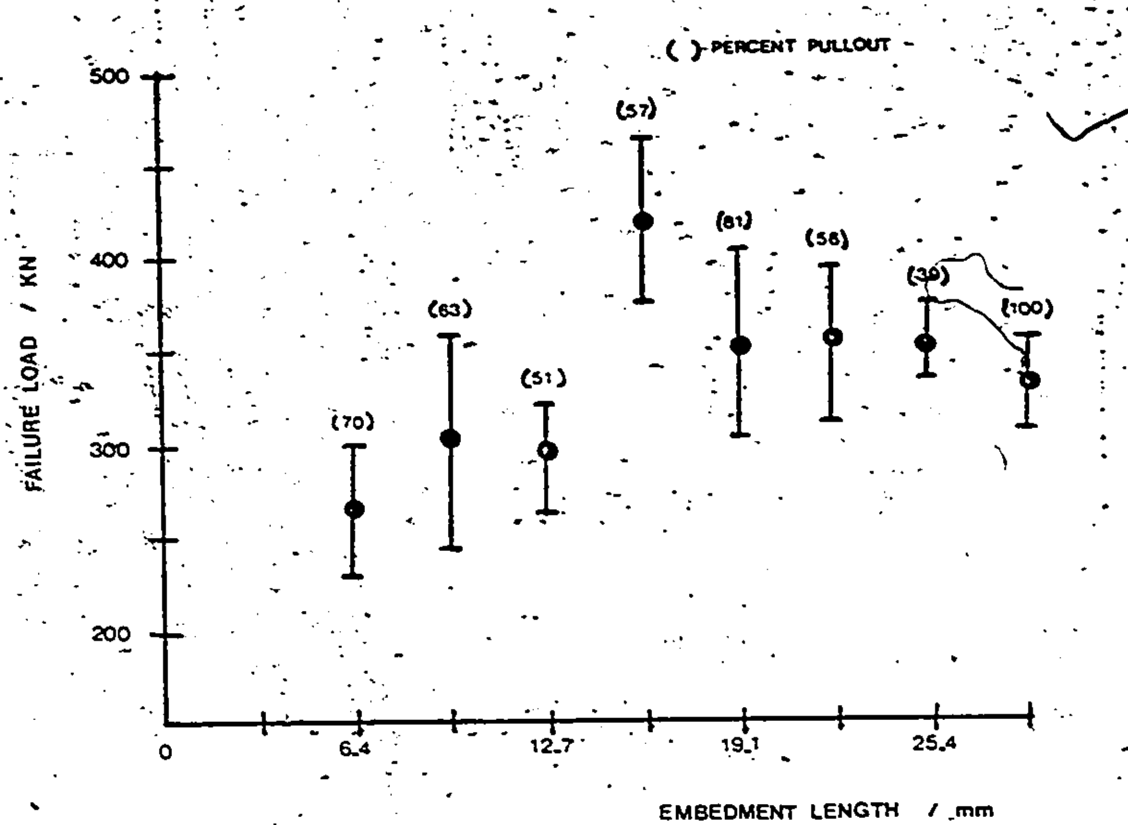


FIGURE 2.18 Results of High Strength Fiber Pull-out Tests

built-up to a maximum at the pull-out load, i.e. bond shear strength, then dropped rapidly releasing energy after debonding as shown in Figure 2.19. Pinchin and Tabor [1978B] expressed concern about the interpretation of this energy release, particularly for rough fibers where the shearing of paste adjacent to the fibers is required before relative movement between the fibers and matrix occurs; for smooth fibers the transfer from bond to frictional strength is more gradual. They attributed this quick release of energy to the inherent flexibility of the testing apparatus which can be overcome with the use of a stiffer or servo controlled testing system. The influence of the apparatus rigidity is reflected by the initial slope of the load-deflection curve given in Figure 2.20. This flexibility must be accounted for when calculating the frictional shear strength during pull-out as detailed in Appendix 3.

All tests showed a linear behaviour near the end of pulling which reflects the development of a constant frictional shear strength as shown in Figure 2.19. Pinchin and Tabor [1978C] also indicated that the actual frictional stress transfer is constant near the end of pulling which was attributed to the rough pulling out process causing localized failures as discussed previously. For the tests reported here, the frictional stress transfer varies between 1.50 and 5.00 N/mm^2 based on computations for several specimens.

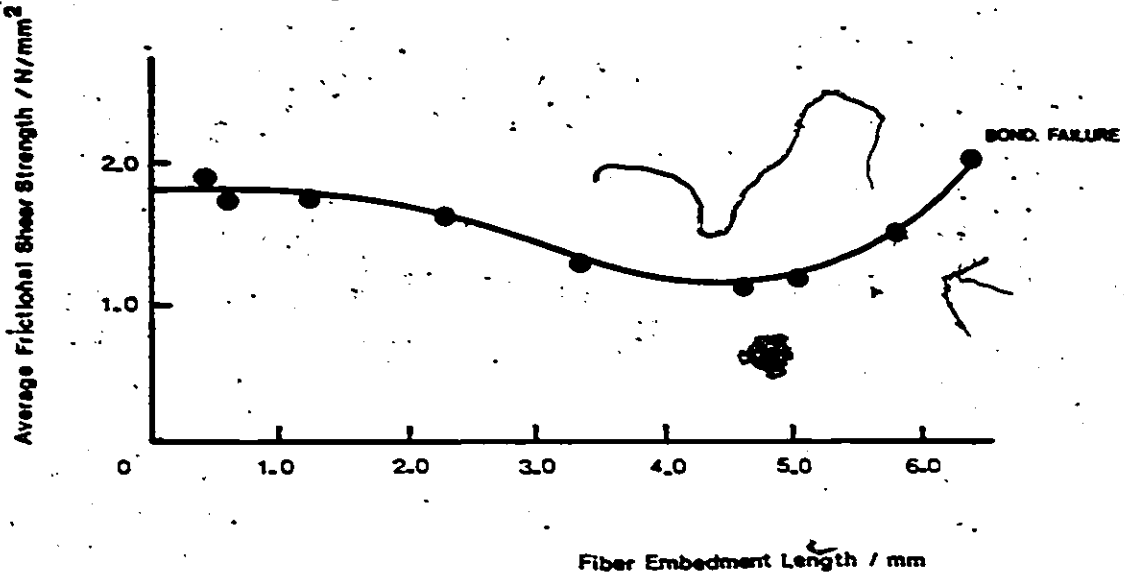


FIGURE 2.19 Shear-stress versus Embedment Length Curve Developed Using Load-deflection Curve from Figure 2.20

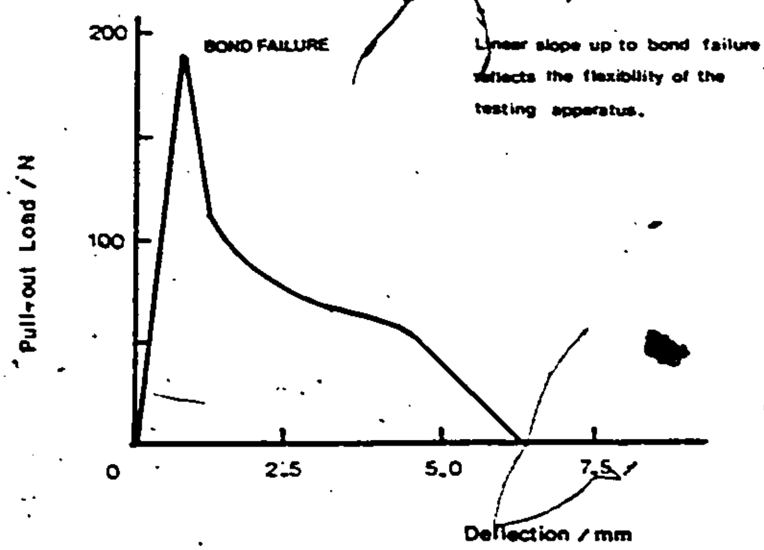


FIGURE 2.20 Average Load-deflection Curve for Specimens Having 6.4 mm Fiber Embedment Length

The frictional stress transfer values were largely dependent on the curing of specimens.

2.7.7 THEORETICAL CRITICAL FIBER LENGTH

The discussion so far has been descriptive in relating the influence of certain parameters on the bond and frictional shear strengths. The optimum fiber length determined experimentally was found to range between 1.27 and 1.91 cm. However, the optimum length can vary depending upon the fiber, mix and curing method used. To calculate the critical fiber length, the variables σ_{fu} and $\bar{\tau}$ was selected from tests completed in Series B, for comparison with the experimental optimum length determined in Series B.

The variables required to determine the critical fiber length using Equation 2-7 given in Section 2.4 include:

Minimum fiber dimension = 0.25 mm

Fiber ultimate strength = 380 N/mm^2

Calculated bond strength= 3.55 N/mm^2

which gives a critical length of 1.8 cm. This compares well with the experimental results. It should be noted that the water cured or autoclaved specimens which had lower bond strengths have longer critical fiber lengths.

2.8 SUMMARY

It was shown that for plain fibers, i.e. without mechanical anchorages, the bond shear strength is dependent on:

1. fiber length - The pull-out load increases with an increase in fiber length. A comparison of fiber tensile strengths for fibers from the same source indicated that the fiber strengths varied. This variation could have resulted from mechanical damage of fibers during the casting of plugs.
2. matrix characteristics - Although sufficient free paste is required to coat the fibers, a change in gradation, sand-cement ratio or water-cement ratio which increases the free paste available over an adequate amount to coat the fibers leads to a decrease in pull-out load. The optimum mix occurred for sand and water to cement ratios of 1.8 and 0.45 respectively.
3. curing method adopted - Curing time and environment both influence the fiber pull-out load. The oven and moist room cured specimens had higher bond strengths than water cured or autoclaved specimens. This indicated that extreme curing could disrupt the fiber-matrix interface and lower the bond strength. Although not conclusive, the bond strengths for moist room specimens appeared to decrease for the specimens having a 0.64 cm fiber embedment length.

4. fiber type - It was shown that contrary to surface roughness observations the high strength fibers had higher bond strengths than the low strength fibers. This was attributed to the twist in the fibers due to the manufacturing process or possibly to anisotropy which would partially influence the lateral deformation of the fibers during pulling.

The load-deflection curves for the pull-out tests as recorded by the Hounsfield Tensometer are largely related to the compliance of the testing system which is reflected by the initial apparent elastic behaviour curve. The rapid release of energy upon debonding is also attributable to the flexibility of the tensometer. After accounting for the influence of the testing apparatus, all tests indicated a constant frictional stress transfer near the end of complete pull-out which is attributed to the rough pull-out process.

It has been indicated that the development of bond strength is very much dependent on the fiber mix and curing method. The theoretical critical fiber length for oven cured specimens which was predicted to be 1.8 cm compares well with the experimental optimum length which ranged between 1.27 and 1.91 cm. However, due to lower bond strengths, the water cured and autoclaved specimens have longer critical fiber lengths.

CHAPTER 3

FLEXURAL STRENGTH PERFORMANCE OF STEEL FIBER REINFORCED CONCRETE

3.1 INTRODUCTION

Researchers do not currently share the same opinions with respect to the function of fibers in improving the mechanical properties of concrete. Regardless of the function of the fibers, experimental data do indicate a significant improvement in the static and fatigue flexural strengths with increases in fiber concentration [Snyder and Lankard, 1972].

This study was not intended to investigate the flexural strengths of fiber concrete in detail, but to compare the performance of 1.27 cm and 2.54 cm long steel fibers. Comparisons made with respect to mechanical properties are presented in this chapter, and concrete flow properties are discussed in the next chapter. The static and fatigue flexural tests were carried out on standard double break prisms, 15.25 cm x 15.25 cm x 105 cm long (6" x 6" x 42"). In addition to these flexural tests, some compression, split-cylinder and impact tests were completed.

3.2 INFLUENCE OF STEEL FIBERS ON THE MECHANICAL PROPERTIES OF CONCRETE

Two simple theories have been developed to explain the influence of steel fiber reinforcement on concrete behaviour. The crack arrest theory (spacing concept) assumes that the fibers inhibit the propogation of cracks. This theory makes use of the criteria that more energy is required for a crack to grow with the inclusion of fibers [Parmi and Rao, 1973]. The composite theory uses an equilibrium approach to account for the forces across a cross-section, assuming that the fibers act as normal reinforcement [Batson et al., 1973]. More advanced theories using fracture mechanics exist that will not be given here [see, e.g., Weiss, 1973; Naaman et al., 1973; and Buckley and Everard, 1973].

3.2.1 EFFICIENCY OF FIBER REINFORCEMENT

The mode of failure for a fiber reinforced composite is largely dependent on its ability to transfer stress between the fiber and matrix. The desired mode of failure for steel-fiber reinforced concrete is by pull-out at the fiber-matrix interface which usually occurs at the ultimate strength of the concrete [Swamy et al., 1973].

Important factors contributing to the efficiency of the fibers as a reinforcement include: relative stiffness of the fiber and matrix; strain compatability of the fiber and matrix; fiber content; aspect ratio; interfacial bond strength discussed perviously; and fiber orientation [Swamy, et al., 1973]. To optimize the influence of fibers in tensile or flexural stress fields, the perferred orientation is in the direction of the applied stress [Hannant and Spring, 1974]. However for most practical applications, no definite control of fiber orientation can be made. Instead, the fibers are assumed to be randomly orientated and uniformly distributed so that the composite can be considered as having similar properties in all directions.

Further factors influencing the reinforcing properties of fibers are related to the fabrication and cost of the fiber reinforced products. In many of the current uses - pavement overlays, tunnel liners, and factory floors - the efficiency of fiber reinforcement is often influenced by technical ability to prepare, place and compact the fiber concrete. The degree to which any of the "negative" factors can be limited is largely dependent on the cost allowed for such applications [see, e.g., Swamy and Kent, 1973; Swamy and Stavrides, 1975; Lankard and Walker, 1975; Ounanian and Kesler, 1976; and Rilem, 1977].

3.2.2 EFFECTIVE FIBER LENGTH

In the previous chapter, emphasis was placed on obtaining the critical fiber length by applying a direct tensile stress field aligned with the fiber. However, due to the random orientation of the fibers in fiber reinforced concrete, the full length of any fiber may not be effective in the direction of the stress field. To account for the random orientation of all fibers, a statistical average for the fiber length in the direction of the stress field is required. This average projection is called the effective fiber length. Various proposed projection factors are given in Table 3.1.

TABLE 3.1

EFFECTIVE LENGTH FACTOR (ß)

REFERENCE	FACTOR (ß)
Romauldi and Mandel [1964]	.405
Krenchel [1975]	.500
Rarmi and Rao [1973]	.637

3.2.3 CRACK ARREST THEORY (SPACING CONCEPT)

The fracture toughness of concrete can be increased by a decrease in the size of inherent flows or a decrease in the intensity factor of the crack tips by the addition of fibers. Because of the greater elastic modulus at any composite strain, the steel fibers exert pinching forces at the crack tips. Romauldi and Batson [1963] showed that the pinching forces are a function of fiber spacing which is valid only up to the first crack strength; thereafter energy is dissipated through fibers pulling. In addition to affecting the mechanical properties, the fiber spacing also influences the concrete workability during mixing since the spacing affects the fibers' ability to rotate [Krenchel, 1975].

The spacing concept (i.e., crack arrest theory) relates an apparent geometric spacing of no physical significance to the percentage of steel fibers added. Romauldi and his co-workers showed both theoretically and experimentally that the stress required to extend a crack beyond the area enclosed by the adjacent group of fibers is inversely proportional to the square root of the fiber spacing as shown in Figure 3.1 [Snyder and Lankard, 1972]. However, the theory offers no unique theoretical means for predicting the state of stress at which failure occurs. Since the derivation of the spacing equation proposed by Romauldi and Mandel [1964], other similar spacing models have been developed with various modifications in the assumptions [see, e.g., Kar and Pal, 1972; Swamy et al., 1973; Parmi and Rao, 1973; and Krenchel, 1975]. Table 3.2 gives some of the different spacing formulae for the first crack strength; where d is the fiber diameter, p the percent volume of fibers and l the fiber length.

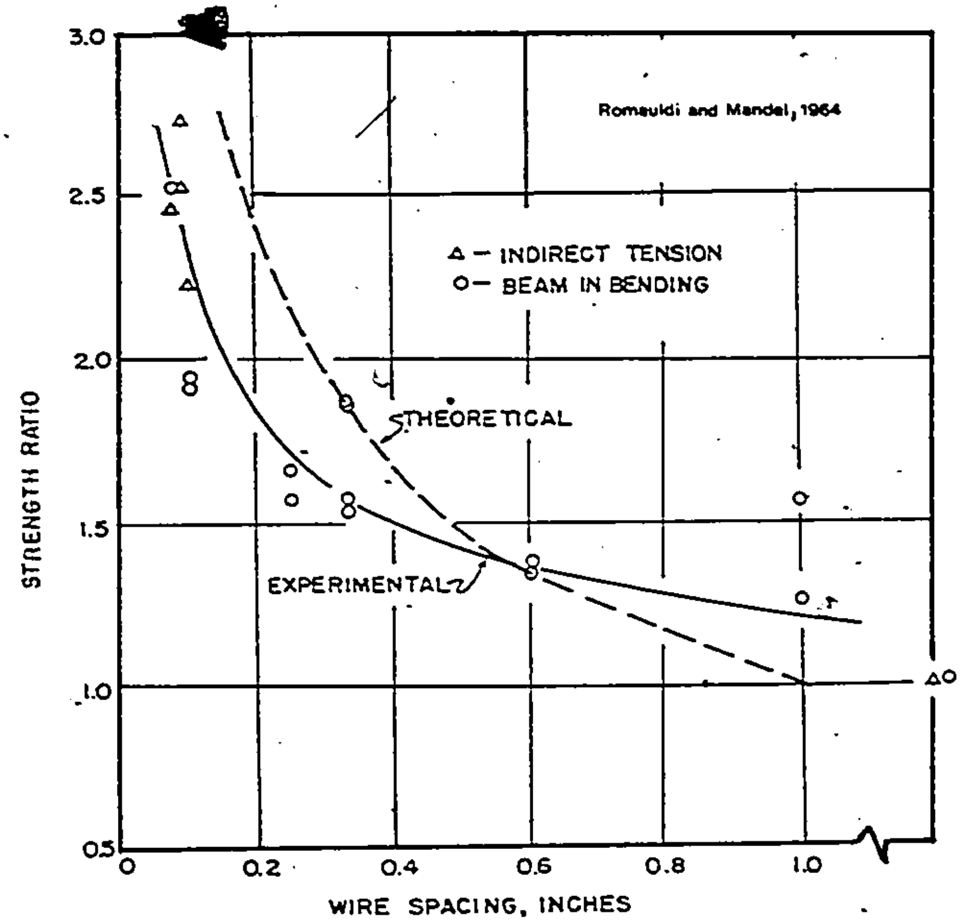


Fig. 3.1–Theoretical and experimental strength ratio as a function of wire spacing

TABLE 3.2

EFFECTIVE SPACING EQUATIONS FOR ROUND FIBERS

REFERENCE	β	EQUATION
Romauldi and Mandel [1964]	.045	$13.8\ d/\sqrt{p}$
Krenchel [1975]	.500	$12.5\ d/\sqrt{p}$
Parmi and Rao [1972]	.637	$11.1\ d/\sqrt{p}$
Swamy et al. [1973]	.045	$27.0\sqrt{d/pl}$

3.2.4 FATIGUE MECHANISM (CRACK ARREST APPROACH)

Bastson et al. [1972] proposed that a fatigue failure mechanism for concrete or mortar develops in three stages: initiation of flaws and cracks; slow growth of flaws and cracks to a critical size; and rapid propogation of flaws or cracks of a critical size. The first stage is an inferent property of the concrete over which little control can be exercised. To maintain the concrete's soundness the growth of cracks (i.e. microcracking) must be retarded or inhibited before development of a critical size.

The fatigue of concrete can perhaps best be understood using Orowan's explanation for the fatigue of metals [see, e.g. Dieter, 1961]. Concrete is assumed to contain areas of high stress concentrations due to inherent flaws which can be treated as plastic regions in an elasted matrix.

As a result of repeated load cycles (i.e. at loads less than those required to cause static failure) the plastic regions experience progressive localized strain hardening which uses up the composite's plasticity resulting in fatigue fracture. However, with the inclusion of fibers, the high stress concentrations are reduced by the pinching forces at the crack tips retarding crack growth.

In the development of the spacing formulae given in Table 3.2, the general format for the equations is given by Equation 3-1 where: S is the fiber spacing; A is the cross-sectional area; β is the effective length factor; and p is the percentage fibers by volume.

$$S = 10\sqrt{\frac{A}{\beta p}} \qquad (3\text{-}1)$$

This equation suggests that fiber orientation, concentration and cross-sectional area all influence the fiber spacing and related fatigue strength of the composite. Equally important in the development of Equation 3-1 is the assumption that the fiber-matrix interface is free of bond shear stress. This is only true for cases in which the fiber length is much longer than the critical fiber length as shown previously in Figure 2.4 [Swamy, 1973]. Therefore, aspect ratio, bond shear strength, and fiber tensile strength also influence the fatigue performance of a fiber reinforced composite.

3.2.5 COMPOSITE MATERIAL CONCEPT

The composite material concept offers a means of predicting a state of stress at which failure of the composite occurs. This concept makes use of the law of mixtures for long continuous fibers given by Equation 3-2:

$$E_c = E_f V_f + E_m V_m \qquad (3\text{-}2)$$

where: E_c, E_f and E_m are the elastic moduli of the composite, fiber and matrix respectively; V_f is the volume fraction of fibers; and V_m is the volume fraction of the matrix [Baston et al., 1973]. Since short discrete fibers are used for fiber reinforced concrete, efficiency factors are required to account for fiber orientation and discontinuity.

There are several theoretical approaches available to predict the influence of fiber orientation and fiber-matrix interface on the elastic properties of the composite. Swamy and Mangat [1974] developed a combined crack-control-composite materials theory for the flexural strength of concrete. The general expression for predicting the composite modulus of rupture using an orientation factor of .405 is given by Equation 3-3:

$$\sigma_{cb} = A\sigma_{mb} V_m + 0.82\ \tau_b V_f \left(\frac{l}{d} \right) \qquad (3\text{-}3)$$

where σ_{cb} is the composite modulus of rupture; σ_{mb} is the matrix modulus of rupture; V_m is the matrix volume fraction; V_f is the fiber volume fraction; τ_b is a factor reflecting the interfacial bond stress; l is fiber length; d is fiber diameter; and A is a factor taking into account the relationship between flexural and tensile strength for matrix and composite.

The composite equations do not appear to have gained wide acceptance mainly because little experimental data is available to validate the theoretical approaches. In addition, it is difficult to extend the composite theory to account for fatigue failure.

3.3 FLEXURAL TESTS

Many of the potential uses for steel fiber reinforced concrete make use of the composite's flexural strength. For this reason, much emphasis in the past has been directed towards research in the laboratory and field on the flexural strength of fiber reinforced concrete [see, e.g., Romauldi and Baston, 1963; Romualdi and Mandel, 1964; Swamy et al., 1973; and Parker, 1974]. A review of some tends observed by past investigators is given in the next section.

3.3.1 STATIC FLEXURAL STRENGTH

General trends associated with the static flexural strength of steel fiber reinforced concrete can be summarized as:

1. Significant increases in first crack and ultimate strengths can be achieved through increases in

fiber length and/or fiber concentration assuming adequate consolidation, fiber distribution and random fiber orientations. A decrease in the fiber diameter has also proven to increase the flexural strengths.

2. The load-deflection curve is dependent on the fiber used, however, the fiber has little influence on the elastic modulus. The steel fiber reinforcement has a greater effect on the ultimate load (and associated deflection) than on the first crack load (and associated deflection), increasing the ductility substantially with increased fiber concentration [Swamy and Mangat, 1975].
3. Beams tend to fail by fiber pullout rather than fiber fracture [Batson et al., 1972].
4. Increased water-cement ratio decreases the flexural strengths. The first crack and ultimate strengths also depend on the smoothness and size of course aggregate; increased smoothness and decreased size increases flexural strength [Swamy and Stavrides, 1975]. The strength-age relationship of fiber reinforced concrete is similar to plain concrete [Hanna, 1977].

5. The method of casting; i.e., horizontal or vertical, and the type of vibration; i.e., internal or external, have a definite influence on the flexural strengths of steel fiber reinforced concrete. These factors tend to effect the fiber orientation and fiber distribution [Hannant and Spring, 1974 and Swamy and Stavrides, 1975].

In earlier investigations of the flexural strength of fiber reinforced concrete (mortar), improvements as high as 2-3 times the bending strength of plain concrete (mortar) were reported using small scale specimens [see, e.g., Romauldi and Mandel, 1964; Snyder and Lankard, 1972; and Shah and Naaman, 1976]. However, Swamy and Stavrides [1975] showed that the first crack and ultimate strengths were dependent on the dimensions of the test specimens. In view of this, the actual improvements in flexural strengths are less than initially expected and therefore the fibers must also be looked upon as a ductility admixture rather than merely a strength reinforcement [McCurrich and Adams, 1973; and Emery, 1977]. Halvorsen [1976] indicated that the ductility of a fiber concrete mix is very much dependent on the fiber material and the individual fiber failure mode.

For realistic conditions of fiber size and fiber concentration, the first crack strength may not increase significantly. Romauldi and Batson pointed out that the theoretical tensile strength of the composite may never be reached because the allowable bond between the fiber and matrix is exceeded long before significant fiber tensile strength is developed by the crack arrest mechanism [Batson et al., 1972]. In addition, Swamy and Mangat [1975] indicated that a minimum fiber volume; i.e. 0.35 for their study, was required to develop an ultimate strength distinct from the first crack strength. Therefore, it is apparent from these factors that the design of a suitable steel fiber reinforced concrete mix requires careful consideration; i.e. the critical fiber length, critical fiber concentration and workability must be established.

3.3.2 FATIGUE FLEXURAL STRENGTH

Highway engineers have long been aware of the need to understand the flexural fatigue behaviour of concrete in connection with pavements and reinforced concrete bridge decks [Norby, 1958]. Much of the previous experience with fiber reinforced concrete is related to field observations on repaired road sections in the United States [see, e.g. Gray and Rice, 1972; Arnold and Brown, 1973; Parker, 1974; and Lankard and Walker, 1975]. Field experience has shown that the steel fiber reinforced concrete appears to have great potential for pavement applications, but further laboratory investigations are required to develop sufficient design criteria.

Little laboratory data is available on the fatigue flexural strength of plain and steel fiber reinforced concrete. Because of the complexity of testing, no detailed guidelines for concrete fatigue tests have been developed, resulting in considerable variations in testing procedures with regard to beam sizes, loading conditions and fatigue failure criteria. Even under strict control of experimental procedures, fatigue test results show considerable scatter, and therefore a large number of test specimens at each stress level is required to adequately define the fatigue strength [see, e.g., Batson et al., 1973; Rilem, 1977; and Anderson and Henager, 1978]. In view of this, the behaviour of both plain and fiber reinforced concrete under cyclic loading, is summarized below assuming that some of the trends observed for plain concrete are applicable to steel fiber reinforced concrete.

Although previous investigations to determine the fatigue performance of unreinforced concrete have been limited, the more important trends are:

1. Age and curing method have a strong influence on the fatigue strength of concrete; longer cured specimens show better fatigue performance. Figures 3.2(a) and 3.2(b) indicate the influence of curing on the fatigue performance of concrete

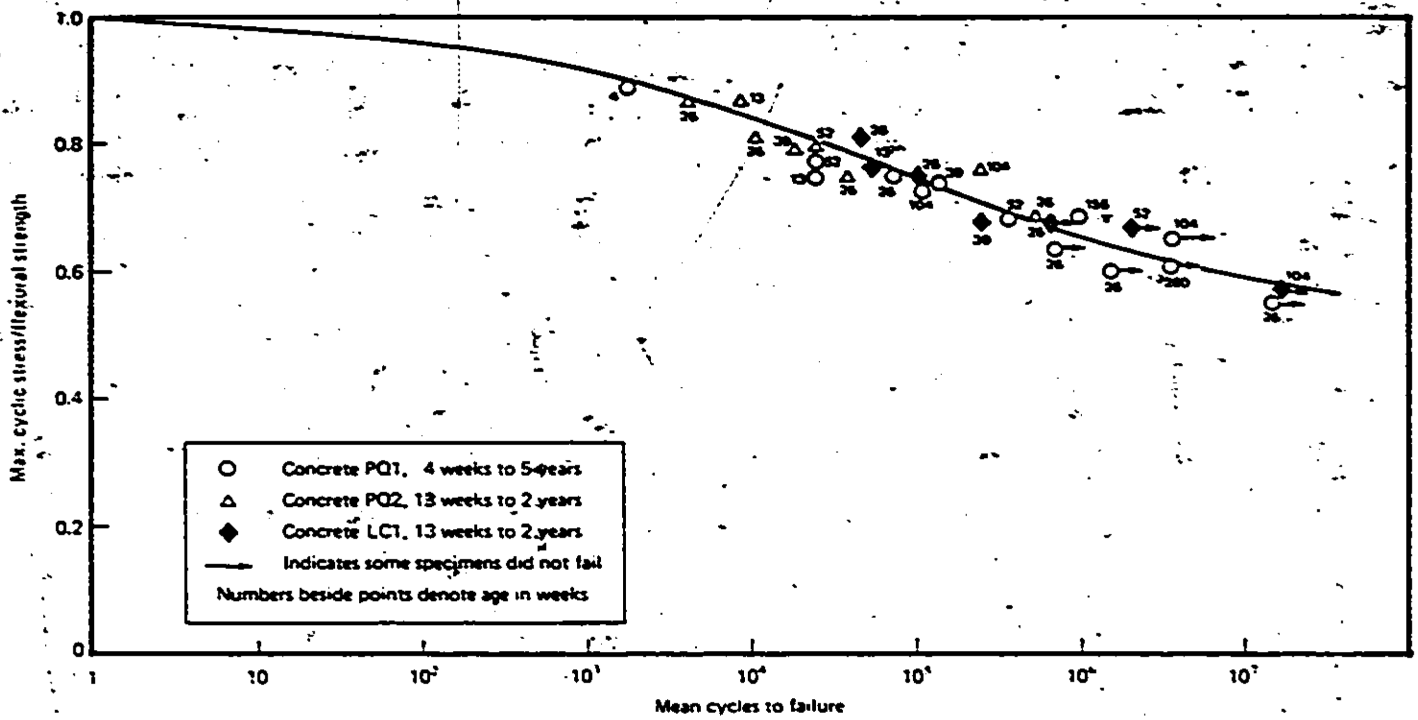


FIG. 3.2(a) FATIGUE PERFORMANCE RELATED TO FLEXURAL STRENGTH – AGES UP TO 5 YEARS (Galloway et al., 1979A)

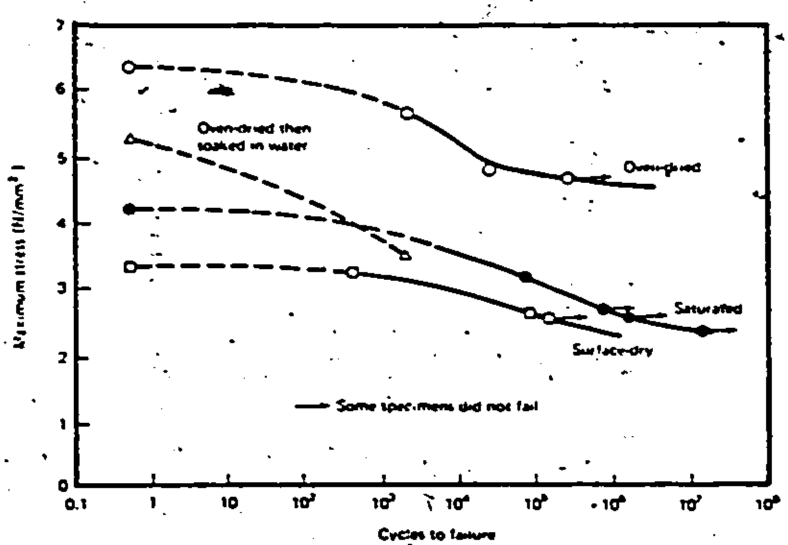


FIG. 3.2(b) MEAN FATIGUE CURVES FOR DIFFERENT MOISTURE STATES (Galloway et al., 1979B)

[see, e.g., Galloway et al., 1979A; and Galloway et al., 1979B].

2. Fatigue strengths decrease slightly with leaner mixes and higher water-cement ratios.

3. Rest periods, for specimens undergoing cyclic loading with no stress reversals, tend to increase the fatigue endurance of concrete. Most of the permenant deformation takes place in the early stages of loading. The deformation of the concrete stops when the repeated load stops, and for sufficiently long rest periods the strain recovery is almost complete.

4. The stress range for cyclic loading has a significant influence on the fatigue life of the concrete; an increased stress range decreases the fatigue life [Murdock and Kesler, 1958].

5. Kesler showed that the fatigue life is affected little for variations in the frequency of loading between 1 and 7.3 Hz, however, more recent research has indicated that loading rates as high 20 Hz have limited influence on the fatigue performance of concrete [see, e.g., Norby, 1958; and Raithby and Galloway, 1974].

6. Under repeated loading, the modulus of concrete changes in various ways depending on the intensity

of loading. Cyclic loading at low stress ratios improves the fatigue performance at higher stress ratios. The concrete also becomes stiffer [Norby, 1958].

7. Murdock and Kesler [1958] showed that S-N curves for concrete do not become asymptotic to a particular stress level, i.e. no fatigue limit, within 10 million cycles. McCall [1958] indicated that no fatigue limit exists up to 20 million cycles.

The trends just summarized are generally in accord with Minor's Hypothesis and PCA pavement design procedure. For thickness design purposes, the fatigue values shown in Table 3.3 are used in conjunction with Minor's Hypothesis that fatigue resistance not consumed by the repetition of one load is available for repetition of other loads where the total fatigue should not exceed 100 percent. The concept of comulative fatigue damage is considered to be a simple but conservative approach to pavement fatigue design. In addition, the fatigue values are also conservative with respect to flexural fatigue research on concrete thus incorporating a "safety factor" [see, e.g., Minor, 1945; and PCA, 1966].

From their review of steel fiber reinforced concrete fatigue behaviour, Anderson and Henager [1958] noted that investigators preferred the static first crack strength of fiber reinforced concrete as a basline for evaluating the

Table 3.3 Stress Ratios and Allowable Load-Repetitions for PCA Pavement Design (PCA, 1966)

Stress* ratio	Allowable repetition	Stress ratio	Allowable repetition
0.51**	400,000	0.69	2,500
0.52	300,000	0.70	2,000
0.53	240,000	0.71	1,500
0.54	180,000	0.72	1,100
0.55	130,000	0.73	850
0.56	100,000	0.74	650
0.57	75,000	0.75	490
0.58	57,000	0.76	360
0.59	42,000	0.77	270
0.60	32,000	0.[illegible]	210
0.61	24,000	0.79	160
0.62	18,000	0.80	120
0.63	14,000	0.81	90
0.64	11,000	0.82	70
0.65	8,000	0.83	50
0.66	6,000	0.84	40
0.67	4,500	0.85	30
0.68	3,500		

*Load stress divided by modulus of rupture.
**Unlimited repetitions for stress ratios of 0.50 or less.

influence of fiber additions on fatigue performance. While for plain concrete a definite first crack strength exists, the location of the proportional limit used to define the first crack strength of fiber reinforced concrete is very subjective. Due to inconsistent methods of determining the baseline for fatigue tests, variations in results from various investigations is expected.

Romauldi presented data for steel fiber reinforced concrete indicating fatigue strengths in excess of 90 per cent of first crack strength at 2 million cycles for non-reveral of loading using 2 to 3 percent steel fibers by volume. Batson et al. [1972] pointed out that size influences could account for high fatigue strengths. They showed that the fatigue strengths were closer to 74 and 83 percent of first crack strength for complete reversal and non-reversal of loading, respectively, using 3 percent fibers by volume. While experimental results are not conclusive, it is generally considered that the fatigue strength of steel fiber reinforced concrete increases with increased fiber content. Similarly, post fatigue static strength has proven to be greater than prefatigue static strength by as much as 10 to 30 percent [Batson et al., 1973].

3.4 COMPRESSION, SPLIT CYLINDER AND IMPACT STRENGTHS

3.4.1 COMPRESSION STRENGTH

Under uniaxial compression, concrete experiences internal microcracking parallel to the direction of the applied load after a certain stress level. This crack development within concrete is gradual, and in the initial stages so slow that material discontinuities are not noticable. Therefore, with the inclusion of fibers, no drastic change in the elastic region of the stress-strain curve is expected as shown in Figure 3.3 [Rilem, 1977].

Much of the compression strength testing of steel fiber reinforced concrete has been inconclusive with considerable variations from no strength increase to as much as 100 percent increase, depending on the manner of casting and compaction. However, extensive studies completed by Williamson [1973] indicated that the addition of steel fibers increased the compressive strength of concrete by 23 percent, but did not increase the compressive strength of mortar and may in fact decrease the strength. He also showed that the modulus and Poisson's ratio for concrete were not changed significantly by the addition of steel fibers.

Hughes and Fattuhi [1976] indicated that the

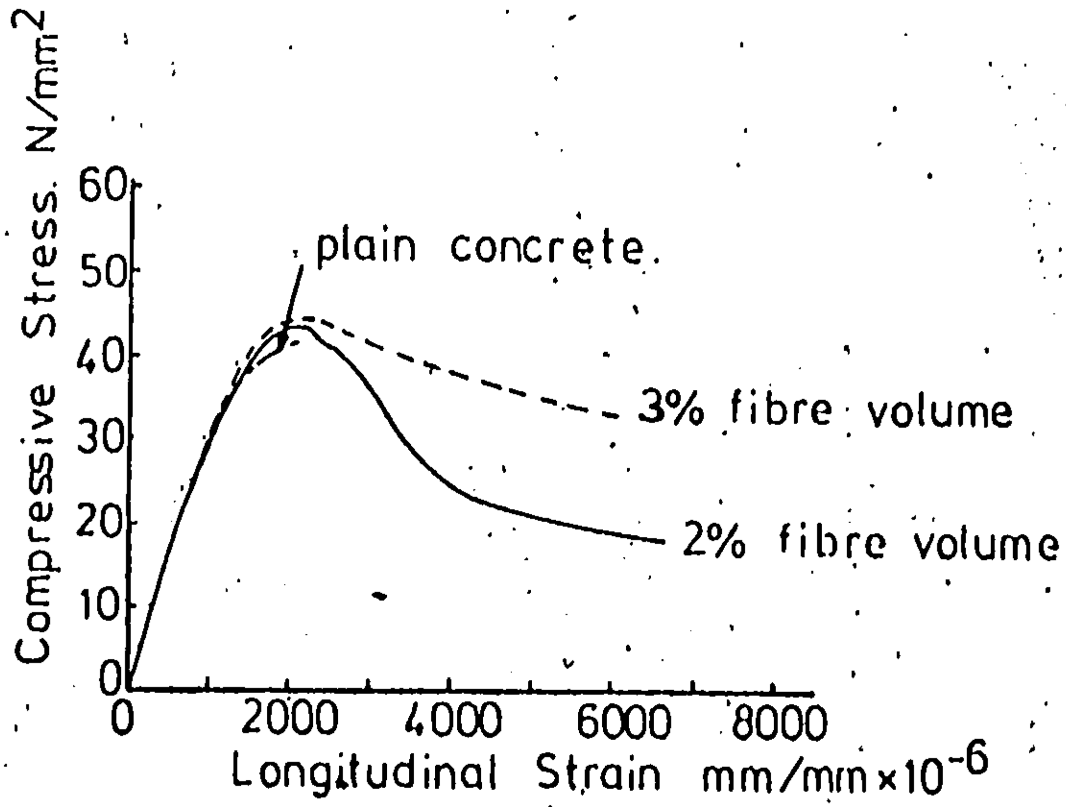


Fig.3.3— Stress-strain deformation in compression of steel fibre concrete. (RILEM,1977)

increased strength and ductility depended on:

1. length and volume fraction of steel fibers in the mix;
2. type and strength of plain matrix; and
3. orientation of steel fibers relative to the direction of the load, which is influenced by both specimen shape and compaction procedure.

They reported a maximum compressive strength increase of 7 percent at a fiber concentration of 1.5 percent by volume using 25 mm long Duoform fibers.

3.4.2 SPLIT CYLINDER STRENGTH

The split cylinder test (ASTM C496) has received considerable attention in the past for steel fiber reinforced concrete in pavement construction. Tests on fiber reinforced mortar have shown strength increases as high as 2.5 times over plain mortar at a fiber concentration of 3 percent by volume. Even though the strength improvements appear significant, the mode of failure in such tests is not representative of actual failure conditions.

Johnston [1973] stated that the relationship between tensile stress and applied load is meaningless once the specimen cracks. While for plain concrete the ultimate and first crack strengths are identical in tension, the ultimate and first crack strengths for steel fiber reinforced concrete can be different as indicated in Section 2.2. According to Johnston, this makes the test most inappropriate for steel fiber reinforced concrete due to the difficulty in accurately recording the first crack strength.

3.4.3 IMPACT RESISTANCE STRENGTH

Different types of impact and impulsive load tests have been applied to steel fiber reinforced concrete in which the falling mass method has become the most popular [Rilem, 1977]. Studies indicate that impact strength increases with the inclusion of fibers, but no relationship between impact strength and aspect ratio or fiber concentration has been found.

Due to the variety of tests, differences in results are expected. Johnston [1973] showed that fibers improved impact resistance measured by the Charpy test, however, there was much scatter in the data. This was attributed to the non-uniformity of fiber distribution and the small size of test specimens. Further studies completed by Bailey et al. [1975] on the impact resistance of stair treads indicated that the inclusion of fibers did not delay the onset of first crack, but did reduce the extent of subsequent crack behaviour by minimizing crack widths. Similar observations were reported by Takazuka et al. [1977] and Rilem [1977].

3.5 EXPERIMENTAL PROCEDURE AND TESTING

For practical and economic reasons, low fiber concentrations have been used in fiber reinforced concrete products. With lower fiber contents, the risk of random failure increases, and for this reason, the influence of low fiber concentrations on the static and fatigue flexural strengths was investigated in the current study. Other important parameters considered include fiber length and first crack stress ratio (ratio of the cyclic stress to the static first crack strength). S-N curves (stress level-cycles to failure) were developed from the fatigue testing results to compare the influences of fiber length and concentration on the fatigue performance of concrete. Compression, split cylinder and impact resistance strengths were also determined. Details of the test results for flexural, compression, split cylinder and impact resistance tests are given in Appendices 4, 5, 6 and 7, respectively.

3.5.1 MIXES AND MATERIALS

The recommended Stelco fiber mix design for 1/2" aggregate and related control mix were used throughout the strength study. The two mixes are given in Table 3.4. For the flexural specimens, only fiber concentration and fiber length were varied. A more detailed description of materials and mixes is given in the next chapter on workability.

TABLE 3.4

STELCO 13mm (½") AGGREGATE, STEEL FIBER AND CONTROL MIXES

Mix Constituent	Steel Fiber		Control	
	kg/m^3	(lb/yd^3)	kg/m^3	(lb/yd^3)
Coarse Aggregate	936.94	(1580)	965.40	(1628)
Fine Aggregate	907.29	(1530)	934.63	(1576.1)
Cement	382.49	(645)	394.35	(665)
Water	177.90	(300)	181.40	(305.9)
Fibers	71.16	(120)	-	-
Admixture	Porzite L-70, 400 ml/100 kg cement			

3.5.2 PREPARATION AND TESTING

Details of the batching procedure and workability measurements (slump, compaction factor and vebe tests) are also given in the next chapter. The workability tests helped in detecting major variations between batches of similar mix design. Following testing of the flow properties, the specimens were cast and allowed to cure 24 hours to gain sufficient strength before being stripped and removed to cure in the concrete moist room. A full description of the casting and curing procedures is given in Table 3.5.

The flexural tests were performed using the MTS closed-loop servo controlled hydraulic ram system shown in Figure 3.4. Flexural testing was completed in two stages:

1. Failing each double break beam statically under third point loading in accordance with ASTM C78 using a 457.2 mm (18") span. A displacement ramp mode was used to control the rate of loading at .000025 cm/sec.
2. Failing the remaining beam section under cyclic loading with the same support conditions used for the static tests. A load control mode was required to maintain uniform cyclic loading using a compressive haversine wave at a frequency of 10Hz. The specimens (cured for 80 to 90 days) were failed in fatigue through a cyclic loading range varying from zero to maximum load at various load levels, between 75 and 90 percent of the first crack strength, as indicated by the schematic in Figure 3.6. The remaining specimens were failed in fatigue at 70 percent of the first crack strength at 180 days.

TABLE 3.5

DETAILS FOR STRENGTH TESTS

Specimen	Number	Size (cm)	Curing Time (days)	Remarks
Flexural	55	15.25x15.25x105	80 to 90 and 180	Cast horizontally, vibrated internally for maximum compaction
Compression	47	15.25x30.5	7 and 28	Cast vertically, vibrated internally for maximum compaction
Split Cylinder	29	15.25x30.5	7 and 28	Cast vertically, vibrated internally for maximum compaction
Impact	8	15.25x5.08	30 and 60	Cast vertically, vibrated internally for maximum compaction

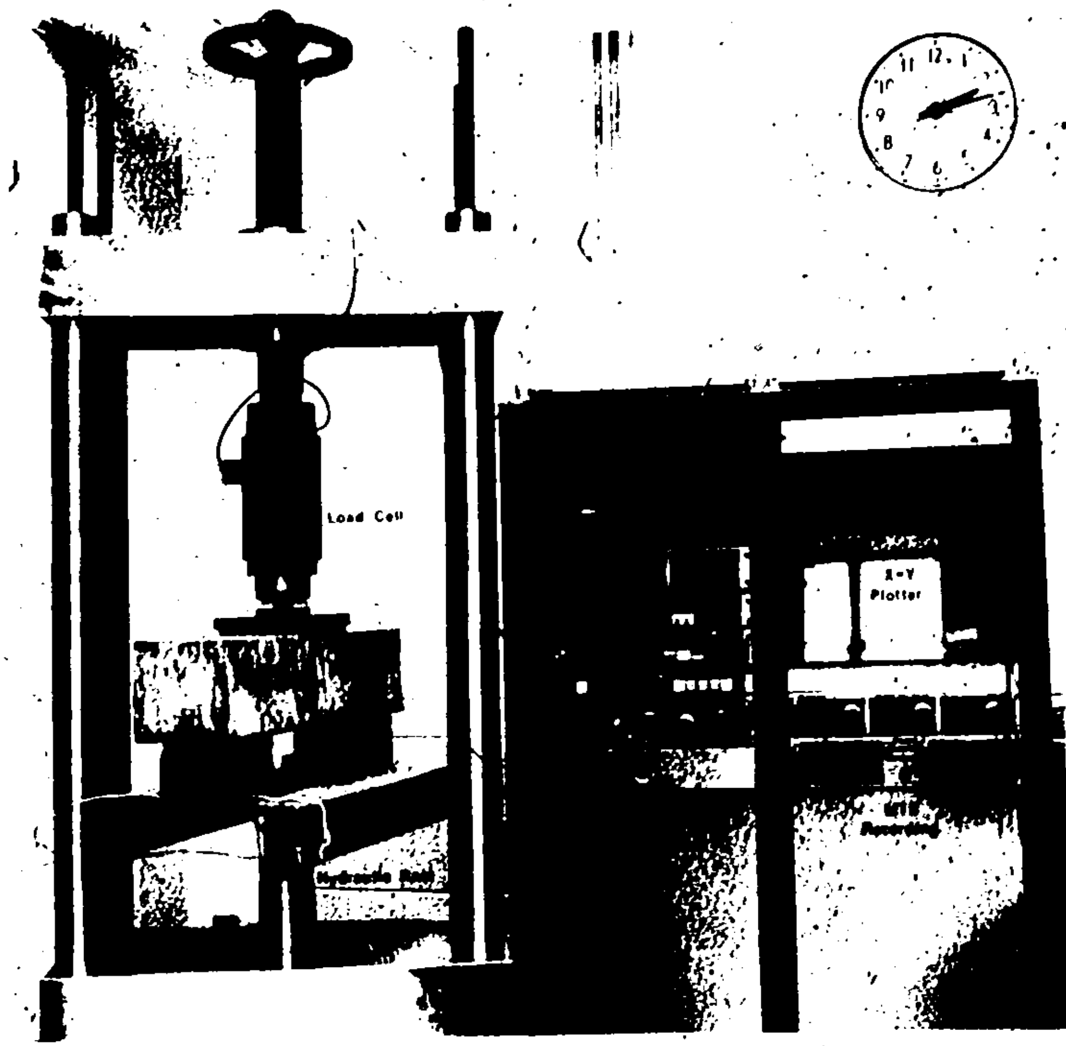


FIGURE 3.4 MTS System Used for Flexural Tests

FIGURE 3.5 Specimen Failed in Flexural Fatigue

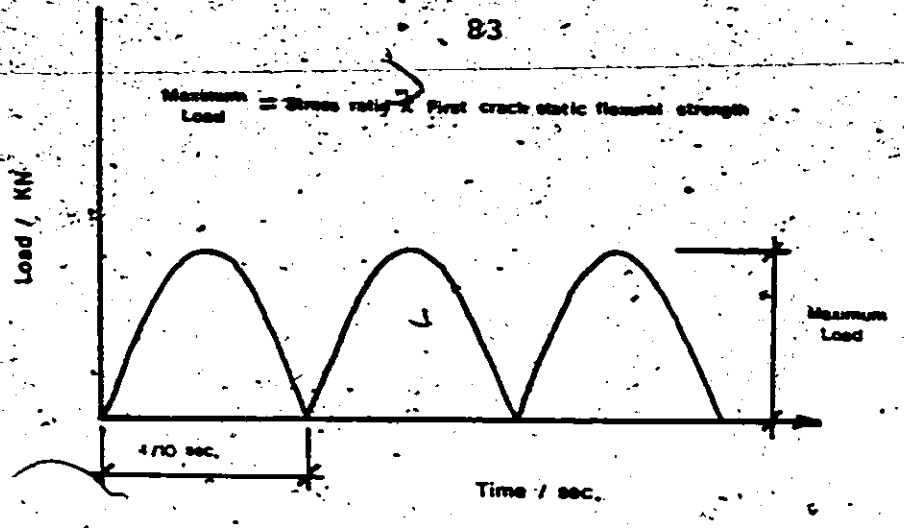


FIGURE 3.6 A Schematic of the Fatigue Cyclic Loading

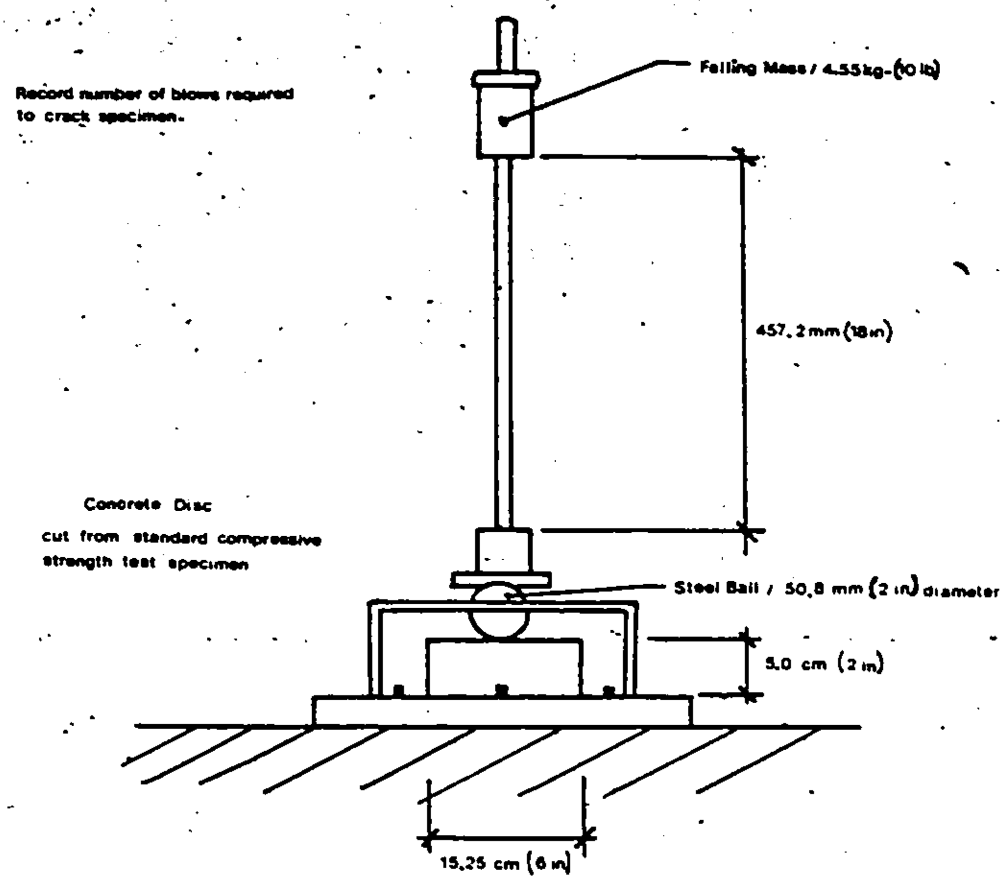


FIGURE 3.7 Impact Resistance Test Setup (proposed by ACI Committee 544)

Compression and split cylinder tests were completed in accordance with ASTM C39 and ASTM C496 using a standard large compression testing machine. The impact test proposed by ACI Committee 544 was used to measure the impact resistance of the plain and fiber concrete specimens as shown in Figure 3.7.

3.6 RESULTS

3.6.1 STATIC FLEXURAL TEST

Although the test results indicated that the inclusion of fibers increased the static flexural strength of concrete, the increase was small; approximately 3 percent as the 1.27 cm long fiber volume increased from 0 to 1.35 percent (0 to 4.32 percent by weight). The influence of fiber concentration on flexural strength is shown in Figure 3.8. It is apparent from Figure 3.8 that much scatter in test results was involved and the increase in average strength was not monotonic with respect to increase in fiber content; strength decreased for fiber volumes of 0.45 and 0.9 percent before increasing. Unlike the 1.27 cm long fiber, the 2.54 cm fiber showed superior reinforcing quality at 0.9 percent fiber concentration as shown in Figure 3.9. The average static flexural strength for the longer fiber mix was 16 per cent greater than the strength of the plain concrete.

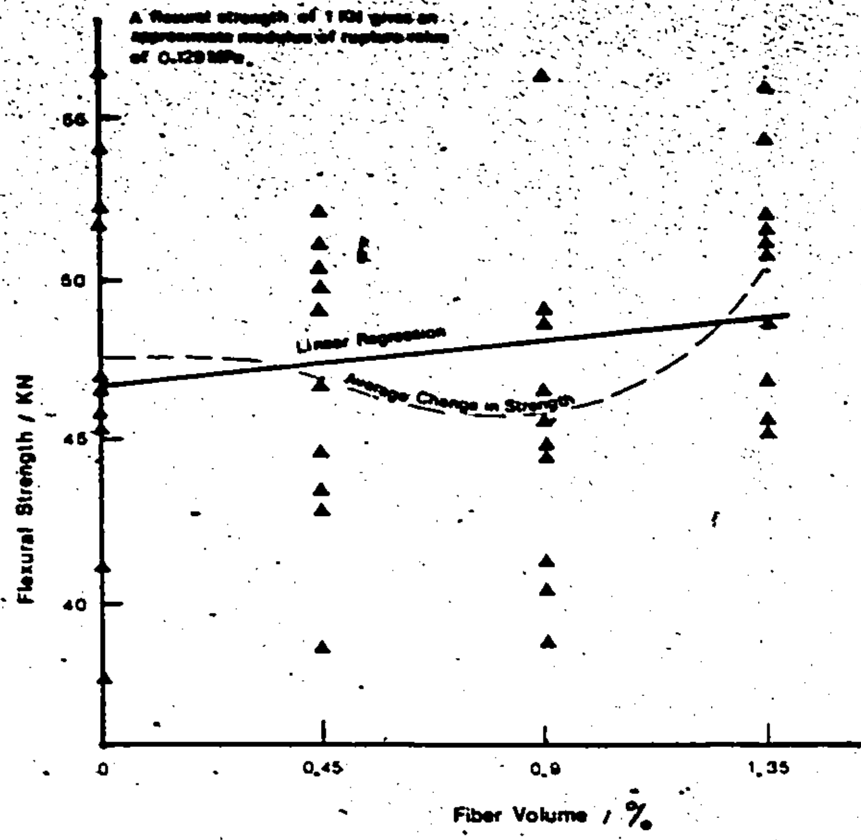


FIGURE 3.8 Influence of Fiber Volume on Static Flexural Strength Using 1.27 cm (0.5") Long Fiber (90 day curing in moist room)

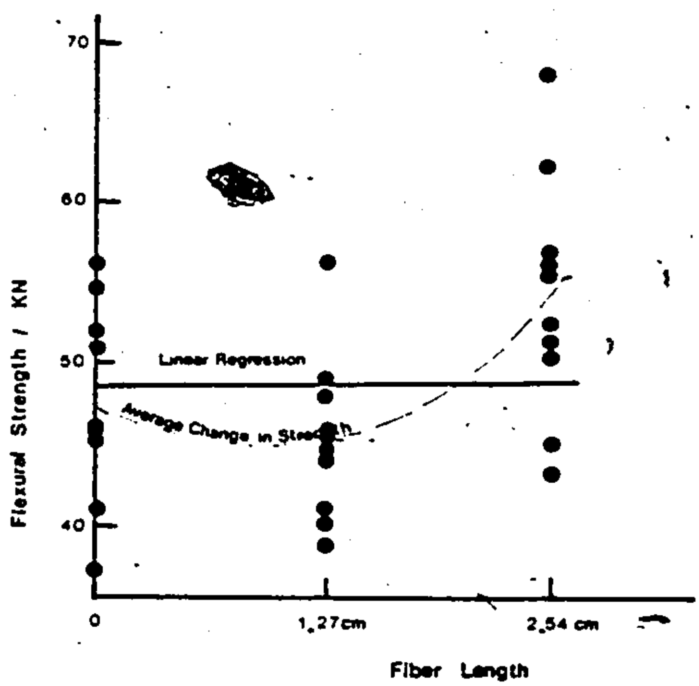


FIGURE 3.9 Influence of Fiber Length on Static Flexural Strength at 0.9 Percent Fiber Volume (90 day curing in moist room)

The Stelco fibers did not appear to significantly increase the ductility of the concrete, i.e. no post cracking strength developed. This is clearly shown in Figures 3.10 and 3.11. As stated previously, Swamy and Mangat [1975] showed that a minimum fiber volume was required to increase the ultimate flexural strength of the composite over that of the unreinforced matrix; below this volume, the fibers increased the onset of cracking but failure occurs simultaneously on cracking. In the current study, no distinct ultimate strength was observed at 1.35 and 0.9 percent fiber content using 1.27 and 2.54 cm fibers, respectively. In spite of the lack in strength improvement, the fibers did impart ductility to the mode of failure. A distinct cracking pattern was observed during failure as fiber volume and length were increased, i.e. the rate of visible crack development decreased with increased fiber content. For plain concrete specimens the failure was immediate, i.e. brittle.

The inclusion of fibers had no influence on the flexural modulus of the concrete as shown clearly in Figure 3.10. However, the modulus did increase after the specimen was subjected to cyclic loading at a low stress level. This increase in modulus is shown in Figure 3.11. The initial non-linearity of the load-deflection curves (actually load versus ram movement) is attributed to the local failures at the supports during loading. For specimens undergoing

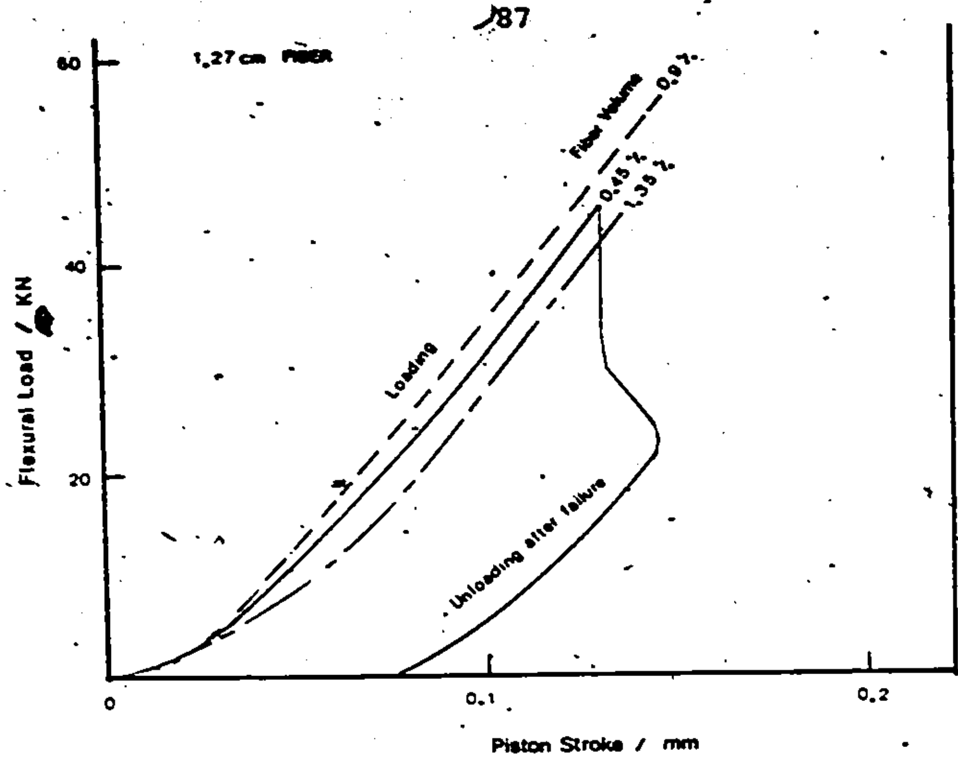


FIGURE 3.10 Load-deflection Curves for Third Point Static Flexure Test

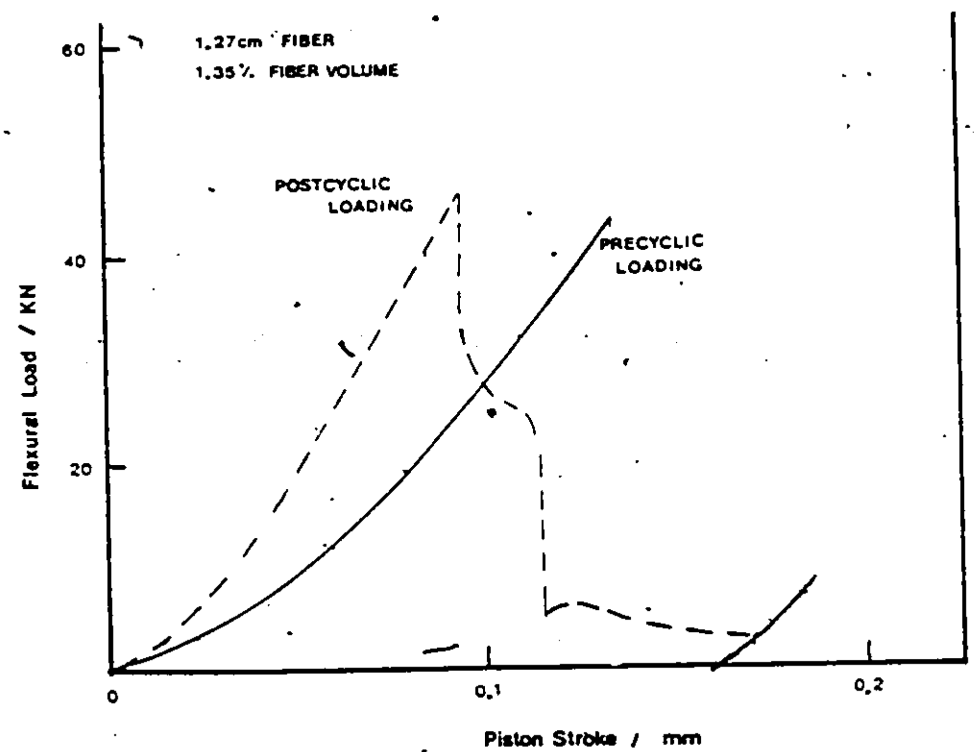


FIGURE 3.11 Influence of Cyclic Loading on Concrete's Stiffness (Unfortunately, actual specimen deflection not recorded)

post-cyclic loading, much of the localized support failures occurred during the initial loading, therefore, less non-linear behaviour in the load-deflection curve is expect as shown in Figure 3.11. Cyclic loading at lower stress levels did not appear to increase the specimen's flexural strength.

3.6.2 FATIGUE FLEXURAL STRENGTH

All fatigue specimens failed quickly without warning. Due to the high frequency of loading, i.e., 10 Hz, the deformation of the specimens could not be satisfactorily recorded during testing. Therefore, permanent deformations, progressive collapse or increased microcracking (internal changes) could not be studied in detail. However, through controlled failing of cracked specimens by fatigue at a stress level of 80 percent of first crack strength, fatigue endurances as high as 9 cycles at 1.35 percent fiber volume were attained. This would suggest that for steel fiber concrete, more extensive internal cracking is required before failure than for plain concrete. A typical specimen failed in fatigue is shown in Figure 3.5.

The influence of stress ratio, fiber length and fiber concentration on the number of cycles to failure is shown in Figures 3.12 to 3.17. Except for Figure 3.17 which compares the various regression curves, the figures show all data recorded during fatigue testing including the data rejected for regression analysis. Since the fatigue stress

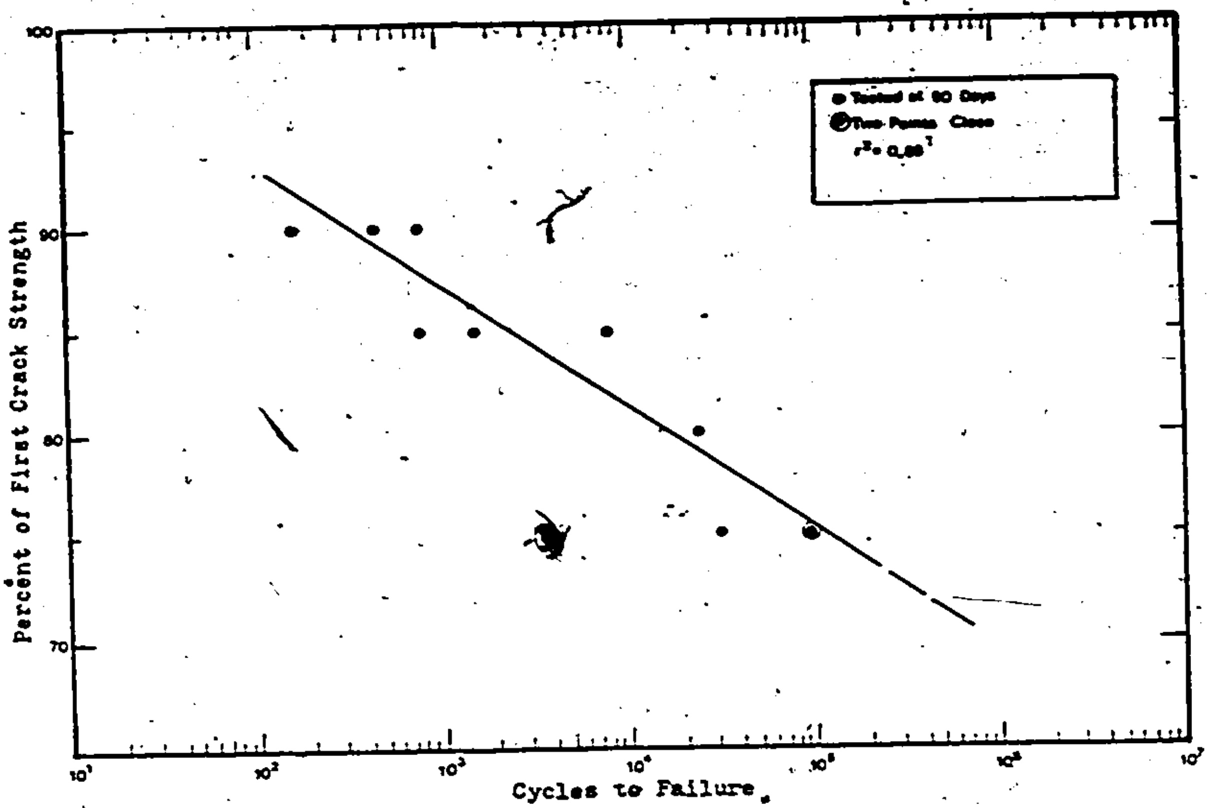


FIGURE 3.12 Fatigue Strength for Plain Concrete

1. Coefficient of determination

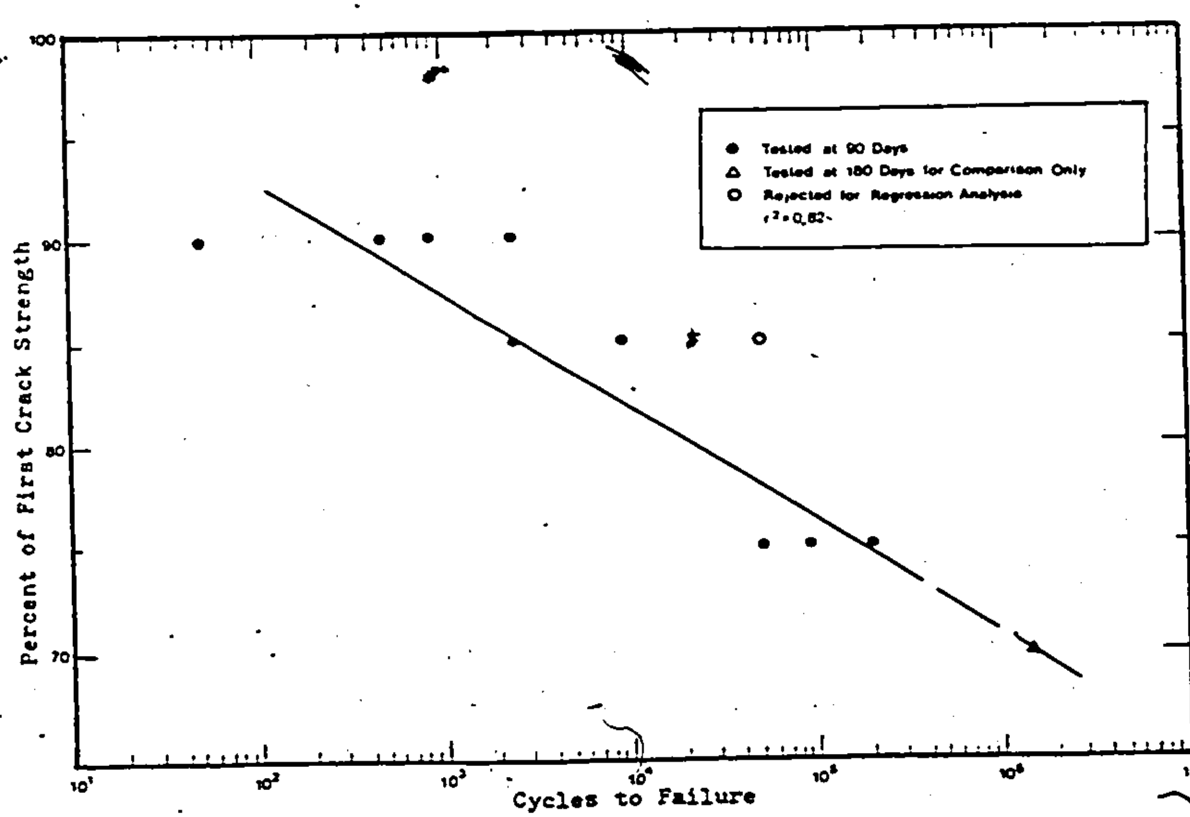


FIGURE 3.13 Fatigue Strength for 0.25 x 0.51 x 12.7 mm Fiber at 0.45 Percent Fiber Volume

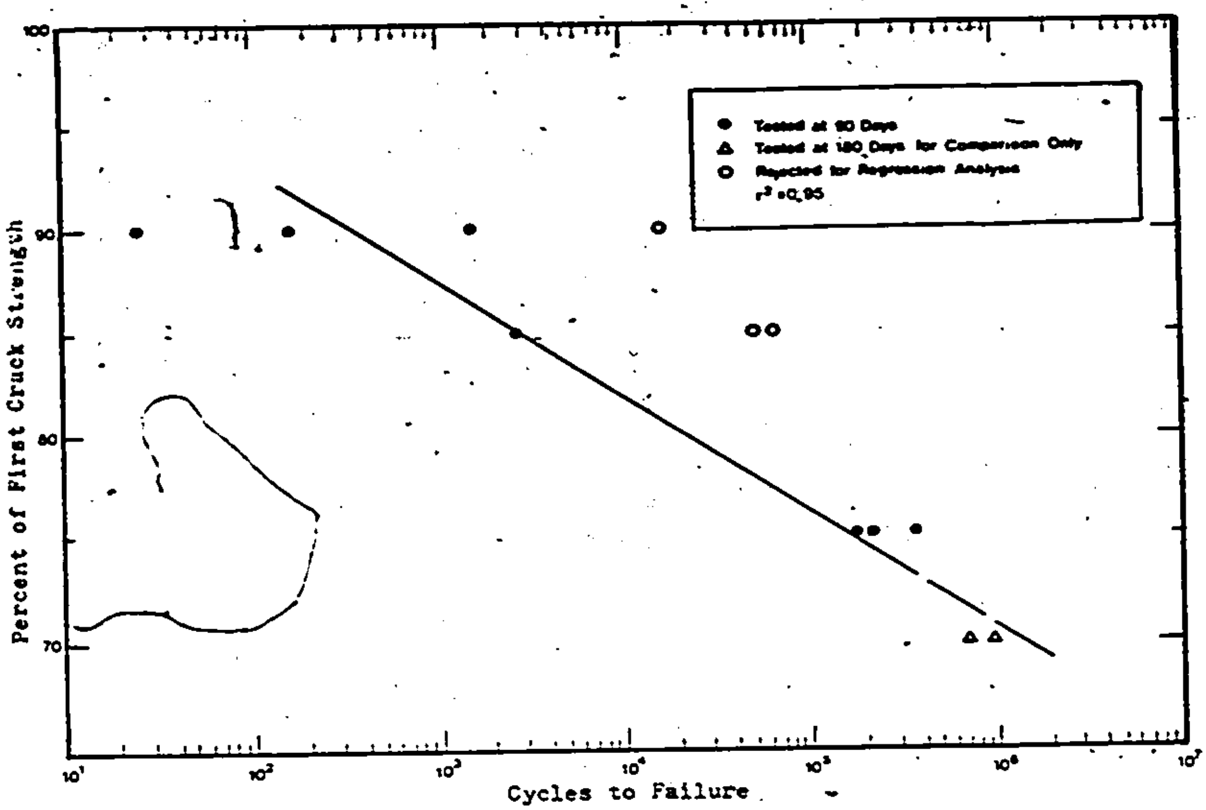


FIGURE 3.14 Fatigue Strength for 0.25 x 0.51 x 12.7 mm Fiber at 0.9 Percent Fiber Volume

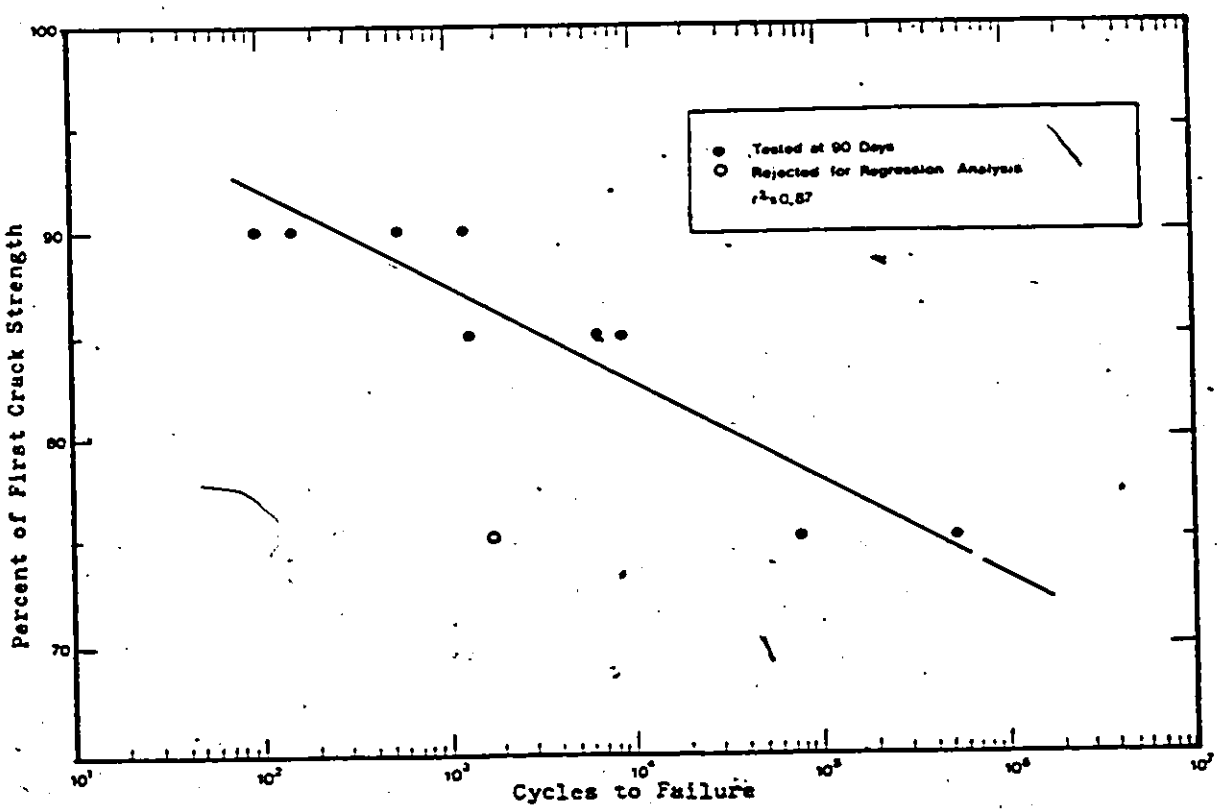


FIGURE 3.15 Fatigue Strength for 0.25 x 0.51 x 12.7 mm Fiber at 1.35 Percent Fiber Volume

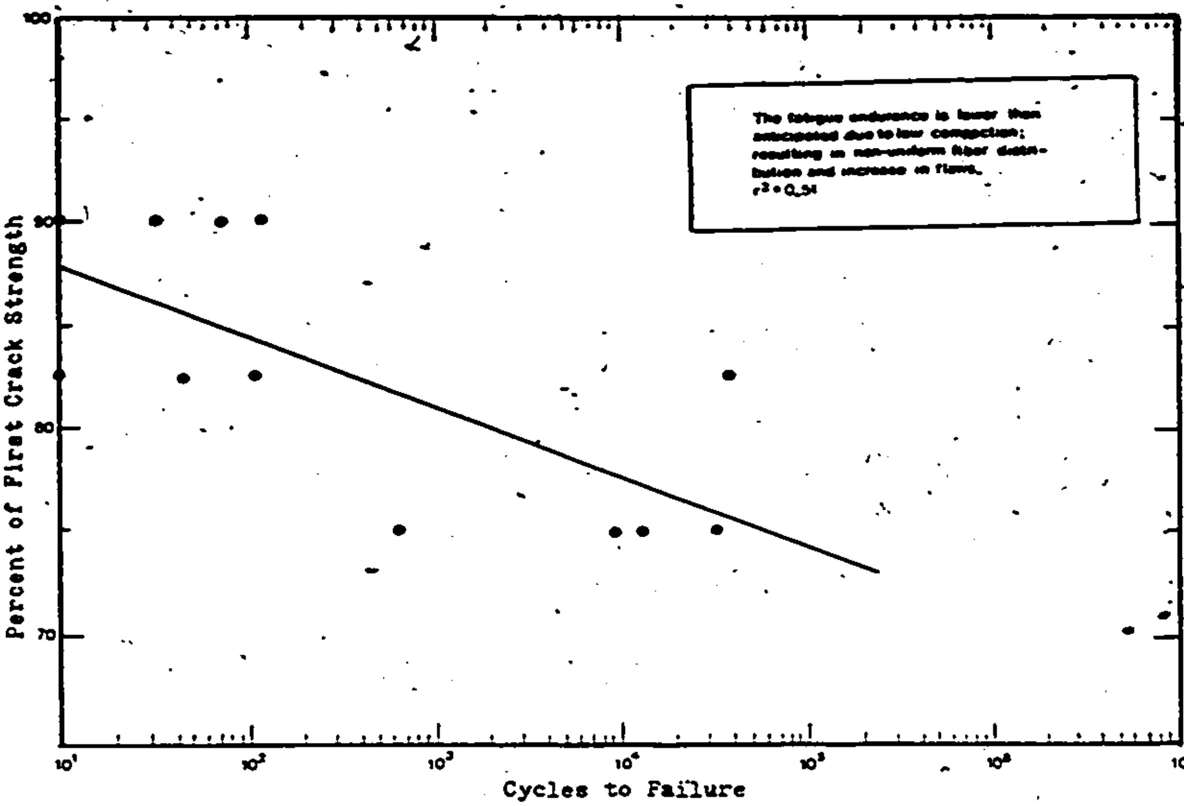


FIGURE 3.16 Fatigue Strength for 0.25 x 0.51 x 25.4 mm Fiber at 0.9 Percent Fiber Volume

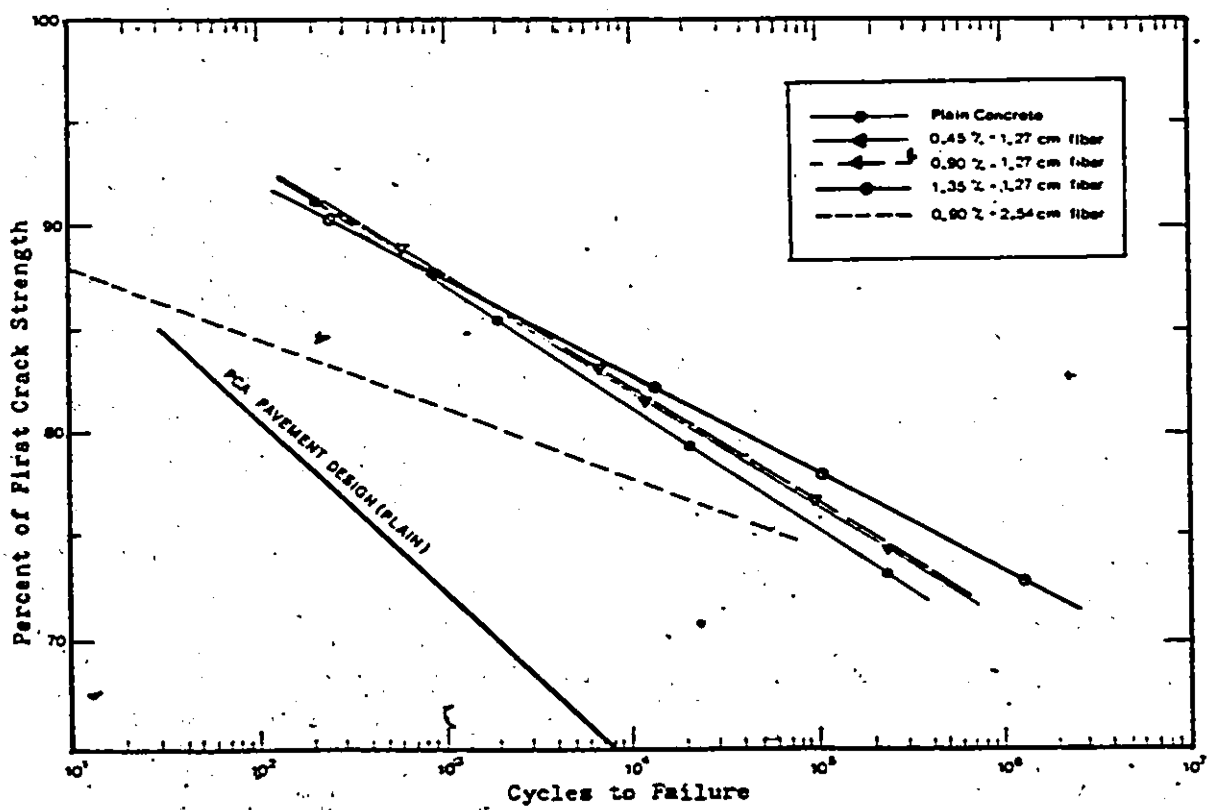


FIGURE 3.17 Comparison of Various Fatigue Curves

ratio is based on the first crack strength of the same specimen undergoing fatigue (taken at a different cross-section) scatter in test results is believed to be due to non-uniform fiber distribution along the specimen's length, specimen seating at supports and dimensional changes along the specimen's length. In the most extreme cases, the variations in specimen width was as much as ±0.25 cm at failure cross-sections due to bulging of specimen molds.

For regression analysis of specimens containing 1.27 cm fibers, the points rejected were based on the overall fatigue performance of the specimens containing extreme fiber concentrations (i.e. 0 and 1.35 percent fiber volume). Theoretically the fatigue endurance should increase with increased fiber volume, therefore for intermediate fiber volumes the regression curves should lie between the two extremes as shown in Figure 3.18. To reject data points it was assumed that all data should lie between one standard deviation of the extreme regression relationships which is also shown in Figure 3.18. In most cases the rejection of data points is conservative from a design viewpoint.

Although the number of tests completed was relatively small, the results indicated that the use of short steel fibers at low fiber volumes does not increase the fatigue strength significantly at the stress ratios tested (except at the lowest stress level of 75 percent which is typical

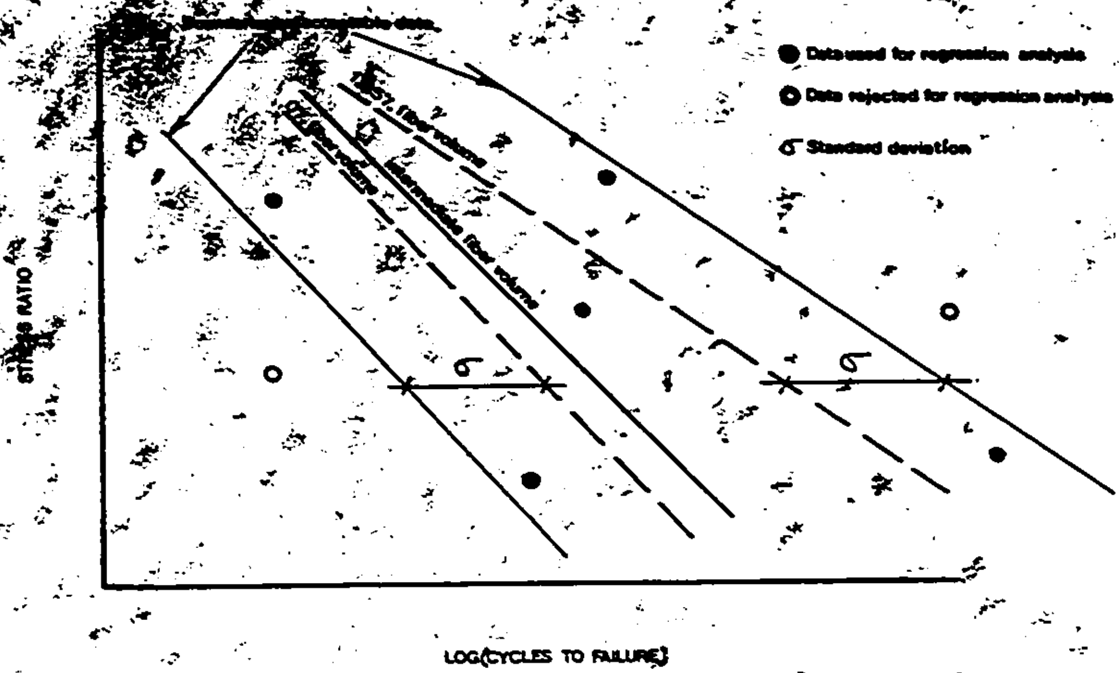


FIGURE 3.18 S-N Curve Illustrating Rejection of Data Points (schematic)

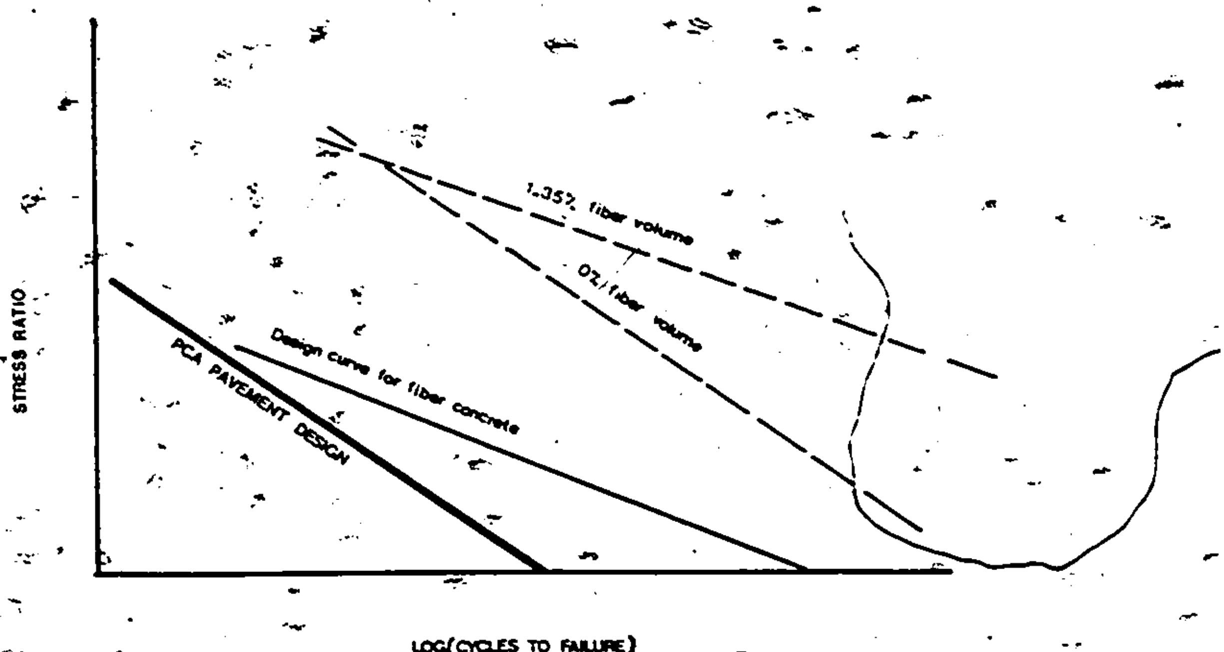


FIGURE 3.19 Example Design Curve for Pavement Design (schematic)

of the stress level for actual pavements) and can in fact result in strength loss at higher stress ratios as shown in Figure 3.17. The poor performance of the specimens incorporating longer fibers is attributed to insufficient concrete compaction due to lack of workability. This would suggest that the results for specimens incorporating longer fiber do not adequately reflect the potential reinforcing capability of the fiber. However, the results do indicate the importance of workability on strength properties, and similar workability related problems are observed in the field, particularly during pavement construction.

It is apparent that a significant safety factor exists between PCA pavement design fatigue values and the plain concrete fatigue values obtained in this study as shown in Figure 3.17. Although the mixes incorporating the 1.27 cm long fiber do not show significant improvement with increases in fiber content, the difference in slope would suggest that at lower stress ratios the improvement would become more significant. Considering that for pavement design the majority of load repetitions is in the lower stress ratio range, reduction in pavement thickness should be possible when using fiber reinforcement as indicated in Figure 3.19.

3.6.3 COMPRESSION TESTS

The experimental results indicate that the water-cement ratio has a greater influence on the compressive strength of concrete than fiber volume or fiber length. The effect of these variables is shown in Figures 3.20(a), (b) and (c). The longer fiber (2.54 cm) increased the compressive strength by 17 percent as the fiber volume was varied for 0 to 1.35 percent fibers. However, the short fibers did not influence the compressive strength significantly.

3.6.4 SPLIT CYCLINDER TESTS

Even though the split cylinder test is not regarded as suitable for fiber concrete, the test did show significant improvements in tensile strength in this failure mode with increased fiber content. The test, an indirect means for determining the tensile strength of concrete, indicated a lower tensile failure stress for the composite than did the modulus of rupture taken from flexural tests, i.e. 40 to 50 percent lower as indicated in Table 3.6. However, due to the actual stress field shown in Figure 3.21, the split cylinder strength is still actually higher than the strength obtained from a direct test.

The influence of fiber volume and fiber length on the first crack strength is shown in Figure 3.22. The shorter

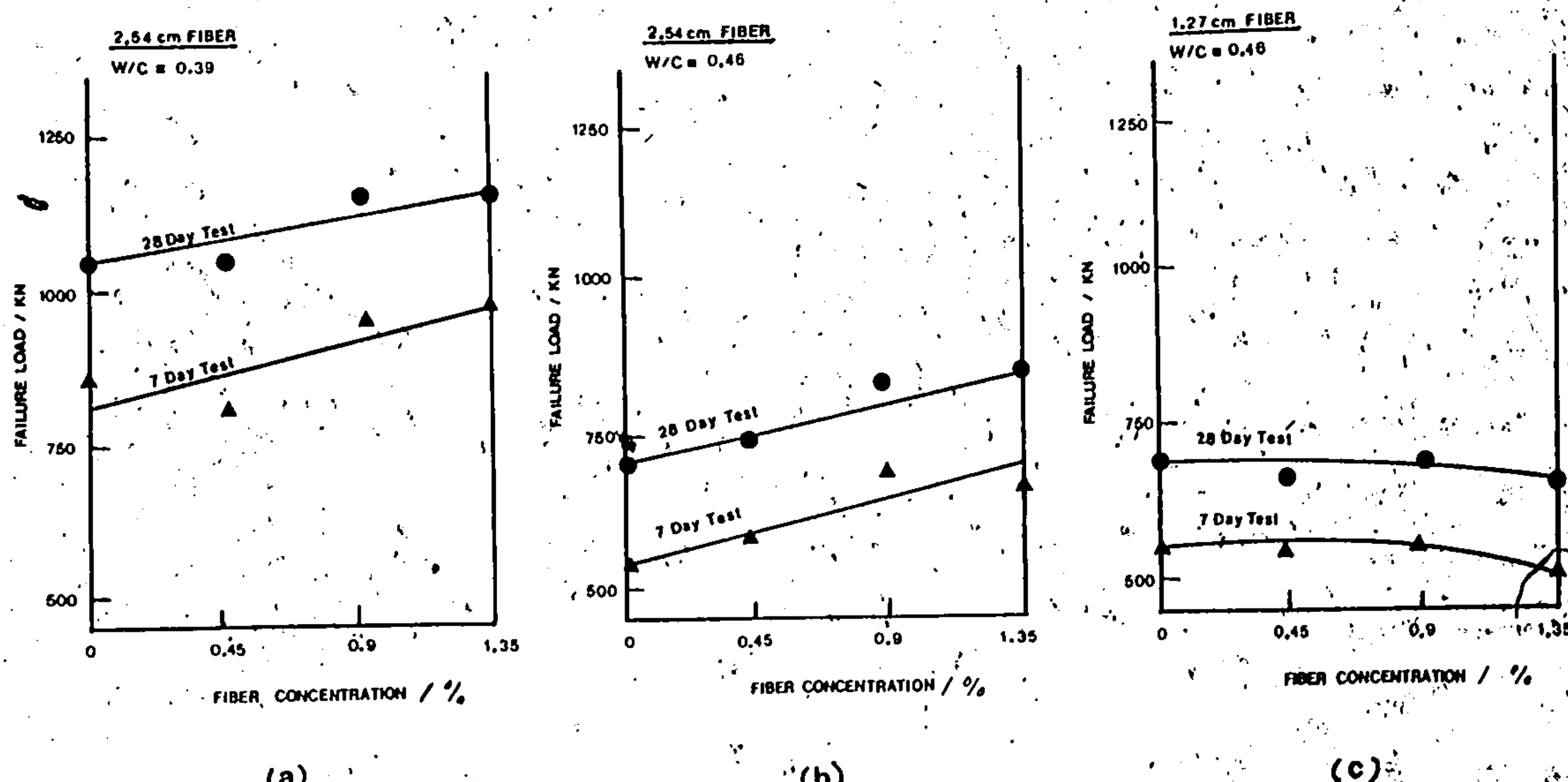


FIGURE 3.20 Influence of Fiber Length and Concentration on the Compressive Strength of Concrete

TABLE 3.6

COMPARISON OF AVERAGE FLEXURAL MODULUS OF RUPTURE AND AVERAGE SPLIT CYLINDER FAILURE STRESS

Concrete	Number of Specimens	Modulus of Rupture		Number of Specimens	Split Cylinder Stress	
(——)	(-)	(MN/m^2)	(Psi)	(-)	(MN/m^2)	(Psi)
Plain	10	6.15	892.50	6	3.27	474.55
1.27 cm - 0.45%	10	6.02	873.33	3	4.28	620.44
1.27 cm - 0.90%	10	5.87	850.83	5	4.69	680.83
1.27 cm - 1.35%	10	6.46	937.50	2	4.45	645.46
2.54 cm - 0.45%	-	-	-	4	3.83	555.98
2.54 cm - 0.90%	10	7.00	1015.00	7	4.25	616.37
2.54 cm - 1.35%	-	-	-	1	4.82	698.51

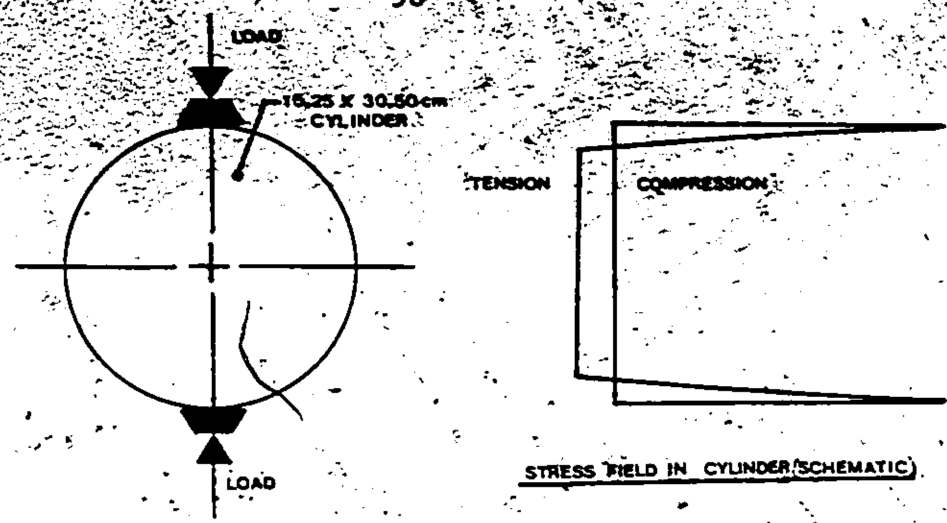


FIGURE 3.21 Load Application and Resulting Stress Field for Split Cylinder Test (Nevile, 1973)

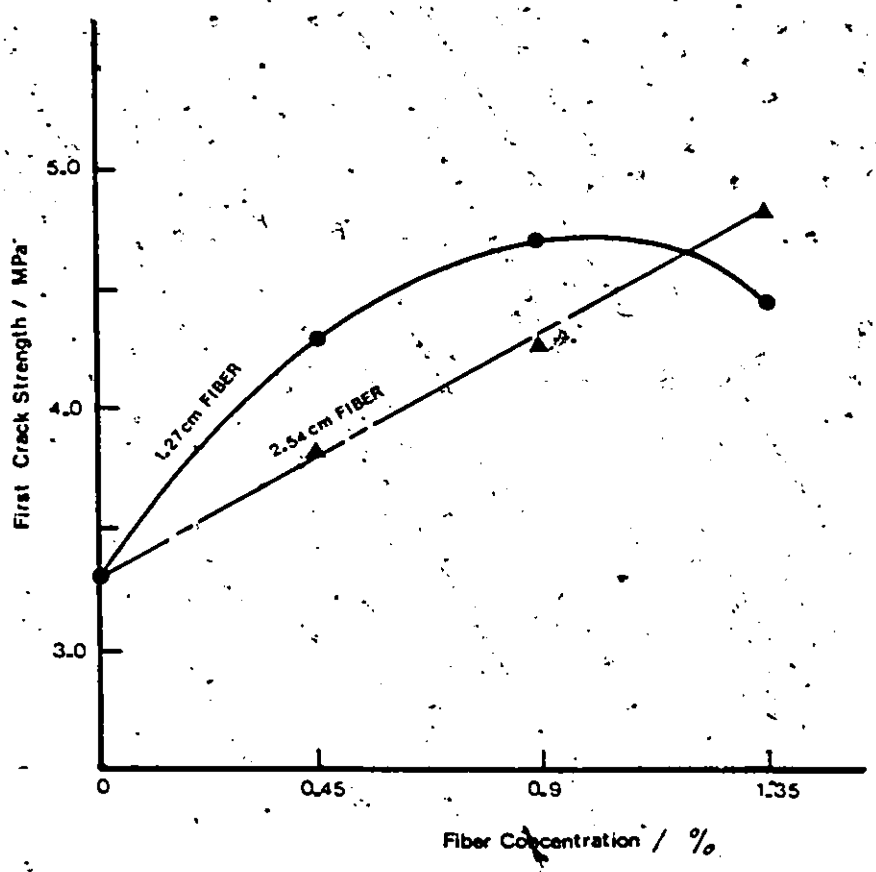


FIGURE 3.22 Influence of Fiber Concentration on Split Cylinder Strength

fibers had a greater influence on reinforcing the concrete at the lower fiber concentrations than longer fibers. The superior reinforcing ability is attributed to the better workability of the mix containing the shorter fibers. However, with increased fiber content the workability of the short fiber concrete mix is reduced accounting for a smaller influence of the short fibers than the longer fibers at a fiber volume of 1.35 percent. With increased fiber content and length, the post-cracking load increased. For plain concrete, the load decreased immediately after the first crack appeared. In addition, the specimens incorporating fiber did not exhibit the clean failure plane normally found with plain concrete specimens.

3.6.5 IMPACT RESISTANCE TESTS

The ultimate impact strength increased with increased fiber content, increased curing time, and decreased water-cement ratio as shown in Figure 3.23. Unlike the ultimate strength, the onset of cracking, i.e. first crack strength was not significantly influenced by increased fiber content which is consistent with the results given in Section 3.4.3. Both first crack and ultimate strengths were greatly influenced by water-cement ratio and curing length.

The impact test proposed by the ACI can be used to compare impact strengths of different concrete mixes, however,

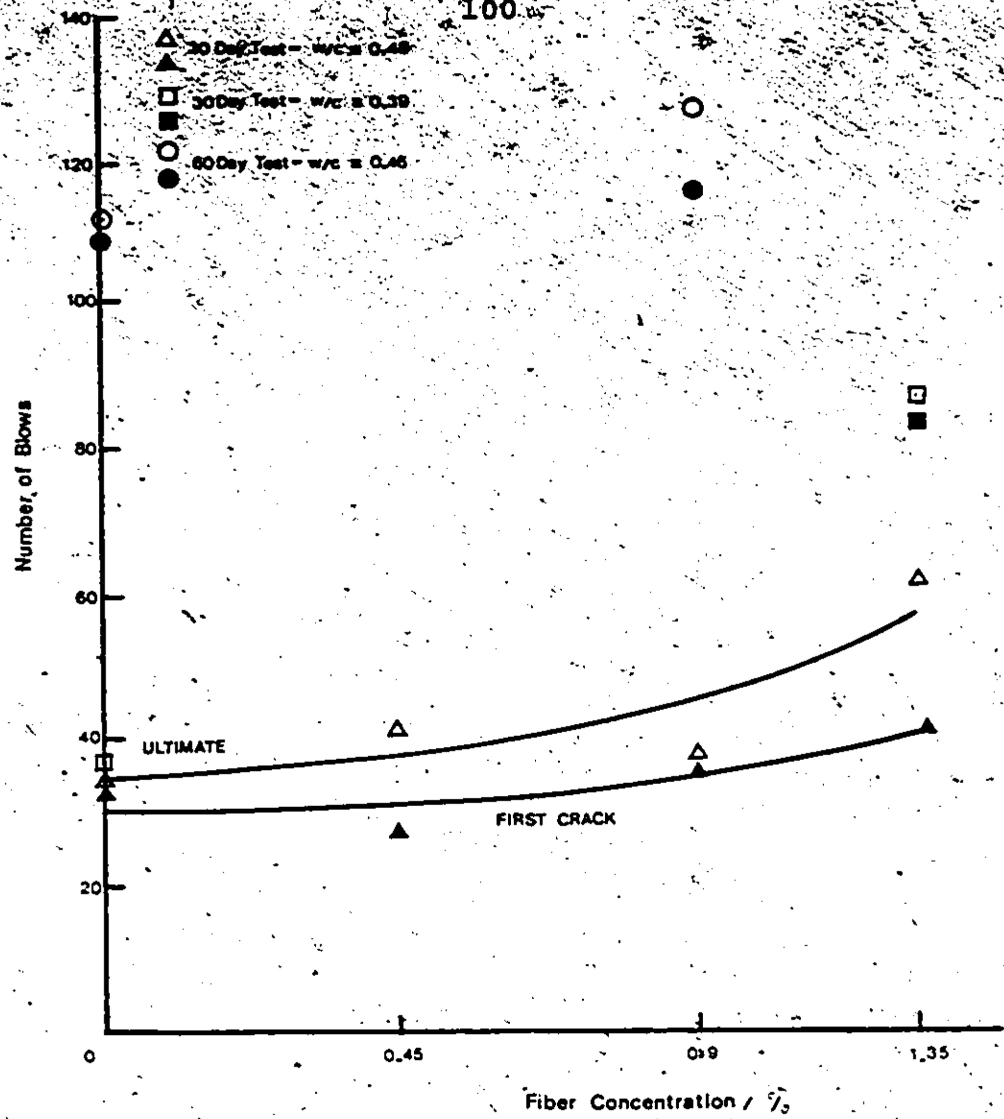


FIGURE 3.23 Influence of Fiber Concentration on Impact Resistance

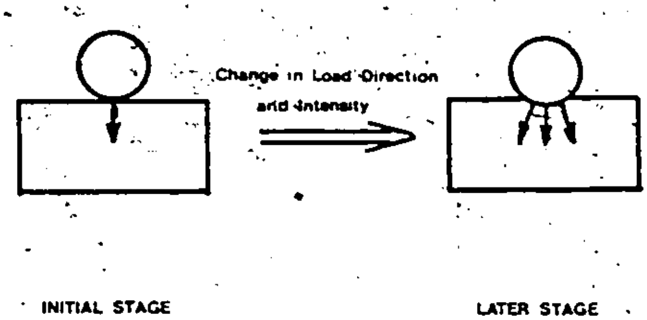


FIGURE 3.24 Change in Applied Load due to Dishing of Specimen

the test is not considered to be a good indicator of impact resistance. With an increased number of blows, the specimen becomes dished changing the direction of the applied load at the interface of the specimen and steel ball as shown in Figure 3.24 resulting in increased transverse loading (wedging"). Therefore, the loading does not remain uniform throughout the test, particularly for concrete using soft aggregate or specimens undergoing a large number of blows.

3.6.6 SCATTER IN STRENGTH RESULTS

Factors contributing to the scatter in flexural testing results include: increased non-homogeneity of mix with inclusion of fibers; change in beam dimensions along length due to bulging of molds; and specimen seating at supports. While the latter two factors have some influence on the results, the increased non-homogeneity of the concrete due to the fibers was considered the prime factor leading to the variation in flexural results and the poor performance of some specimens, particularly those using longer fibers.

Increases in fiber content or length increase the reinforcing of the concrete, however, the strength of the composite may decrease due to the increased number of "flaws" introduced [Rilem, 1977]. This increase in flaws can be attributed to the reduced workability influencing concrete compaction. The increased reinforcement, with resulting flaw generation, is considered to account for the higher static but lower fatigue flexural strength for the concrete using the 2.54 cm fibers. In addition to more flaws, increased fiber content and length also increases the non-uniformity of distribution and probably decreases the random fiber orientation. These factors are discussed further in the next chapter.

The compression and split cylinder tests also showed scatter, and explanations for the scatter and influence of fiber on the respective strength properties can be developed from similar arguments as given for flexural specimens. That is, the increase in strength due to the fibers is related to increased reinforcement, while any decreased strength is due to an increased number of internal flaws, and a decreased random fiber orientation and uniform fiber distribution with increased fiber content or length. The actual strength increase or decrease is dependent upon the predominant factor(s) influencing the strength properties for the particlar mix design and constituents.

The Impact resistance test was the most sensitive test with respect to minor internal variations. Five impact resistance test specimens could be cut from each cylinder

(15.25 cm x 30.5 cm). The average number of blows to failure, and standard deviation, for each specimen is given in Table 3.7. The results indicate that considerable internal variation occurred throughout each cylinder for both plain and fiber concrete specimens; the standard deviation being as large as the average number of blows in several cases. Therefore, aside from the factors contributing to experimental scatter discussed previously, the method of compaction (vibration) and the casting procedure also influence the strength properties. The casting-compaction stage was the only time in which redistribution and reorientation of the concrete constituents could occur.

3.7 SUMMARY

It is apparent from the current study that short fibers at low fiber concentrations had little influence on the static and fatigue flexural strengths of concrete. The poor performance of some fiber concrete mixes is attributed to inability to obtain adequate compaction, uniform fiber distribution and random fiber orientation due to lack of adequate workability. In the fatigue study, the mixes incorporating the shorter fibers showed an increase in fatigue endurance at lower stress ratios which suggests that at stress ratios encountered in pavement design a reduction

TABLE 3.7

SCATTER ANALYSIS OF IMPACT RESISTANCE SPECIMENS (30-DAY TEST)

Specimen	W/C	Percent Fiber	Average Number of Blows		Standard Deviation	
			First Crack	Ultimate	First Crack	Ultimate
Plain	0.39	0	34	36	34	34
2.54 cm Fiber	0.39	1.35	84	97	87	93
Plain	0.46	0	39	41	33	33
Plain	0.46	0	24	27	18	20
2.54 cm Fiber	0.46	0.45	27	42	16	18
2.54 cm Fiber	0.46	0.90	35	39	29	29
2.54 cm Fiber	0.46	1.35	40	61	22	33

in pavement thickness should be possible as anticipated from the success of field experiments. A major research program would be necessary to develop reliable design data for such fiber reinforced pavements.

Compression, split cylinder and impact resistance tests all showed improvements in strength with increases in fiber volume. However, for the compression and impact resistance tests, changes in water-cement ratio had a greater influence on the strengths than the fiber volume or length (compression test). Due to the mode of failure for the various tests, it is apparent that increases in fiber volume or fiber length definitely increase the fracture toughness of the specimens. However, the strength increase or decrease of the composite is also greatly influenced by the increased number of flaws, non-random fiber orientation and non-uniform fiber distribution resulting from incorporating fibers.

The ductility of the concrete increased with increased fiber volume. Split cylinder and impact resistance tests show well defined post-cracking strengths which increase with increased fiber volume. For flexural and compression tests, no first crack strength distinct from ultimate strength could be defined, however, the mode of failure appeared to be more ductile with increased fiber volume and length.

The relative performance of the 1.27 and 2.54 cm fibers was dependent on the mode of failure for the type of test. The shorter fiber mixes were more workable and therefore allowed better compaction, more uniform fiber distribution and more random fiber orientation as discussed previously. As a result, the shorter fiber concrete performed better in the fatigue flexural test and the split cylinder test (lower fiber volumes). For the static flexural, compression and split cylinder tests (high fiber volume), the influence of increased fiber length was greater than the influence of decreased workability and therefore the longer fibers were more effective.

CHAPTER 4

WORKABILITY OF STEEL FIBER REINFORCED CONCRETE

4.1 INTRODUCTION

The strength properties of concrete are largely related to the rheological properties of the composite following mixing. Increased workability without modification of the basic mix is considered to be a key to improvement of the properties of concrete in its hardened state. The observed scatter in laboratory strength tests of fiber concrete is partly due to the difficulty in obtaining a homogeneous mix and adequate compaction due to the inclusion of the steel fibers [Snyder and Lankard, 1972]. For this reason, the workability of steel fiber concrete was considered as an important area in this study that has not received adequate attention in the past. Previous research with Eirich Maohines Limited had indicated that intensive mixing did not help in the workability area, as fiber damage often resulted [Emery, 1977].

In large production field applications the workability of the fresh concrete mix becomes more important, particularly when the composite is produced on site or when the fibers are dispersed by conventional mixing techniques [Rilem, 1977]. A requirement for these applications is that conventional means of placement be used. The well known difficulties in placing and compacting fiber concrete often arise from the simple interference between the fibers and the coarse aggregate, i.e. the relative aspect ratio influence [Swamy and Stavrides, 1975].

From previous studies, it has been recognized that long fibers present a problem with respect to uniform fiber distribution and random fiber orientation [Snyder and Lankard, 1972]. Since the control of crack propogation is dependent on the fiber spacing, it would appear reasonable that shorter fibers could be more efficient than the longer fibers, particularly when considering the improved workability. With superior flow properties, higher fiber concentrations at constant workability could be used for better control of crack propogation.

In recent years, the advent of superplasticizers has lead to the development of high flow concrete. This admixture type has been used in two ways; to increase the flow of concrete at a constant water-cement ratio (flow concrete) or to increase the strength with a decrease in water-cement ratio (water-reduced concrete) [see, e.g., Hewlet, 1978; Meyer, 1978; Sprinkel; 1978]. The use of various workability admixtures with emphasis on superplasticizers was investigated in this study. Additional factors influencing workability were also considered: comparison of 1.27 cm and

2.54 cm long steel fibers; water-cement ratio; aggregate concentration; and aggregate gradation.

4.2 RHEOLOGICAL PROPERTIES OF PLAIN CONCRETE

The factors which influence the workability of plain concrete also affect the workability of steel fiber reinforced concrete. These factors are summarized first in this section; then the effects of the steel fiber addition on the flow properties of fiber concrete are discussed. For a full consideration of workability, the stability of the fresh mix must also be taken into account.

4.2.1 WORKABILITY OF PLAIN CONCRETE

Much work has been conducted on the workability of concrete. Tattersall [1976] has listed the following factors which influence the flow characteristics of the composite:

1) ellapsed time since mixing;
2) properties of the aggregate, particularly the particle shape and size distribution, porosity and surface texture;
3) properties of the cement to a small extent;
4) presence of admixtures; and
5) relative portions of the mix constituents.

The water-cement ratio is an important factor governing the concrete quality and should be kept as low as possible since it influences the strength of the concrete [BRS, 1971].

However, enough water must be available to allow for proper cement hydration and water absorption by the aggregate, in addition to some available free water to maintain adequate workability. The quantity of cement in the mix is also influenced by the relative cost between it and the other constituents of the composite [see, e.g., Cordon and Thorpe, 1975]. Since the cost of cement is much higher than the other elements of the mix, the concrete quality is often slightly "sacrificed" by cement reduction and/or substitution.

One further factor of significance is the influence of the aggregate on the workability. The aggregate-cement ratio should be kept as low as possible in order to reduce the amount of creep and shrinkage. However, increased amounts of aggregate above that for a suitable mix tend to reduce the workability considerably. The aggregate gradation also influences the flow of the concrete. Increased fines reduce the harshness of the mix, but decrease the workability due to a higher specific area to be covered by the paste. To maintain adequate flow the paste content must be increased resulting in higher costs and in higher creep and shrinkage [see, e.g., PCA, 1968; BRS, 1971; Hobbs, 1976; and Tattersall, 1976].

4.2.2 STABILITY OF PLAIN CONCRETE

The stability of a mix has been shown to be an important aspect to the rheology of fresh concrete. It can be characterized by the bleeding and segregation tendencies, and is of considerable importance to the production of a satisfactory and durable mix [Ritchie and Rahman, 1973].

The following factors increase the tendency of concrete to segregate [Tattersall, 1976]:

1) increased particle size over 25 mm and decrease in fines;
2) increased specific gravity of coarse aggregate compared to specific gravity of fine aggregate;
3) change from continuous grading to gap grading;
4) unfavourable change in particle shape;
5) decreased cement content; and
6) change in water content making the mix too dry or wet.

Bleeding refers to the appearance of water at the surface of the concrete after consolidation. This is a natural property of the concrete and as long as localized channels do not develop, bleeding is not considered unfavourable. Generally, the bleeding rate can be reduced by [Tattersall, 1976]:

1) increased cement finess;
2) increased cement content;
3) addition of pozzolanas;
4) decreased water content; and
5) addition of air-entraining agents.

4.3 RHEOLOGICAL PROPERTIES OF STEEL FIBER CONCRETE

The addition of fibers to a wet mix reduces the workability of concrete. The loss in flow is twofold; being due to both the increase in surface area and increase in harshness because of fiber geometry [Rilem, 1977]. For this reason, additional factors must be considered with the aim of obtaining a homogeneous mix by both the use of an appropriate mix design and a correct mixing procedure [Moens, 1976].

The ability of fiber concrete to flow is dependent on the volume of paste surrounding each individual fiber. With increased fiber content more of the free paste available for flow is used to cover the additional fibers. In addition, a small amount of the free water is lost due to absorption at the fiber surface. It is the actual fiber geometry which encourages bundling and interlocking of the fibers [Mangat and Swamy, 1974], with the severity of interlocking and bundling related to both the fiber volume and fiber aspect ratio. Increased fiber concentration or aspect ratio results in the decreased ability of the fiber to rotate due to increased fiber-fiber interaction, i.e. decreased fiber spacing [Moens, 1976].

The volume fraction and gradation of the aggregate have a substantial influence on the flow of fiber concrete. From a strength viewpoint. the coarse aggregate must have its largest dimension smaller than the fiber length in order for the fiber to be effective [Swamy and Stavrides, 1975]. With regard to workability, the interaction of the coarse aggregate and fibers produce a very harsh mix. Therefore, conventional concrete mixes cannot be used with steel fiber reinforcement. A greater percentage of fines is required to reduce the concrete's harshness, which in turn requires an increase in paste in order to coat the increased aggregate surface area. The maximum size of coarse aggregate suggested for fiber concrete is 9.5 mm [McCurrich and Adams, 1973].

With higher paste requirements, fiber concrete is more susceptable to creep and shrinkage [see, e.g., Halvorsen et al., 1976; PCA, 1976; and Rilem, 1977]. To reduce the amount of water in the paste, additives such as water reducing agents, plasticizers or fly ash are used to maintain adequate workability. The higher cement content, addition of fibers and use of additives increase the unit cost of producing the composite. This increase in production costs must be offset by improved engineering properties before the composite may be considered commercially feasible.

The factors influencing the flow of steel fiber concrete can be summarized as:

1. increased aspect ratio reduces workability;
2. increased volume fraction of fiber reduces workability;

3. fiber type, surface area and roughness influence paste requirements;
4. size and smoothness of coarse aggregate influence fiber-aggregate interaction;
5. composite's stability, i.e. fibers reduce bleeding and control segregation; and
6. technique of placing, i.e. vibration.

Unless sufficient compactability of concrete with random fiber orientation and uniform fiber distribution can be obtained in the field, any beneficial properties demonstrated in the laboratory with workability admixtures and shorter fibers are of no help. Field demonstration is beyond the scope of this study, but recent experience in the United States tends to confirm the favourable workability trends observed with short fibers [see, e.g., Gray and Rice, 1972; Gray, 1972; Arnold and Brown, 1973; Lankard and Walker, 1975; and Schrader and Munch, 1976].

4.4 WORKABILITY MEASUREMENTS

The effect which the fibers have on the major types of workability tests will be discussed briefly so that the significance of the workability study results will be clearer. The three common workability measures are the slump (ASTM C143), compaction factor and Vebe tests (Part 2 of BS1881). Each test depends on a different mechanism to allow for the measurement of workability.

The slump test makes use of the concrete's bulk weight to initiate flow. The flow continues as long as the driving force (i.e. body weight) is greater than the resistance to flow due to the fiber and aggregate interactions. The compaction factor test also relies on the mix's body weight for workability measurement. Unlike the slump test which is static, the compaction factor test may be considered "pseudo dynamic", where the concrete is allowed to fall through two stages. The density of the concrete after self compaction is compared to the concrete density after vibration. For plain concrete, the slump test is regarded as a sensitive indicator of changing workability since the range covered is large. The compaction factor test is considered to be a more exact, but more complex test for workability.

For stiffer mixes which are often encountered in fiber concrete mixes; the sensitivity of these two tests is greatly reduced. A more reliable workability indicator for low flow concrete is the Vebe test which better simulates the method of compaction encountered in practice [Mangat and Swamy, 1974]. This test measures the input of energy required to deform a constant volume of concrete a predetermined amount. However, unless the flow is near "zero slump" the Vebe measurement is operator dependent. Therefore for fiber concrete mixes, one type of test alone is not sufficient for measuring the range of flows encountered.

The approximate correlation given by Tattersall [1976] between slump and Vebe is given in Table 4.1. The relationship between the various workability measurements and varying fiber concentrations reported by Ritchie and Rahman (1973) is shown in Figure 4.1.

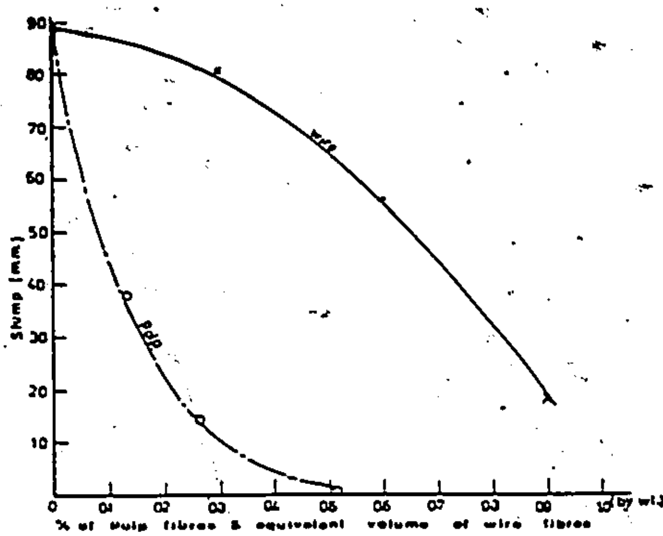
TABLE 4.1

CORRELATION BETWEEN SLUMP AND VEBE TESTS

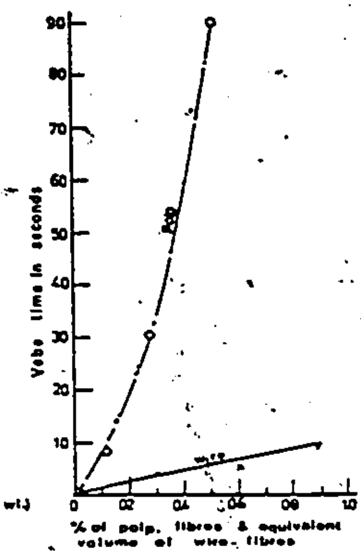
[Tattersall, 1976]

Slump (mm)	Vebe (Sec.)
0 - 10	12
10 - 30	6 - 12
30 - 60	3 - .6
60 -180	0 - 3

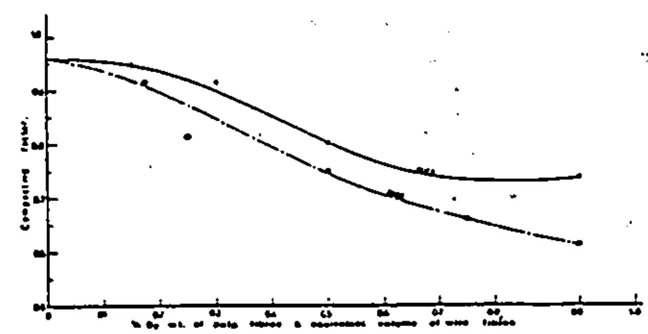
A test which appears to have great potential for measuring the workability of fiber concrete is the concrete flow table described in DIN 1048. A fixed quantity of concrete is placed on a smooth surface which is then vibrated in a standard manner. The total energy input is constant and the radial deformation due to energy input is



(a) The effect of fiber concentrations on slump



(b) The effect of fiber concentrations on vebe time



(c) The effect of fiber concentrations on compacting factor

FIGURE 4.1 Influence of Fiber and Fiber Concentration on the Workability of Concrete Measured by Slump, Vebe, and Compaction Factor Tests measured. Unlike the other more common tests, the flow table test is not as operator dependent. The test also represents the real placing situation of concrete a little closer, even though it is not as sensitive as the slump test over the wide range of workabilities covered by a plain concrete mix [CAA and CCA, 1977]. However, the test would appear to be more sensitive in the range of workabilities covered by a typical fiber concrete mix.

4.5. EXPERIMENTAL PROCEDURE AND TESTING

The objective of this testing program was to study the workability of fresh steel fiber reinforced concrete just after mixing. The workability is of considerable importance with respect to the placing and compaction of the composite in the field as indicated previously. Besides affecting the long term strength properties, the need for a workable mix is reflected in the cost. The more work required to place the composite, the less attractive is the investment for using the fiber additive unless the cost is readily offset by demonstrated performance improvement. The following parameters were considered to be of interest:

1. fiber length and content;

2. use of workability admixtures;
3. aggregate-cement ratio (A/C);
4. coarse-fine aggregate ratio (C'/F); and
5. water-cement ratio (W/C).

The slump, compaction factor and vebe tests were used to measure the workability. Besides measuring the influence of the parameters on the workability of fiber concrete, the various tests were compared with respect to their sensitivity. Due to the lack of a suitable concrete flow table, the effectiveness of this test could not be directly compared to the other flow tests. However, a mortar flow table was used to measure the influences of the parameters for steel fiber concrete. Because of the small flow table mold volume to coarse aggregate size ratio, this measure of workability was not considered to be sensitive, i.e. the effect of the coarse aggregate could overshadow the influence of the parameter under consideration. This test was only used in conjunction with the results obtained by the other measures of workability and appears to have potential when increased in size to accommodate concrete mixes.

4.5.1 MIXES AND MATERIALS

The mix design around which the study of parameter influences was carried out is referred to as the "Stelco Mix Design for 1/2" Aggregate. Table 4.2 gives the fiber mix along with the suggested control mix. More details with regard to the Stelco mixes are given in Appendix 8.

TABLE 4.2

STELCO 13 mm (1/2") AGGREGATE, STEEL FIBER AND CONTROL MIXES

Mix Constituent	Steel Fiber		Control	
	kg/m^3	(lb/yd^3)	kg/m^3	(lb/yd^3)
Coarse Aggregate	936.94	(1580)	965.4	(1628)
Fine Aggregate	907.29	(1530)	934.63	(1576.1)
Cement	382.49	(645)	394.35	(665.0)
Water	177.90	(300)	181.40	(305.9)
Steel Fiber	71.16	(120)	-	-
Admixture[1]	-	-	-	-
TOTAL	2475.78	4175	2475.78	4175

[1]Porzite L-70, Dosage 400 ml/100 kg cement.

Symbol 10 normal Portland Cement supplied by the St. Lawrence Cement Company was used throughout the workability study. All bags were sealed in plastic bags as in other studies to ensure minimum hydration due to moisture in the air. The fine aggregate was a concrete sand from the Lake Erie bar and the coarse aggregate was a 3/8" ACI crushed and washed limestone (i.e. few or no fines), from Canada Crushed Stone, Dundas (supplier of all aggregates). The grading curves for the two aggregates are given in Appendix 9. The fibers came in 1.27 cm and 2.54 cm precut lengths as supplied by Stelco. The details for the workability mixes tested are given in Table 4.3 and Table 4.4. The first table summarizes the mixes used for the strength tests. These mixes were made over a few weeks while the mixes given in the second table were all completed in one day in order to maintain a better control over the variation in the aggregates' moisture contents.

TABLE 4.3

MIX DETAILS FOR WORKABILITY DURING CASTING OF STRENGTH SPECIMENS

Series	W/C	A/C	C/F	Fiber Length	Fiber Concentration	Admixture
A	.39	4.82	1.033	2.54 cm	0,.45,0.9,1.35%	Porzite L-70
C,D,F, G,H,I, J,K	.46	4.82	1.033	—	—	Porzite L-70
B,D,L	.46	4.82	1.033	2.54 cm	.45,0.9,1.35%	Porzite L-70
C,F,G, H,I,J, K	.46	4.82	1.033	1.27 cm	.45,0.9,1.35%	Porzite L-70

TABLE 4.4

WORKABILITY STUDY MIXES

Series	W/C	A/C	C'/F	Fiber	Admixture	Variable
I	.41	4.82	1.033	2.54 cm	Porzite L-70	Fiber Concentration
II	.41	4.82	1.033	1.27 cm	Porzite L-70	Fiber Concentration
III,IV,V,VI	.41	4.82	1.033	1.27 cm-0.9%	Note*	Additive Concentration
VII	.41	-	1.033	0	Porzite L-70	A/C
VIII	.41	4.82	-	0	Porzite L-70	C'/F
IX	.41	-	1.033	1.27 cm-0.9%	Porzite L-70	A/C
X	.41	4.82	-	1.27 cm-0.9%	Porzite L-70	C'/F

NOTE* The various admixtures used include: Melment L-10, Mulcoplast CF, Porzite L-77 and Porzite L-76 with maximum dosages being 4500 ml, 2000 ml, 380 ml and 400 ml per 100 kg cement, respectively.

The workability results from the strength study (40 mixes) were used to detect large deviations in the mixes for control purposes. These mixes were also used to check the consistency between the Vebe and slump tests. For the workability study, thirty mixes were made to determine the influence of changing the parameters.

4.5.2 PREPARATION AND TESTING

All of the aggregate was bagged the day before the mixing in order that the moisture content measured would remain relatively unchanged between time of testing for moisture and mixing. To save time, all aggregates, cement and fiber were preweighed to the desired batch proportions. Only the water and admixture for the mix were measured on the day of testing since moisture adjustments due to water held by the aggregate had to be taken into account. The procedure for the adjustment of the water content is given in Appendix 10.

With the exception of the flow table work, all mixing was performed in an Eirch E-2 batch mixer. The order of adding the mix constituents and the mixing times are given in Table 4.6. The concrete for the strength and workability studies was batched in .056 and .028 cubic meter volumes respectively. The times given in Table 4.6 refer to the mixing time after

TABLE 4.5

INTRODUCTION OF MIX CONSTITUENTS AND

SUBSEQUENT MIXING TIMES

Constituent Added	Mixing Time (Sec)
Fine and Coarse Aggregate	15
Cement	10
Water with Admixture	60
Fibers	180

the constituent was added except for the fibers which were sieved through a "1/2"" sieve to ensure even distribution throughout the concrete within the 180 seconds. The admixture was mixed with the water before the batching procedure. Workability measurements were taken immediately after mixing. Slump and vebe tests were completed for all batches, however the compaction factor was only used for a few mixes since it was not found to be a reliable workability measure for fiber concrete. The washout test, which is described in Appendix 11, was used to check the percentage of material in a mix. Further discussion regarding these tests is given in the next section.

The flow table work was done to confirm the influence of the parameters measured by the other tests in the workability series. The concrete was mixed in a small bowl mixer generally used for mortars (ASTM C305) with the mixing time given in Table 4.7.

TABLE 4.6

ADDITION OF CONCRETE CONSTITUENTS AND RESPECTIVE MIXING TIMES USING HOBART BOWL MIXER

Constituent Added	Mixing Time (sec)
Fine and Coarse Aggregate	10
Cement	15
Water and Admixture	30
Fiber	90

Additional ASTM standard procedures used for mixing and testing include:

C127 TEST FOR SPECIFIC GRAVITY AND ABSORPTION OF COARSE AGGREGATE:

C128 TEST FOR SPECIFIC GRAVITY AND ABSORTION OF FINE AGGREGATE; and

C230 FLOWTABLE FOR USE IN TESTS OF HYDRAULIC CEMENTS.

4.6 DISCUSSION OF RESULTS

4.6.1 SENSITIVITY OF MEASUREMENTS

The results are restricted to a limited range of fiber concentrations (i.e. 0 to 1.35 % fibers by volume) in which the fibers still have some freedom of rotation and translation. However, the concentrations were high enough to measure definite changes in workability which resulted largely from the fiber-fiber and fiber-coarse aggregate interactions, and are representative of field applications. There was no apparent balling of fibers even with the longer fibers. However, at higher fiber concentrations, the failed concrete specimens revealed increased non-uniform fiber distribution for longer fiber mixes. This non-uniformity was less severe for the shorter fiber mixes at higher fiber concentrations (i.e. 1.35%).

The study did show that the vebe test was far superior to the slump test as a workability measure for low flow concrete. This is shown clearly in Figure 4.2(a) where for a constant zero slump, the workability measured by the vebe test indicated definite changes in workability. As the workability of the concrete increased, the sensitivity of the vebe test decreased and for high flows the workability measurements became unreliable. For the higher workability mixes, the slump test was a better indicator. Figures 4.3 and 4.4 illustrate the sensitivity of the various tests while Figure 4.5 gives the approximate relationship between the slump and vebe tests.

The compaction factor test indicated relative changes in workability with changes in fiber concentration and/or water-

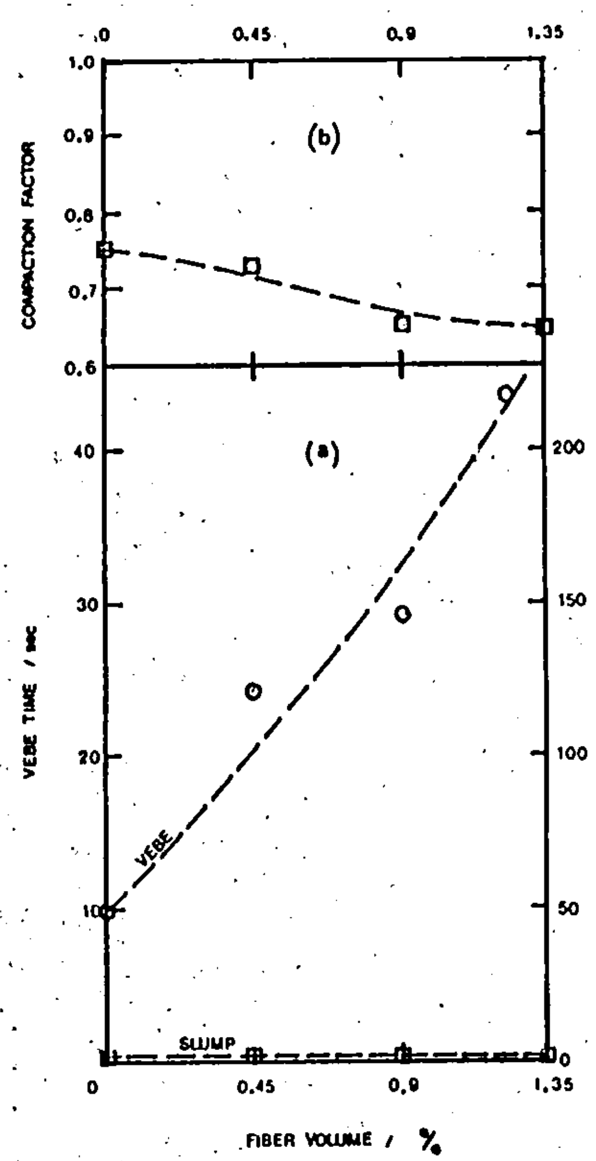


FIGURE 4.2 Influence of Fiber Volume on Workability Using 2.54 cm Fiber Length at w/c=.39

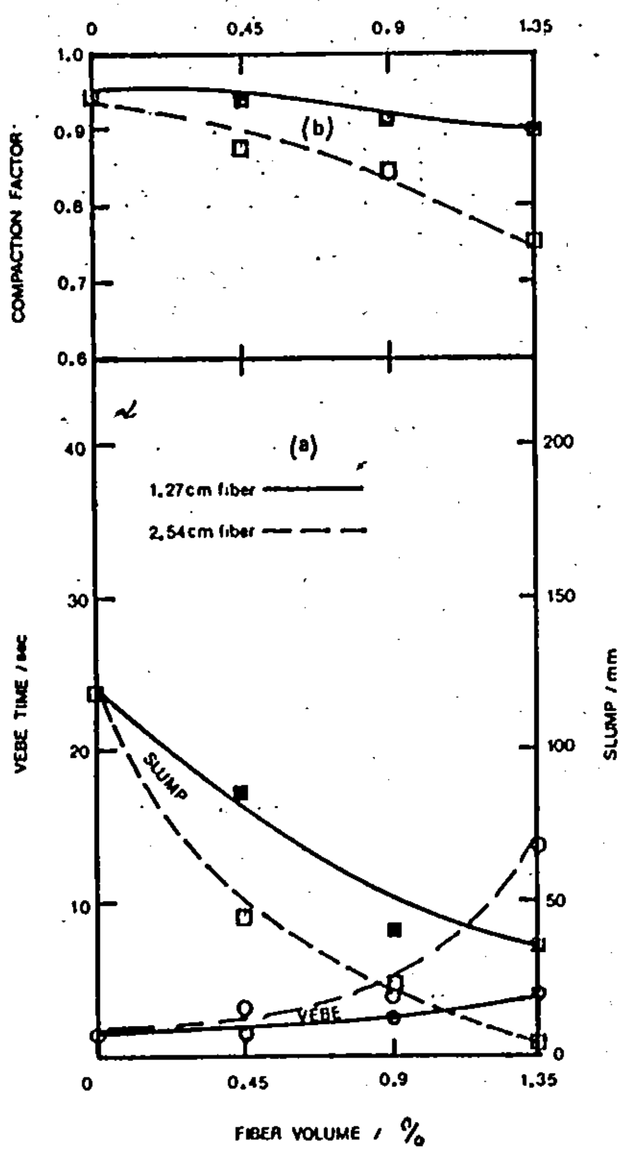


FIGURE 4.3 Influence of Fiber Volume on Workability at w/c=.46

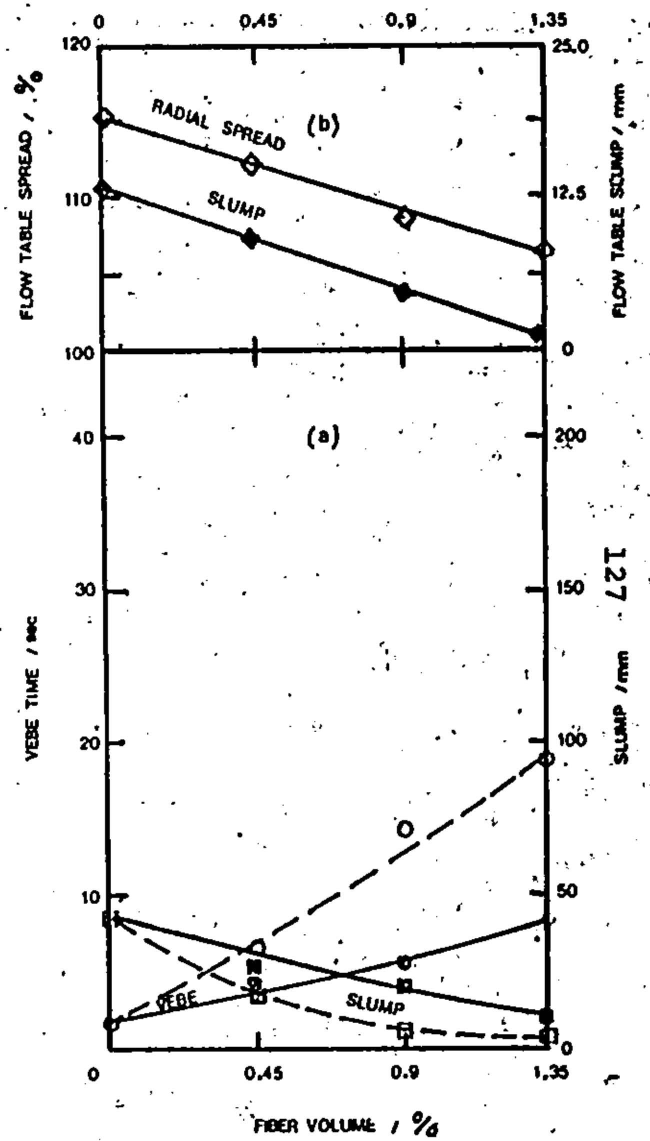


FIGURE 4.4 Influence of Fiber Volume on Workability at w/c=.41

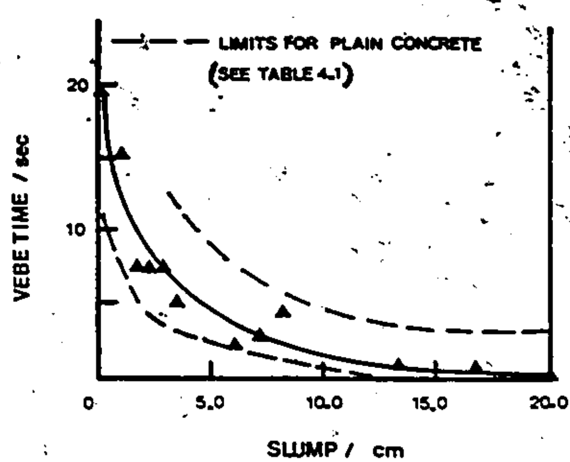


FIGURE 4.5 Relationship Between Slump and Vebe Time from Current Study

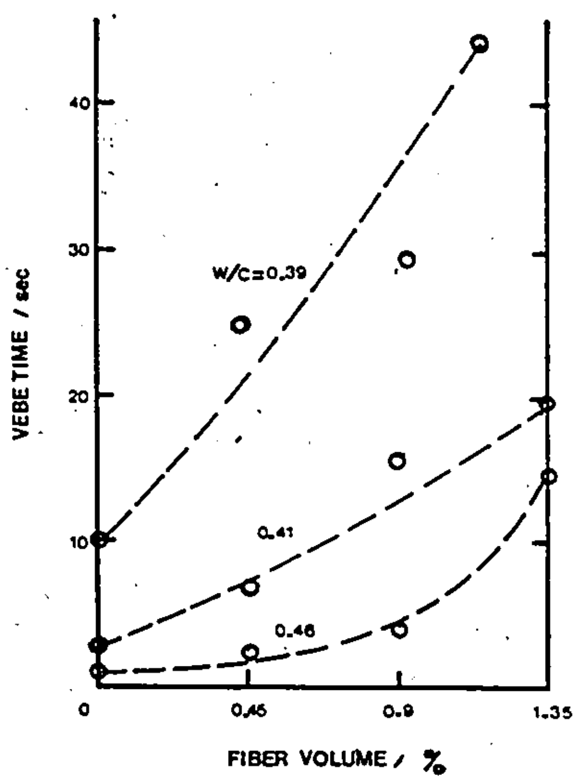


FIGURE 4.6 Influence of Fiber Volume on Workability of Fiber Concrete at Various Water-cement Ratios (using 2.54 cm fiber length)

cement ratio. These changes are shown in Figures 4.2(b) and 4.3(b). However, in comparison to the other tests for workability, the compaction factor test was only moderately sensitive. This would suggest that while there was adequate "pseudo dynamic" force to cause relative changes in the self compacted density of all mixes, this force did not produce adequate sensitivity for the workability tests in this study.

The flow table test results suggested a linear relationship between the workability of concrete and the change in fiber concentrations. The relationship between the radial spread (and slump) and fiber concentration is shown in Figure 4.4(b).

In almost all cases the flow table results confirmed the effect of the parameter which had been measured by the more common tests. However, when the flow was relatively high, segregation of the paste from the aggregate and fibers resulted. This was attributed to the small mold - coarse aggregate size volume ratio discussed previously. It is believed that with the use of a large concrete flowtable, the influence of the coarse aggregate would have been minimized. From the few tests completed in the study, it is suggested that the large flow table test has great potential for measuring the workability of fiber concrete. Additional workability results concerning flowtable measurements is given in Appendix 12.

4.6.2 INFLUENCE OF FIBER LENGTH AND FIBER CONCENTRATION

For very low flow concrete, the vebe test was more responsive to changes in workability while for higher flows the slump test was more responsive. The remaining discussion is focused on the influence of the various parameters on the workability of concrete. To maintain charity with regards to indicating the influence of the steel fibers on the workability of concrete, only the most apprepriate test is referred to when discussing the results.

Much of the emphasis in the workability study was placed on the response of the concrete flow to changes in fiber length and fiber concentration. The results of changes in these parameters are shown in Figures 4.3 and 4.4. It is apparent from the figures that the shorter fiber mixes (i.e. 1.27 cm) had greater workability due to less fiber-fiber and fiber-matrix interactions. This would also suggest increased degrees of freedom for the fibers to rotate and translate. Therefore the spacing equations discussed in Chapter 3 should also incorporate the length of the fiber as suggested by Swamy, Mangat and Rao [1973]. The reduction in fiber-fiber interaction for the short fibers was also noticed when handling the fibers before adding them to the mix. Unlike longer fibers, the shorter fibers did not cluster when removed from the shipping boxes.

All tests showed that increases in fiber concentration resulted in decreased workability. The influence of the fiber concentration also increased with decreased water-cement ratio. It is suspected that this trend was largely due to the absence of sufficient free paste required for the concrete to flow. For a water-cement ratio of .39, only 1.25% fibers by volume could be achieved before the mixer was stopped through lack of power. The influence of fiber concentration at different water-cement ratios is shown in Figure 4.6. No definite relationship was obtained between the workability of concrete and fiber concentration for the slump and vebe tests.

4.6.3 INFLUENCE OF PASTE VOLUME

The remaining discussion reflects the influence of paste volume and the effect of various workability admixtures on the flow of steel fiber concrete. For a steel fiber concrete mix at constant fiber volume, the free paste available for flow can be altered in the following ways:

1. changing the water-cement ratio;
2. changing the aggregate-cement ratio; and
3. varying the gradation of the aggregates.

The proportioning of concrete mixes is dependent on the workability of the fresh concrete, required properties of hardened concrete and economy [PCA, 1968]. From economic considerations, the amount of cement required should be minimized without sacrificing concrete quality. Since the quality depends primarily on the water-cement ratio, the water content should also be minimized without considerable loss in workability. The influence of the water-content on the workability of concrete is shown in Figure 4.7.

The test results indicated that for a fiber concentration of 1.35% by volume, little change in workability of the low flow concrete occurred between water-cement ratios of .41 and .46. The small change in workability confirmed the importance of the fiber influence on the flow characteristics of concrete. The results suggested that the concrete quality could be increased without a great loss in workability. However, the tests also indicated that for water-cement ratios less than .41, a rapid decrease in workability resulted as measured by the vebe test. This rapid decrease would suggest that the minimum paste requirement to cover the fibers and aggregate was being reached leaving very little free paste for flow.

Studies involving the sensitivity of the workability of steel fiber concrete to changes in aggregate-cement ratio and aggregate gradation are shown in Figures 4.8 and 4.9 respectively. The influence of the fibers on the workability of concrete again appeared to be very important. The fiber

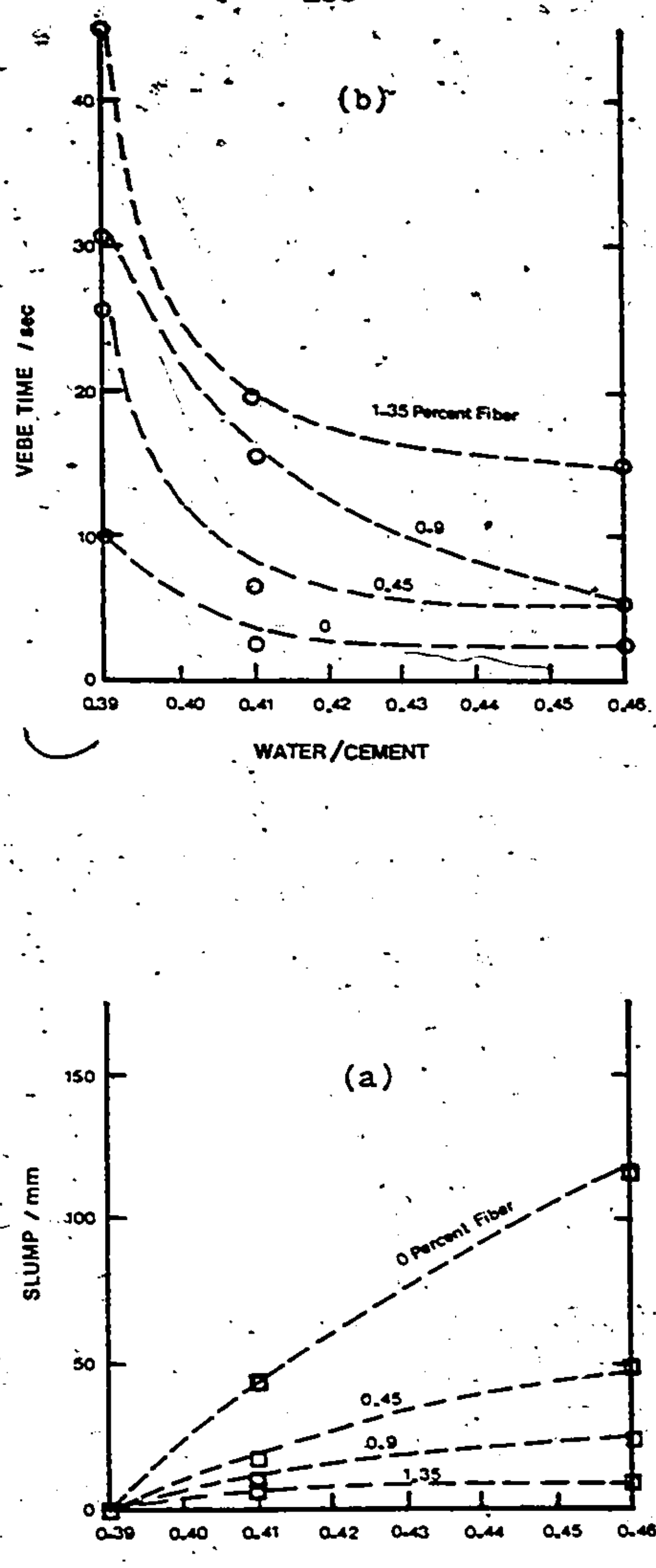


FIGURE 4.7 Influence of Water-cement Ratio on Workability of Fiber Concrete (using 2.54 cm fiber length)

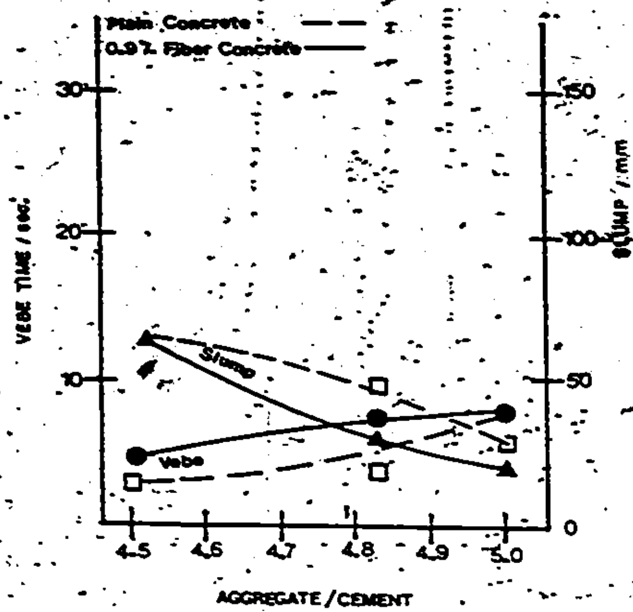


FIGURE 4.8 Effect of Aggregate-cement Ratio on Workability of Fiber Concrete

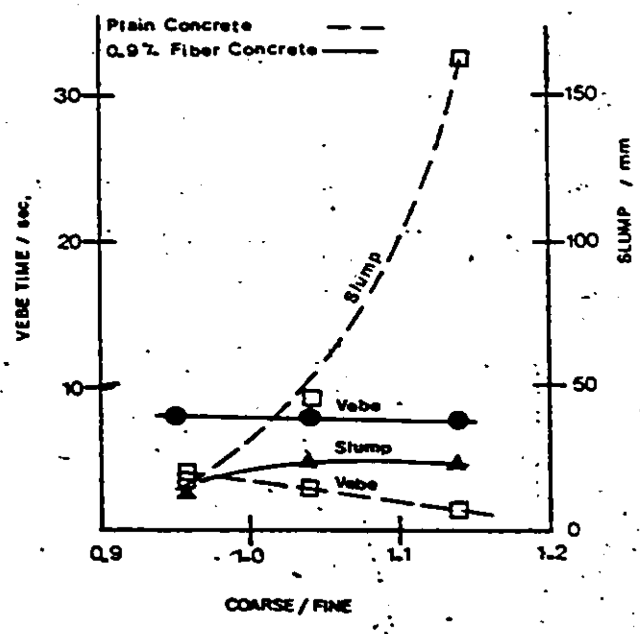


FIGURE 4.9 Effect of Gradation on Workability of Fiber Concrete

mix responded more quickly to an increase in aggregate content than did the plain concrete mix. However, the influences of the fibers decreased with a further increase in aggregate-cement ratio.

To study the influence of the gradation on the fiber concrete mix, the proportion of fines was altered by plus and minus 5 percent from the Stelco mix. No change in workability was observed when the fines in the fiber mix were altered, however a substantial increase in workability was observed with decreased fines in the plain concrete mix. This behaviour is shown clearly in Figure 4.9.

At higher fiber concentrations, the influence of the fibers to a large degree superceded any of the other constituent changes (W/C, A/C, gradation) which would suggest that the optimization of a mix lies in economic, harshness of concrete and strength considerations. The important influence of workability admixtures is discussed in the following section.

4.6.4 INFLUENCE OF WORKABILITY ADMIXTURES

Two groups of admixtures were used; normal plasticizers and superplasticizers. Both groups of admixtures are workability agents and allowed a reduction in water content, however, they are chemically distinct from one another. The results of the admixture study are shown in

Figures 4.10(a), (b), (c) and (d). The figures clearly show that the superplasticizers imparted greater workability to the fiber concrete mix than did the normal plasticizers.

At the lower dosages, the superplasticizer mixes, particularly the Melment L-10, handled better than the concrete mixes containing the normal plasticizers at full dosage even though the workabilities were similar. The superplasticized concrete showed signs of bleeding at full dosage just after casting of specimens, yet they set faster than the concrete using the normal plasticizers. Plain concrete mixes were not tested and therefore the full influence of the fibers on the workability of concrete using different admixtures was not covered. However it is suspected that the fibers still influenced the workability of the admixture fiber concrete, but the degree is not known.

4.6.5 RELATIONSHIP BETWEEN WORKABILITY AND FLEXURAL STRENGTHS

It was mentioned previously that the observation of fractured surfaces of the failed specimens showed an increased number of flaws and non-uniformity of fiber distribution with increased fiber length or concentration. While the influence of improper seating and variable geometry have some affect on the strength of the specimens, the flaws and fiber distribution appear to be the prime factors

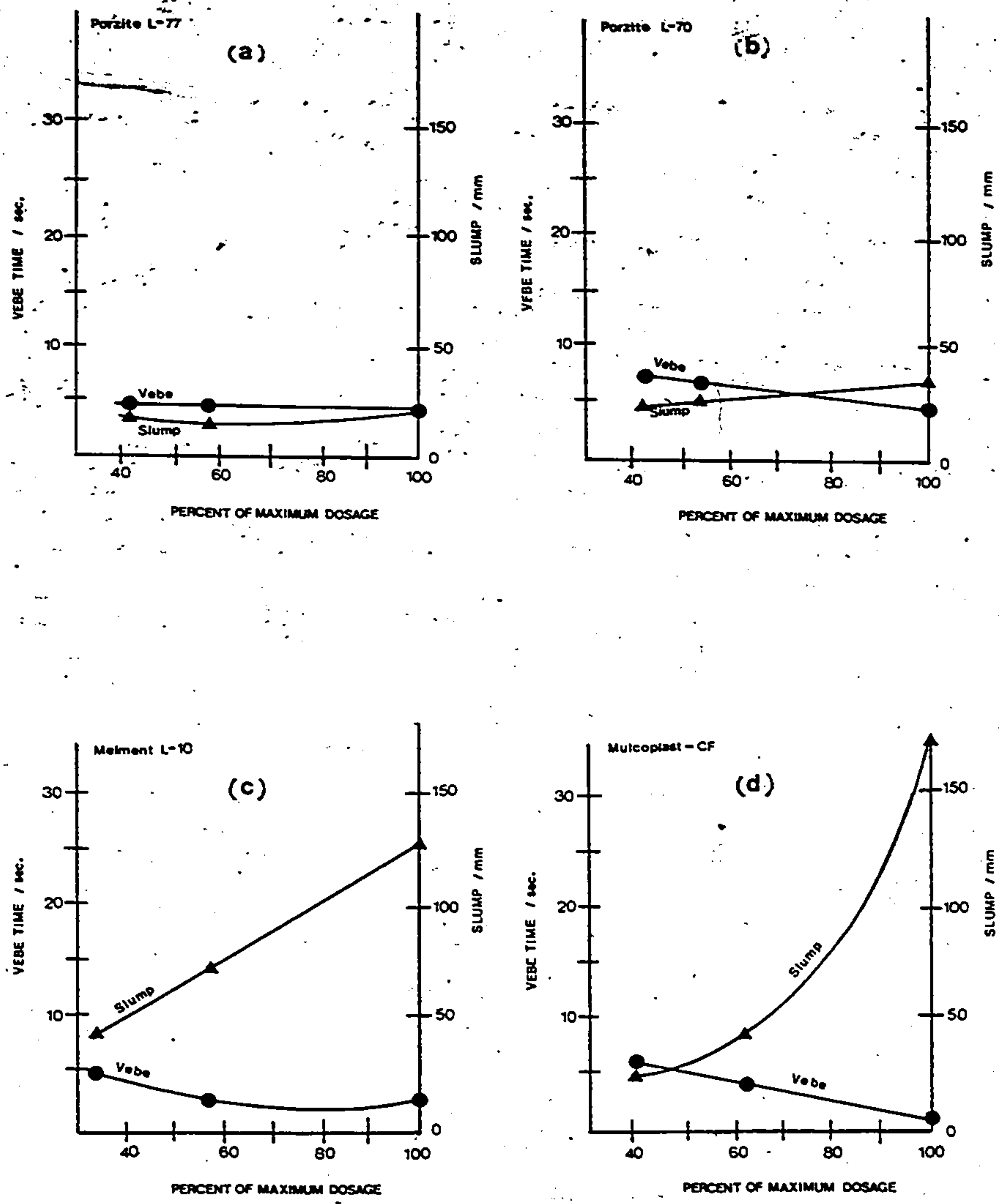


FIGURE 4.10 Influence of Workability Admixtures on the Workability of Fiber Concrete

influencing the concrete's strength. It was indicated in Chapter 3 that the actual strength increase or decrease with an increase in fiber content or length is dependent on the predominant factor influencing the strength; i.e. the fibers which increase strength or the flaws and non-uniform fiber distribution which decrease the strength. The negative factors are dependent on the workability of the composite which in turn is influenced by the inclusion of fibers. Therefore an optimum fiber mix is controlled by both the desired strength increase and necessary workability of the composite.

In order that the strength properties can be predicted with a higher degree of confidence, the quality of the fiber concrete composite must remain uniform. To maintain the desired quality, the fibers must be uniformly distributed and randomly orientated. To achieve this, a more workable mix is required so that the constituents can flow freely without excessive interparticle interferences which is inherent in fiber concrete. It is believed that with the use of superplasticizers, a more dependable steel fiber concrete mix can be obtained.

4.7. SUMMARY

Increases in fiber length or fiber concentration had a marked influence on the workability of concrete. The fiber influences at higher fiber concentrations appeared to supercede the influence of the small changes in the other parameters to a large degree. Only the superplasticizers showed a large improvement in the workability of steel fiber concrete. The workability of fresh concrete is very important in establishing a high quality concrete since the ability to achieve a uniform composite is related to the capability of the mix to flow into place without excessive interparticle interference.

CHAPTER 5

CONCLUSIONS

Research on steel fiber reinforced concrete has shown that although the composite has great potential as a superior pavement material, the actual improvement in pavement performance is very much dependent on the workability of the fiber concrete. The lack of workability due to the presence of fibers affects the uniformity of fiber distribution, random fiber orientation and compaction, which in turn influence the mechanical properties of the composite. Therefore, the influence of fiber length and concentration on the workability and mechanical properties of steel fiber reinforced concrete was considered in detail. In addition, fiber pull-out tests were completed in order to determine optimum fiber lengths.

The development of sufficient bond strength is of prime importance to the reinforcing ability of the fiber. To fully utilize the fiber's tensile strength, a minimum fiber development length is required which was shown to lie between 1.27 and 1.91 cm using a direct pull-out test. The predicted critical fiber length was shown to be 1.8 cm which lies in the range of the experimental optimum length. Under actual loading conditions encountered in practice, longer fiber lengths are required due to the random fiber distribution and reduced efficiency of fiber groups. The direct pull-out tests indicated that an optimum paste requirement appears to exist. Although sufficient paste is required to coat the fibers, too much paste can reduce the fiber-matrix bond strength. The fiber-matrix bond strength was also shown to be dependent on curing method and actual fibers used for pull-out testing.

Plain fibers at low fiber concentrations were found to have a limited influence on the static flexural strength of concrete; an increase of only 3 percent as fiber volume increased from 0 to 1.35 percent for short fibers (1.27 cm), and an increase of 16 percent at a fiber volume of 0.9 percent for long fibers (2.54 cm). The higher strength of the concrete specimens incorporating the long fiber was attributed to superior bond development at the fiber-matrix interface. Visual inspection of the fractured surfaces of these specimens indicated non-uniform fiber distributions and flaw generation due to the decreased workability of the fresh concrete. These factors are expected to decrease the potential strength increase with the incorporation of fibers. Additional factors observed due to the fiber incorporation include:

1. Although the mode of failure became more ductile with increased fiber content, the first crack and

ultimate strengths were not distinct from one another, leaving little post-crack strength.

2. For the concrete incorporating the short fiber, the flexural strength increase was not monotonic with increasing fiber content initially the flexural strength decreased before increasing.
3. The elastic modulus was not influenced by the addition of fibers, however, the modulus did increase with precyclic loading.

The specimens incorporating the short fiber showed an increase in fatigue flexural strength with an increase in fiber concentration, whereas the specimens with the long fibers showed a strength decrease. At a stress ratio of 75 percent of first crack strength, the short fibers increased the fatigue endurance from 90,000 to 400,000 cycles while the long fibers decreased the fatigue endurance to 40,000 cycles as the fiber volume increased from 0 to 1.35 percent. The poor performance of the specimens incorporating the long fiber is attributed to the susceptability of the fatigue strength to the increased non-uniformity of fiber distribution and flaw generation discussed previously. Increases in fatigue strength using the short fiber indicated significant improvement only at the lower stress ratio (i.e. at 75 percent); at the 90 percent stress level the fatigue strength was actually slightly less than obtained with plain concrete. Due to the limited amount of data developed no fatigue design criteria were established in the study.

Additional tests completed on compressive, split cylinder and impact resistance specimens indicated:

1. The compressive strength was influenced more by the water-cement ratio than increases in fiber length or concentration; a decrease in water-cement ratio from 0.46 to 0.39 increased the compressive strength by 50 percent. The short fibers showed no improvement with increased fiber content, while the long fibers increased the compressive strength by 17 percent as the fiber volume increased from 0 to 1.35 percent.
2. The split cylinder strength increased with an increase in fiber concentration; an increase of 33 and 45 percent using short and long fiber reinforcement, respectively, as the fiber volume increased from 0 to 1.35 percent. The specimens with short fibers showed a maximum strength increase of 42% at a fiber volume of 0.9 percent which then decreased with further fiber addition. The specimens incorporating the long fiber showed monotonic strength increase with increased fiber content. The post-cracking load increased as fiber content and length increased.

3. The impact resistance strength increased with increased fiber concentration, increased curing time and decreased water-cement ratio; increases in ultimate strength were greater than increases in first crack strength. Too few tests were completed to relate the strength increase to the changes in variables.
4. The actual strength increase or decrease of specimens depended on whether or not the strength increase due to the fiber reinforcement was greater than the increased flaw generation or non-uniformity of fiber distribution which decrease concrete performance.

In measuring the workability of fiber concrete, the vebe test appeared to be a better workability indicator than the slump test except at higher workabilities. However, workability results using a mortar flow table indicated that a concrete flow table would perhaps be better than either vebe and slump tests due to the high sensitivity of the flow table in the complete range of workabilities encountered with fiber concrete. The workability results indicated that the influence of fiber length and concentration on the flow of fresh concrete was greater than the influence of small changes in in matrix design or concentration of normal workability admixtures. This lack of workability was attributed to a decrease in free paste availability and an increase in fiber interaction with the addition of fibers. The only significant improvement in workability outside the fiber factor was with the use of superplasticizers. The test results on workability indicated:

1. An increase in fiber length or concentration decreased the workability of the concrete significantly. At a water-cement ratio of 0.46 and a fiber volume of 1.35 percent, an increase in fiber length (1.27 to 2.54 cm) increased the vebe time from 3.5 to 14 sec; i.e. a decrease in slump from 35 to 10 mm. An increase in fiber volume from 0 to 1.35 percent decreased the slump from 120 to 35 mm using the 1.27 cm fiber.
2. A small increase in paste quality (i.e. decreased aggregate concentration or coarser gradation) increased the workability of plain concrete but had limited influence on the workability of fiber concrete at a fiber volume of 0.9 percent. A decrease in the water-cement ratio below 0.41 resulted in a significant decrease in workability of all concrete. Between water-cement ratios of

0.41 and 0.46, the influence of the change in paste quantity decreased with increased fiber volume; i.e. the influence of the fibers was greater than the influence of the water-cement ratio.

3. Changes in concentration of the normal workability admixtures had very little influence on the workability of fiber concrete. However, superplasticizers gave a significant increase in workability; at maximum allowable concentration, a 170 mm slump was obtained.

It is apparent from this study that the workability of concrete also critically influences the composite's mechanical properties. By increasing the strength through the addition of fibers, the workability of the composite decreases, and the potential strength increase of the composite due to the fiber addition may not be obtained. An increase in paste quantity which would increase the workability unfortunately also decreases the volumetric stability of the concrete and the bond strength as indicated by the pull-out tests. Therefore, other factors must be considered to design an optimum fiber mix interms of the concrete's workability and mechanical properties. From the test results of this study it would appear that the use of superplasticizers and short fibers with meohanical anchorages of some type (which received little attention in this study) should be investigated. Short fibers with mechanical anchorages would decrease the influence of fiber geometry on workability and would allow the build-up of adequate bond strength over the short bond development length.

Based on this study, the following topics appear to require further study or extention:

1. Although much work was done on pull-out tests, further testing is required. A more rigid testing apparatus is suggested since the compliance of the tensometer used in this study appeared to have a significant influence on the pull-out results.
2. The concrete flow table appears to show promise for the testing of fiber concrete workability. The actual sensitivity of this test with respect to the slump and vebe tests requires study. In studying the workability of fiber concrete, the use of superplasticizers and short fibers with positive anchorages (either at the end or along the fiber) should be investigated.
3. The mechanical properties of fiber composites appear to improve with the addition of fibers. Although the fatigue results do indicate that some improvement in fatigue strength can be

achieved with the inclusion of fibers, further studies are required to develop actual pavement design criteria for steel fiber reinforced concrete at stress levels used during design. Due to the difficulty in handling prisms and maintaining uniform geometry, the use of double break prisms is not recommended. The uniformity of fiber distribution is not necessarily maintained using these long specimens.

4. When the question of the consistent fatigue resistance of fiber versus plain concrete is resolved, if ever, the full economics for typical pavements should be carried out, including any extra mixing and placement costs.

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APPENDIX 1

PULL-OUT TESTS ON STEEL FIBERS IN MORTARS
(Emery, 1977)

(Developed by W.Scott for the Construction Materials Laboratory)

The prime objective of the pull-out test procedure developed by W. Scott along the lines suggested by Hughes and Fattuhi (1975), is to determine the optimum steel fiber length - the maximum length for which the fibers just pull out of the matrix (bond failure) without tensile failure of the fibers. This test is comparative and involves the following general steps:

a. casting fibers cut to a pre-determined embedment length into a plaster plug with the fibers sticking out by the desired embedment length on either side of the plug. While a 3 x 3 grid was generally used, up to 81 fibers and many spacings are possible.

b. using molds, cast the plug with fibers into the mortar matrix so that a beam with the fibers in the central zone, and oriented along the axis, is formed.

c. after curing the mortar beam for 7 days, pulling the beam in tension until failure in the fiber zone occurs (pull-out and/or tensile failure of fibers). The failure load and type of failure for each fiber is observed.

For tests reported here, long fibers from Stelco's new cut sheet fiber plant were used. While tensile strength tests indicated 276 MN/m^2 (40,000 psi) for the fibers, the production was based on cold-worked steel with a tensile strength of 380 MN/m^2 (55,000 psi). The lower tensile strength measured is probably related to stress raisers, and based on 380 MN/m^2 (55,000 psi) the tensile load for each rectangular fiber of average area 1.17 mm^2 (.000182 in^2) is about 445 N (100 lbs). However, it has been found that there is a significant variation in load from fiber to fiber. This is reflected in the pull-out test results shown in Table A1.1.

From these pull-out tests it would appear that a fiber length of 1.27cm (½") to 2.54 cm (1"), which is somewhat shorter than current practice, is optimum in terms of bond strength (embedment length of .64 cm (¼") to 1.27 cm (½"). While the variability in testing is recognized, it is clear that in the 1.27 cm (½") to 2.54 cm (1") fiber length bond failure controlled and beyond this tensile failure controlled. To better utilize bond strength, a higher tensile strength and more uniform fibers would be required. However, longer fibers do lead to workability problems and Stelco have adopted a 1.27 cm (½") fiber length for current minus .95 cm (3/8") and 1.91 (3/4") aggregate mix design trials with most promising

TABLE A1.1

PULL-OUT TEST RESULTS BY SCOTT

3 x 3 MATRIX

FIBER LENGTH FOR MIXES IS 2 x EMBEDMENT LENGTH

Embedment Length (cm)	Failure Load (N)	Average Failure Load (N)	Type of Failure Bond (%)	Type of Failure Tensile (%)
0.64	157*	405	100	0
	405		100	0
1.27	489	423	89	11
	356		89	11
	245*		100	0
	409		100	0
1.91	467	540	67	33
	540		67	33
	600		67	33
2.54	489	554	22	78
	600		22	78
3.81	445	409	0	100
	365		0	100
5.08	428	428	0	100
	428		0	100
6.35	405	405	0	100
	209*		0	100

*Not included

results. The use of shorter fibers is motivated by the interacting aspects of improved workability, increased crack control and economics for a given fiber volume in the mix.

APPENDIX 2

PULL-OUT TEST RESULTS

Group	Embedment Length (cm)	Number of Fibers Pulled	Failure Load (N)
2A	0.64	9	137.95
		9	253.65
		7	89.00
		8	178.00
		9	213.60
		5	338.20
3A	0.64	9	84.55
		5	258.10
		8	186.90
		9	133.50
		9	120.15
		8	200.25
4A	0.64	8	267.00
		6	284.80
		6	289.25
		6	275.90
		9	253.65
		6	111.25
5A	0.95	3	333.75
		5	298.15
		5	315.95
		5	267.00
		4	302.60
		5	356.00
6A	0.95	8	226.95
		8	222.50
		9	200.25
		7	280.35
		7	324.85
		7	338.20
7A	0.95	5	431.65
		7	324.85
		6	231.40
		6	307.05
		7	298.15
		3	400.50

APPENDIX 2 (Cont'd)

Group	Embedment Length (cm)	Number of Fibers Pulled	Failure Load (N)
8A	0.95	7	404.95
		9	160.20
		7	351.55
		8	351.55
		9	307.05
		6	342.65
9A	0.95	9	311.50
		7	280.35
		4	338.20
		9	62.30
		3	213.60
		8	200.25
11A	0.95	5	302.60
		0	449.45
		5	351.55
		8	226.95
12A	0.95	2	253.65
		1	320.40
		7	311.50
		9	453.90
		8	418.30
		7	378.25
13A	1.27	8	213.60
		3	418.30
		7	298.15
		5	427.20
		1	431.65
		5	342.65
14A	1.27	4	373.80
		4	396.05
		5	356.00
		5	333.75
		5	320.40
		6	356.00

APPENDIX 2 (Cont'd)

Group	Embedment Length (cm)	Number of Fibers Pulled	Failure Load (N)
2B1	0.64	8	213.60
		7	57.85
		8	209.15
		5	289.25
		8	142.40
		6	284.80
1B2	0.64	6	360.45
		8	440.55
		6	351.55
		6	396.05
		8	382.70
		9	320.40
2B2	0.64	8	284.80
		8	244.75
		8	298.15
		8	315.95
		9	302.60
		8	231.40
3B2	0.64	1	445.00
		3	445.00
		0	449.45
		2	453.90
		1	462.80
		0	458.35
[illegible]B2	0.95	0	445.00
		1	449.45
		4	413.85
		5	458.35
		0	471.70
		1	453.90
5B2	0.95	0	467.25
		1	471.70
		0	431.65
		2	445.00
		0	409.40
		1	431.65

APPENDIX 2 (Cont'd)

Group	Embedment Length (cm)	Number of Fibers Pulled	Failure Load (N)
6B2	0.95	0	453.90
		1	445.00
		1	431.65
		0	453.90
		0	440.55
		0	436.10
7B2	1.27	0	445.00
		0	445.00
		0	400.50
		0	453.90
		0	453.90
		0	453.90
8B2	1.27	0	467.25
		0	453.90
		0	427.20
		0	453.90
		0	436.10
		0	418.30
9B2	1.27	0	453.90
		0	440.55
		0	449.45
		0	431.65
		0	400.50
		0	467.25
10B2	1.59	0	445.00
		1	431.65
		1	445.00
		2	400.50
		2	382.70
1D	1.59	0	445.00
		0	445.00
		0	445.00
		0	453.90
		0	458.35
		0	458.35

APPENDIX 2 (Cont'd)

Group	Embedment Length (cm)	Number of Fibers Pulled	Failure Load (N)
1E	0.64	9	231.40
		9	240.30
		9	226.95
		0	33.45
		5	249.20
		7	178.00
		7	280.35
		7	351.55
		4	191.35
2E	0.95	9	329.30
		9	280.35
		9	445.00
		6	347.10
		1	307.05
		1	97.90
3E	1.27	9	195.80
		9	289.25
		9	--
		0	280.35
		4	244.75
		5	289.25
		1	280.35
4E	1.59	8	480.60
		9	409.40
		9	511.75
		3	489.50
		4	560.70
		0	614.10
		6	685.30
		2	582.95
5E	1.91	4	511.75
		9	311.50
		9	400.50
6E	2.22	5	333.75
		6	422.75
		6	356.00
		3	534.00

APPENDIX 2 (Cont'd)

Group	Embedment Length (cm)	Number of Fibers Pulled	Failure Load (N)
7E	2.54	2	378.25
		5	436.10
8E	2.86	9	333.75
		9	333.75
		9	418.31
F		Almost all fibers broke prematurely due to rusting	
G1 Barnes 40	0.64	9 x 6	66.75
G2 ASTM C109	0.64	9	89.00
		9	115.75
		9	83.00
G3 S/C = 1.8	0.64	9	115.75
		9	66.75
		9	124.60
G4 S/C = 2.2	0.64	9	89.00
		9	80.10
		8	253.65
G5 W/C = 0.5	0.64	9	120.15
		9	53.40
		9	53.41
H S/C = 1.33	0.95	9	31.13
		9	93.45
		9	131.01
		9	205.19
		9	182.94

APPENDIX 2 (Cont'd)

Group	Embedment Length (cm)	Number of Fibers Pulled	Failure Load (N)
1I 90 day	0.64	7	356.00
		5	364.90
		4	369.35
		5	315.95
		6	320.40
		7	409.40
		7	311.56
		9	302.60
		9	209.15
2I 45 day	0.64	4	391.60
		2	431.65
		1	471.70
		3	413.85
		0	431.65
3I* 28 day		2	449.91
		1	445.50
		0	441.41
4I* 14 day	0.64	1	350.97
		0	398.67
5I* 90 day	0.95	8	324.85
		0	436.00
6I* 45 day	0.95	5	378.25
		2	409.75
		0	422.75
7I* 90 day	1.27	0	400.50
		5	267.00
		6	298.15
8I 90 day	1.27	All fibers broke prematurely due to rusting	
9I 28 day	1.27	All fibers broke prematurely due to rusting	

*Not all specimen strengths are shown due to high degree of corrosion. Only averages are indicated.

APPENDIX 3

DETAILS CONCERNING THE LOAD-DEFLECTION BEHAVIOUR OF THE PULL-OUT TESTS

A3.1 LOAD-DEFLECTION GRAPHS

The load-deflection graphs recorded with the tensometer give the loads realistically over the full deformation range, but the deflections recorded reflect both the compliance of the tensometer and specimen deformation. Only a very small portion of the total deflection up to pull-out load is due to specimen deformation as shown in Figure A3.1. Therefore, the, apparent linear increase in deformation due to load reflects the elastic behaviour of the apparatus which "overshadows" the specimen behaviour which is not necessarily elastic. This elastic behaviour is also present during unloading and reloading of specimens in the frictional stress transfer region of the load-deflection curve. This linearity is helpful when studying the behaviour of the frictional stress build-up during pull-out load.

In Figure A3.1 it is shown that the frictional stress transfer was not monotonic, i.e., the frictional load increased and decreased alternately. This trend was observed for several specimens while the others gave a smooth load-deformation curve. This would indicate that a possible wedging of the fibers occurred due to the accumulation of

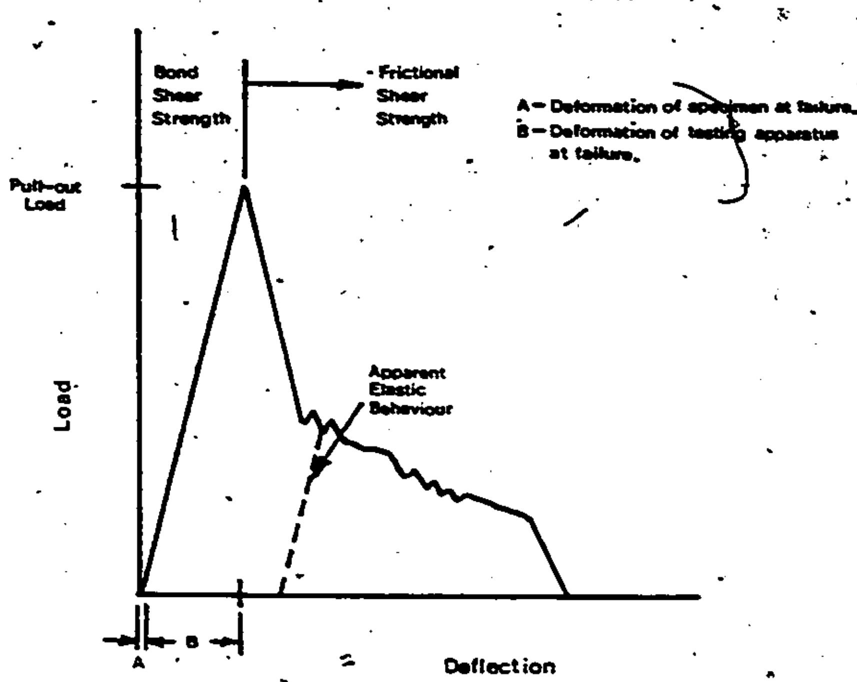


FIGURE A3.1 Load-deflection Curve for Pull-out Test

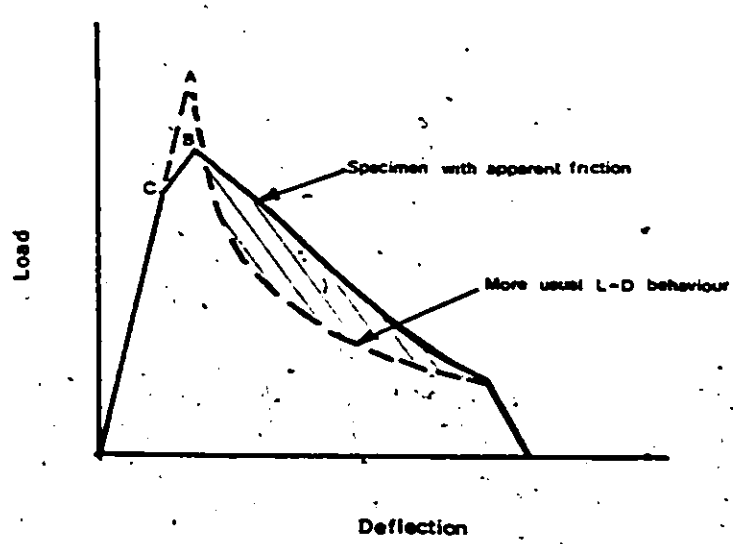


FIGURE A3.2 Effect of Apparent Frictional Build-up particles adjacent to the fibers such that the frictional load is build-up to a maximum and then quickly released when the current frictional maximum has been overcome. At each stage as the energy is released, the interface appears to undergo a change such that the same frictional level can no longer be obtained.

In specimens where unequal embedment lengths occurred, it appeared that the frictional shear stress transfer was greater than the bond shear stress at failure. This value for frictional stress transfer is an apparent value and does not reflect the actual average stress at the fiber-matrix interface. This trend also appeared when fibers were pulling through the hydrostone where additional frictional force was developed at the fiber-hydrostone interface during pulling.

The shaded area in Figure A3.2 illustrates the additional absorbed energy due to the fibers of unequal length. The peak pull-out load at A reflects the pull-out load which should have been obtained if all fibers debonded simultaneously. Instead, the peak load is reached at B. The slope C-B is less than C-A which reflects the decrease in stiffness of the specimen under the tensile stress field; therefore indicating that some fibers debonded. Thereafter, when the final fibers do debond, a more ductile behaviour is observed. This ductility is masked by the lack of stiffness of the tensometer, but is indicated by the gradual decrease in pull-out load.

A.3.2 CALCULATION OF FRICTIONAL SHEAR STRESS

The theoretical critical fiber length for an assumed true rectangular fiber can be expressed as:

$$\ell_c/a = \frac{2\sigma_{fu}}{3\bar{\tau}} \qquad \text{(A3-1)}$$

where ℓ_c and a are the critical fiber length and minimum lateral dimension respectively, and σ_{fu} and $\bar{\tau}$ are the ultimate fiber and average bond shear strengths respectively. The average bond shear stress is the parameter which is determined by the fiber pull-out test. The bond strength is found using the following formula:

$$\bar{\tau} = \frac{P_o}{S\ell} \qquad \text{(A3-2)}$$

where P_o is the fiber pull-out load, S is the fiber perimeter and ℓ is the fiber embedment length. It be should noted that $\bar{\tau}$ is assumed to be a constant. Therefore, pull-out load and embedment length are assumed to vary linearly for an ideal test series. After debonding, Equation A-2 is no longer applicable and must be modified. To develop the equation for determining the frictional stress transfer the idealized load-deformation curve recorded by the tensometer shown in Figure A3-3 is used.

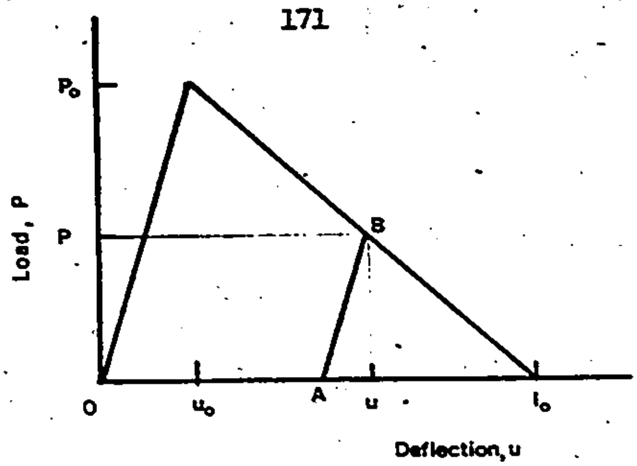


FIGURE A3.3 Idealized Load-deflection Curve

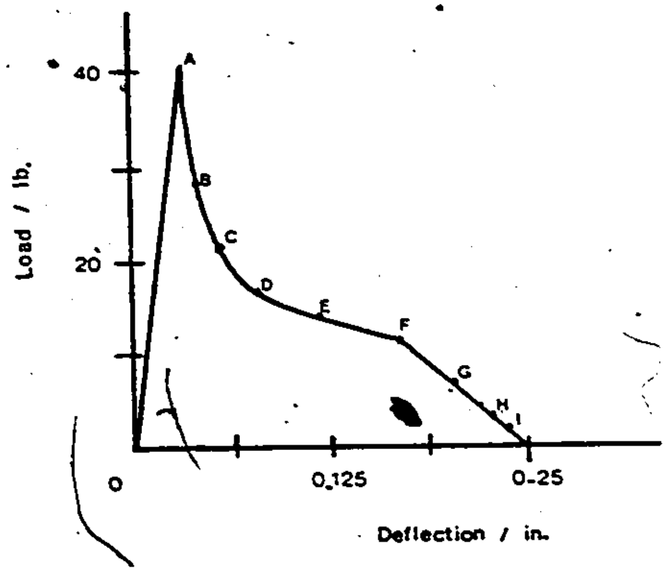


FIGURE A3.4 Load-deflection Curve for Sample Calculation of Frictional Stress Transfer

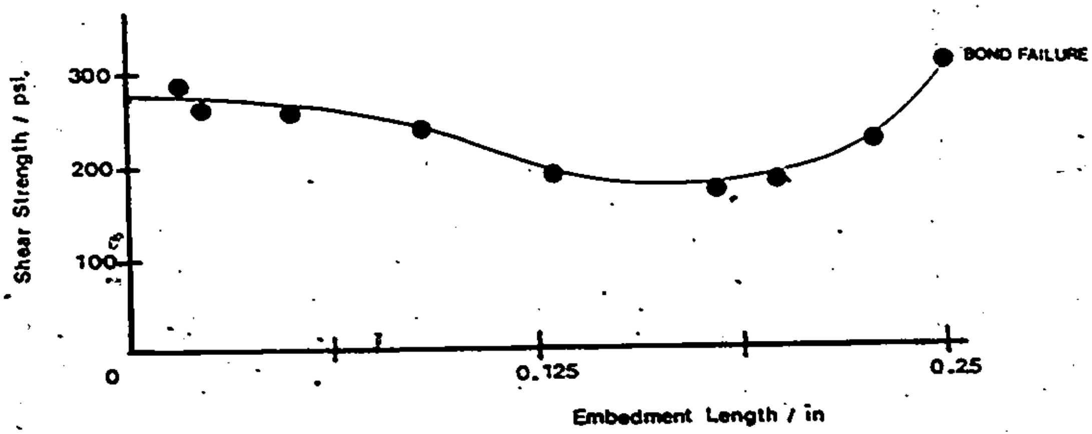


FIGURE A3.5 Shear-stress versus Embedment Length Curve Developed Using Load-deflection Curve from Figure A3.4

The deformation up to u_o due to the load increase is almost entirely due to the flexibility of the tensometer. Fortunately, the apparent linear elastic behaviour is also elastic in the frictional transfer region as indicated by A-B in Figure A3.3. To determine frictional stress after debonding, the remaining embedment length is required. This is found by taking the initial embedment length l_o and subtracting the deformation due to fiber pull-out $u - \frac{u_o P}{P_o}$, where $\frac{u_o P}{P_o}$ is the deformation due to the elastic behaviour of the tensometer. Therefore, the remaining embedment length becomes $l_o - u + \frac{u_o P}{P_o}$. This length is substituted for l_o and P for P_o in Equation A3-2 yielding:

$$\bar{\tau}_f = \frac{P}{S[l_o - u + \frac{u_o P}{P_o}]} \qquad \text{(A3-3)}$$

where τ_f is the frictional shear stress. The following sections in this appendix include calculations for the bond shear strength of Series B and a sample calculation for the frictional shear stress distribution.

A.3.3 CALCULATION OF BOND SHEAR STRENGTH

Results from Series B indicated that the critical fiber length ranged between 12.7 and 19.1 mm. To calculate the theoretical optimum length, $\bar{\tau}$ must be determined. This is done by using Equation A3-2. P_o, S and l are known simultaneously only when all fibers pull. For Series B, this occurred when l was equal to 6.4 mm, and P_O was equal to 320.4 and 302.6 N. S is constant for the a fibers, i.e. 13.716 mm. Therefore, the average $\bar{\tau}$ is: 311.5/13.716(6.4) = 3.55 N/mm^2.

A.3.4 SAMPLE CALCULATION FOR THE FRICTIONAL SHEAR STRESS

Equations A3-2 and A3-3 are used in the following computations for the frictional shear development related to the load-deflection curve shown in Figure A3.4. The calculations are shown for imperial units since the load-deformation curve obtained from the Tensometer was calibrated for inches and pound force units.

Table A3.1 shows the values required to calculate the shear stress development at each point indicated in Figure A3-4. Point A represents bond failure.

The shear transfer versus embedment length plot for Figure A3.4 is given in Figure A3.5. Both Figures A3.4 and A3.5 are also shown in Section 2.7.5, however with metric units. Figures A3.6, A3.7 and A3.8 summarize various random calculations for different curing methods.

TABLE A3.1

CALCULATIONS FOR BOND AND FRICTIONAL STRESS DEVELOPMENT

Point	Load (lb)	Deformation (in)	Elastic Deformation (in)	Embedment Length (in)	Stress Transfer (psi)
A	40	.033	.033	.250	296.30
B	28	.042	.023	.231	224.37
C	20	.058	.017	.209	177.64
D	16	.085	.013	.178	166.27
E	13.5	.125	.007	.132	188.79
F	11	.169	.009	.090	226.15
G	7	.206	.006	.050	260.43
H	3	.231	.003	.022	252.53
I	1.7	.240	.002	.012	262.35

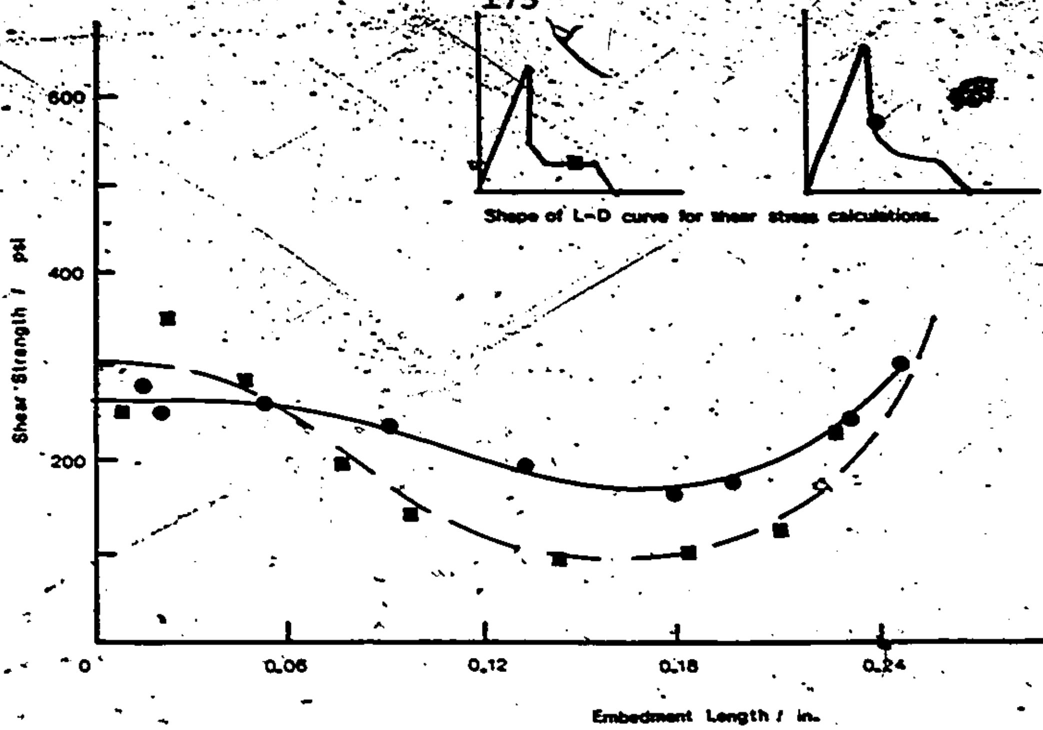


FIGURE A3.6 Relationship Between Shear Transfer and Embedment Length for Series A, Water Cured

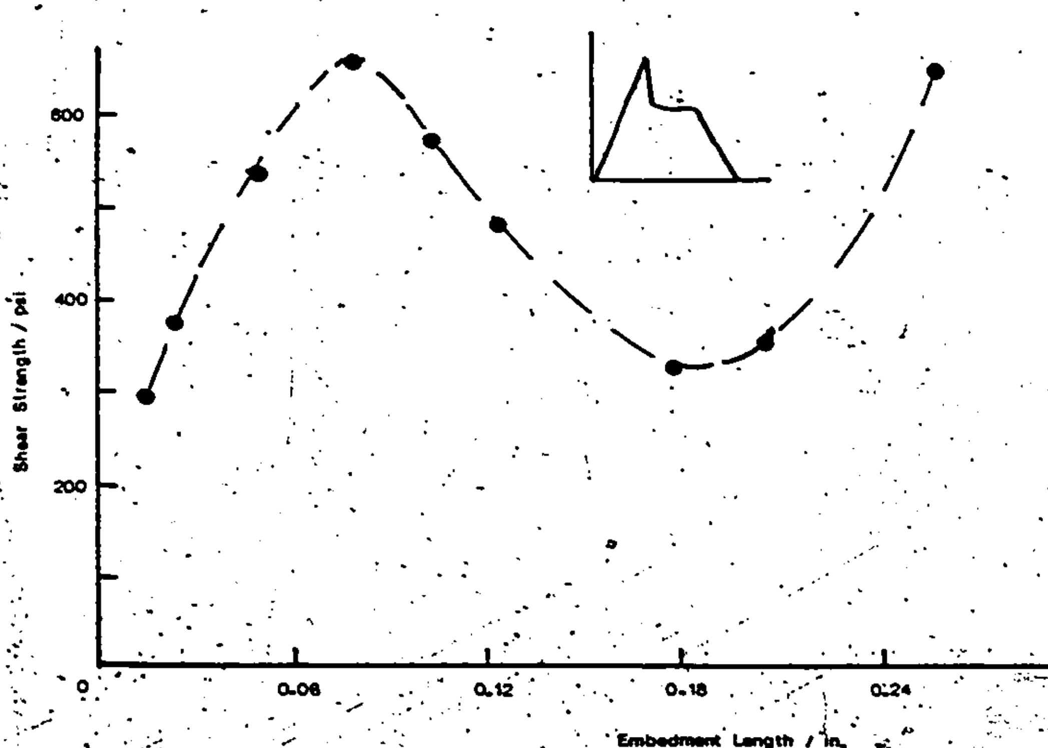


FIGURE A3.7 Relationship Between Shear Transfer and Embedment Length for Series B, Oven Cured

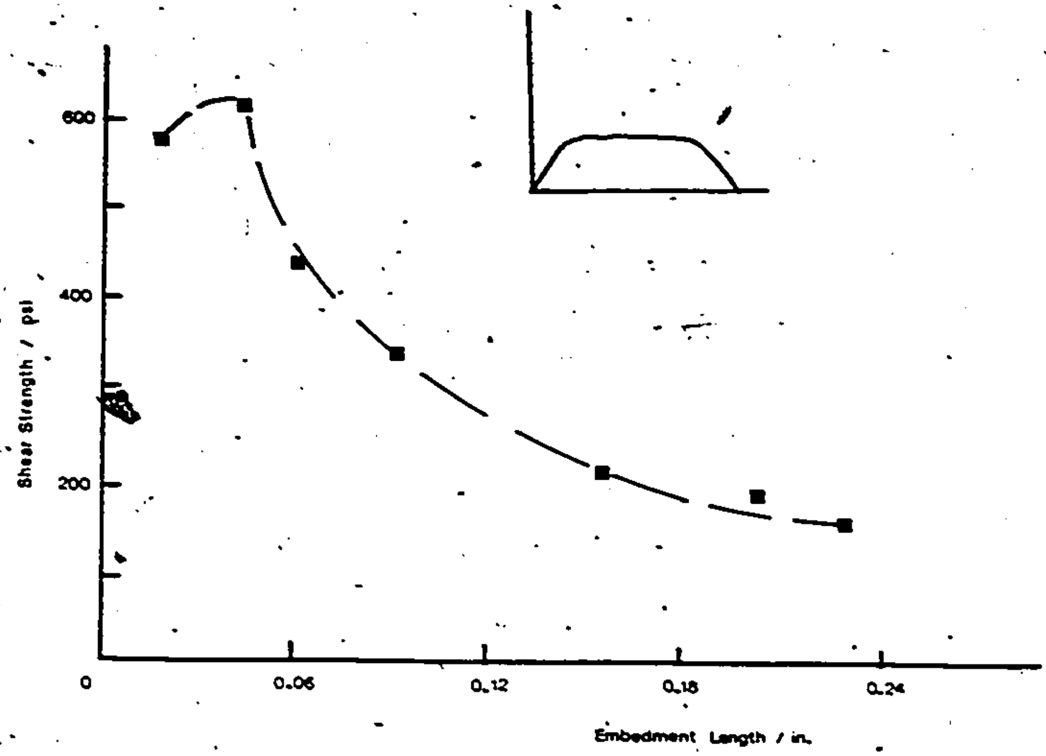


FIGURE A3.8 Relationship Between Shear Transfer and Embedment Length for Series E, Autoclaved

In Figure A3.5, the curves appears to become constant. It was indicated in Section 2.7.5 that Pinchin and Tabor [1978C] also noted similar behaviour. The following analysis shows that a constant frictional shear stress transfer is a result of a linear variation on the load-deformation plot.

It is assumed that τ_f is a constant equal to β, where with substitution into Equation A3-3:

$$P = \frac{l_o - u}{S\beta - \frac{u_o}{P_o}} \qquad \text{(A3-4)}$$

is obtained. Since l_o, S, β, u_o and P_o are constant, P varies directly with u. The scatter in Figures A3-6, A3.7 and A3.8 is attributed to inability to accurately measure P and u off the load-deformation plots. However, the linear behaviour of the load-deformation plots does reflect the constant frictional stress transfer near the end of complete fiber pull-out. It should be noted that for the autoclaved specimens the fiber-matrix stress transfer increased even after fibers started to pull. This suggests that the stress transfer was frictional throughout and indicated the influence of curing on the stress transfer loads.

APPENDIX 4

STATIC AND FATIGUE FLEXURAL STRENGTHS (80-90 DAY TESTS)

Specimen	Percent of First Crack	Static Flexural Strength (KN)	Number of Cycles to Failure
Plain Concrete	90	41.0	457
		45.2	716
		51.4	156
	85	46.5	1,458
		56.2	720
		52.0	7,473
	80	46.7	23,789
	75	45.6	92,561
		54.1	93,144
		37.6	32,035
1.27 cm Fiber Volume - 0.45%	90	43.4	829
		46.7	455
		49.5	2,544
		51.8	49
	85	48.8	48,310
		38.4	2,057
		50.2	8,891
	75	42.7	188,126
		44.0	90,192
		50.8	51,299
1.27 cm Fiber Volume - 0.90%	90	41.2	1,352
		44.6	156
		44.0	15,155
		56.2	24
	85	46.4	2,645
		38.7	55,157
		48.6	44,025
	75	48.8	158,675
		40.3	317,791
		44.6	185,682

APPENDIX 4 (Cont'd)

Specimen	Percent of First Crack	Static Flexural Strength (KN)	Number of Cycles to Failure
1.27 cm Fiber Volume - 1.35%	90	50.6	144
		46.7	561
		51.0	1,052
		54.0	96
	85	51.8	1,076
		45.3	6,154
		55.7	8,711
	75	48.5	79,732
		45.4	545,000
		51.3	1,676
2.54 cm Fiber Volume - 0.90%	90	55.2	9
		62.3	109
		68.0	71
	82.5	54.7	21
		54.3	39,064
		56.5	103
		56.3	10
		52.5	46
	75	50.6	640
		43.2	31,666
		51.0	11,142
		45.8	9,929

APPENDIX 5

COMPRESSION TEST RESULTS (15.25 x 30.50 cm CYLINDERS)

Specimen	W/C	Percent Fibers	Tested at 7 Days (KN)	Tested at 28 Days (KN)
			889.6	1,031.9
Plain	0.39	0	867.4	1,056.4
			800.6	1,010.6
			778.4	1,048.0
2.54 cm Fiber	0.39	0.45	805.1	1,034.2
			780.6	1,031.9
			925.2	1,154.7
2.54 cm Fiber	0.39	0.90	920.7	1,154.3
			974.1	1,127.6
			983.0	1,125.3
2.54 cm Fiber	0.39	1.35	951.9	1,127.6
			969.7	--
			511.5	509.3
			689.4	689.4
Plain	0.46	0	582.7	733.9
			553.8	794.0
			578.2	780.6
			364.7	785.1
			607.2	673.9
			444.8	638.3
2.54 cm Fiber	0.46	0.45	573.8	647.2
			622.7	820.7
			631.6	834.0
			549.3	829.6
			573.8	778.4
			720.6	765.1
2.54 cm Fiber	0.46	0.90	640.5	774.0
			685.0	896.3
			711.7	911.8
			722.8	858.5

APPENDIX 5 (Cont'd)

Specimen	W/C	Percent Fibers	Tested at 7 Days (KN)	Tested at 28 Days (KN)
2.54 cm Fiber	0.46	1.35	711.7 649.4 627.2 622.7 685.0 669.4	756.2 751.7 796.2 929.6 911.8 854.0
Plain	0.46	0	498.2 496.0 556.0	627.2 602.7 660.5
1.27 cm Fiber	0.46	0.45	542.7 511.5 556.0	658.3 647.2 667.2
1.27 cm Fiber	0.46	0.90	549.3 564.9 551.6	665.0 678.3 676.1
1.27 cm Fiber	0.46	1.35	489.3 507.1 --	647.2 640.5 649.4

APPENDIX 6

SPLIT CYLINDER TEST RESULTS
(15.25 x 30.50 cm Cylinder)

Specimen	Percent Fibers	First Crack Load (KN)	Ultimate Load (KN)
		182.4	
		262.4	
Plain	0	255.8	
		249.1	
		282.5	
		202.4	
		333.6	
	0.45	302.5	
		300.2	
		393.7	
1.27 cm Fiber		351.4	
	0.90	373.6	
		300.2	
		295.8	
	1.35	346.9	
		302.5	
		344.7	
	0.45	315.8	
		255.8	290.0
		201.1	244.6
		302.5	378.1
		342.5	
		333.6	
2.54 cm Fiber	0.90	309.1	400.3
		322.5	
		273.6	369.2
		286.9	403.6
			373.6
	1.35	351.4	509.3

APPENDIX 7

IMPACT RESISTANCE RESULTS

Specimen Details	Days Curing	Cylinder Position	Number of Blows First Crack	Number of Blows Ultimate	Average First Crack	Average Ultimate
Plain W/C = 0.39	30	Top	87	89	34	36
		2	9	11		
		3	25	27		
		4	3	5		
		Bottom	45	49		
2.54 cm Fiber 1.35% W/C = 0.39	30	Top	93	101	84	97
		2	37	57		
		3	17	25		
		4	42	45		
		Bottom	231	255		
Plain W/C = 0.46	30	Top	4	6	39	41
		2	73	76		
		3	47	49		
		4	67	68		
		Bottom	6	8		
Plain W/C = 0.46	30	Top	10	12	24	27
		2	20	22		
		3	20	23		
		4	56	61		
		Bottom	15	16		
2.54 cm Fiber 0.45% W/C = 0.46	30	Top	40	64	27	42
		2	19	26		
		3	14	30		
		4	14	30		
		Bottom	47	60		
2.54 cm Fiber 0.90% W/C = 0.46	30	Top	64	69	35	39
		2	11	13		
		3	20	27		
		4	11	15		
		Bottom	68	73		

APPENDIX 7 (Cont'd)

Specimen Details	Days Curing	Cylinder Position	Number of Blows First Crack	Number of Blows Ultimate	Average First Crack	Average Ultimate
2.54 cm Fiber 1.35% W/C = 0.46	30	Top	10	25	40	61
		2	29	43		
		3	70	113		
		4	44	59		
		Bottom	45	66		
Plain W/C = 0.46	60	Top	42	44	109	111
		2	183	185		
		3	124	126		
		4	108	111		
		Bottom	88	90		
2.54 cm Fiber 1.35% W/C = 0.46		Top	31	46	115	128
		2	175	188		
		3	124	135		
		4	194	210		
		Bottom	49	62		

APPENDIX 8

MIX DESIGN FOR STRENGTH AND WORKABILITY STUDIES

A.8.1 STELCO MIX DESIGN FOR 8mm (1/2") AGGREGATE

In this mix design, the fiber concentration is .9% by weight, i.e. 7.49 Kg/m^3 (120 lb/yd^3). The fiber mix is given in Table A8.1.

TABLE A8.1

STELCO "½"" AGGREGATE FIBER MIX DESIGN

Coarse Aggregate	936.94	(Kg/m^3)	1580	(lb/yd^3)
Fine Aggregate	907.28	"	1530	"
Cement	382.49	"	645	"
Water	177.90	"	300	"
Fiber	71.16	"	120	"
Admixture				
TOTAL	2475.78	"	4175	"

If a .028 cubic meter (1 ft^3) mix is desired, then the total weight required for the mix is 70.13 Kg (154.63 lb). Therefore, the constituent portions become:

Coarse Aggregate $= 70.13 \times \frac{936.94}{2475.78} = 26.54$ Kg (58.39 lb)

Fine Aggregate $= 70.13 \times \frac{907.28}{2475.78} = 25.70$ Kg (56.54 lb)

Cement $= 70.13 \times \frac{382.49}{2475.78} = 10.83$ Kg (23.84 lb)

Water $= 70.13 \times \frac{177.90}{2475.78} =$ 5.04 Kg (11.09 lb)

Steel Fiber $= 70.13 \times \frac{71.16}{2475.78} =$ 2.02 Kg (4.43 lb)

Admixture = 400 ml/100 Kg cement

$= 10.83 \times \frac{(400)}{100}$

= 43.32 ml of Porzite L-70

For the mixes in the current study, the fiber concentrations were 0, 0.45, 0.9 and 1.35 percent by volume. To account for variations in fiber content, the following relationship was used: Let P be the percentage of fibers desired, then the weight of fibers required for the mix is,

$$\frac{2.02}{.9} \times P \text{ (Kg)} \quad \text{or} \quad \frac{4.43}{.9} \times P \text{ (lb)}$$

For 1.35 percent fibers, 3.03 Kg (6.67 lb) of fiber is required for a .028 m^3 (1 ft^3) mix.

A.8.2 SUGGESTED CONTROL MIX

In designing the control mixes, all variables are held constant, with exception of the fiber content. W, C and A are the water, cement and aggregate contents respectively. For the fiber concrete mix; W/C =0.46 and A/C = 4.82. The portions of each constituent for a cubic meter (yd^3) mix is:

$$\therefore W + C + A = 2475.78 \text{ Kg (4175 lb)}$$

$$.46C + C + 4.82\ C = 2475.78$$

$C = 394.23$ Kg (665.0 lb)

$W = 181.35$ Kg (305.9 lb)

$A = 1900.2$ Kg (3204.1 lb)

Fine Aggregate (Sand) = 0.4919 (1900.2) = 934.71 Kg (1576.1 lb)

Coarse Aggregate (-½") = 0.5081 (1900.2) = 965.49 Kg (1628.0 lb)

TABLE A8.2

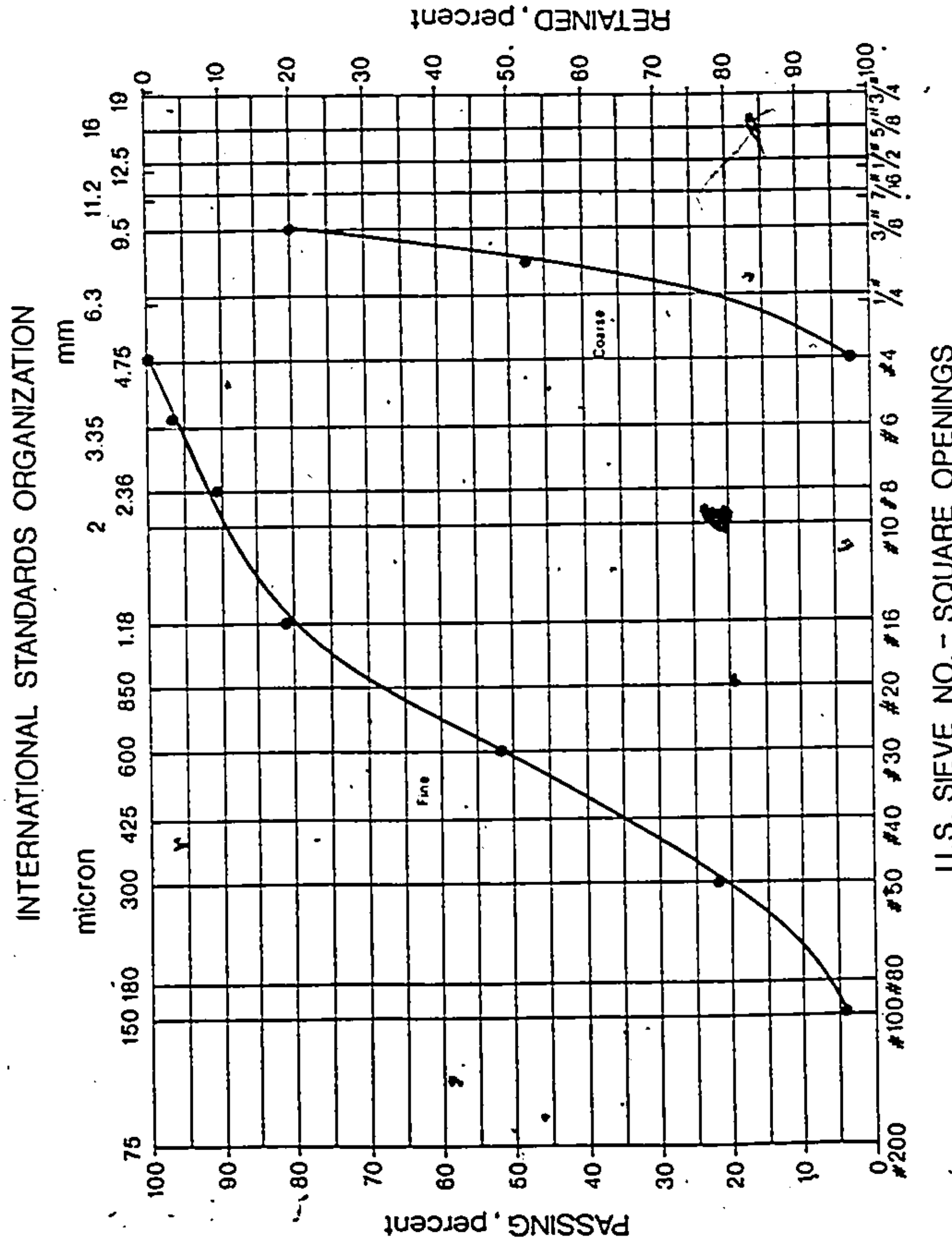
SUGGESTED CONTROL MIX

Coarse Aggregate	965.49 Kg	(1528.0 lb)
Fine Aggregate	934.71 Kg	(1578.1 lb)
Water	181.35 Kg	(305.9 lb)
Cement	394.23 Kg	(665.0 lb)
Admixture	400 ml/100 Kg cement (Porzite L-70)	

The desired quantities for a mix can be found in a similar manner as described for the fiber mix.

APPENDIX 9

AGGREGATE GRADING CURVES FOR CONCRETE MIX DESIGN



APPENDIX 10

EXAMPLE OF CALCULATIONS FOR ADJUSTING BATCH WEIGHTS FOR MOISTURE IN AGGREGATES (Emery, 1977)

It has been shown that the quality of concrete is greatly dependent upon the quality of the cement paste which itself depends on the ratio of water to cement used, and the extent of curing. Generally speaking, a higher ratio of water to cement gives less strength than a lower ratio of water to cement. During batching of concrete, it is desirable to keep the water to cement ratio constant as well as other factors such as aggregate to cement ratio and fine aggregate to coarse aggregate ratio so that constant concrete strength development and desired workability can be achieved. The ideal case is to have the aggregates in the saturated surface-dry (s.s.d.) condition which is the percent moisture at which the aggregates will neither draw mixing water from the paste nor supply additional mixing water to the paste. However, there is one practical difficulty - in most cases the aggregates in the field are not in the saturated surface dry condition; either they are a little too dry or too wet. Hence, the following example is provided to illustrate how the appropriate mixing water and other proportions are adjusted.

Example (procedure followed in the Construction Materials Laboratory)

Aggregates are stored in sealed plastic bags to achieve a constant moisture content. For the aggregates involved in this example, tests show that the saturated surface dry condition for the 3/8" aggregate is 2.5% and that for beach sand is 4%.

The ingredients for a 7.62 cm (3") slump mix in the Eirich R7 mixer for .028 cubic meter (1 cu ft) of concrete are found as follows:

BATCH PROPORTIONS AND WEIGHTS

w/c = 0.55 (water to cement)

a/c = 5.28 ÷ 0.28 cu meter = (1 cu ft) (aggregate to cement)

cement:	10.50 Kg (23.1 lb)	
water:	5.77 Kg (12.7 lb)	157.8 lbs
sand:	26.55 Kg (58.4 lb)	
aggregate:	28.91 Kg (63.6 lb)	
	71.73 Kg (157.8 lb)	

Note: The weights of both sand and aggregate are such that they are in a saturated surface dry condition.

Now, moisture content tests show the following for the aggregates being used: moisture content of coarse aggregate 3.5%; and moisture content of sand 5.0%.

The following table is then constructed:

Materials	Batch weights (aggregates in s.s.d. conditions)	Moisture content of aggregates percent above or below s.s.d.	Correction for moisture in aggregates Kg (lb)	Adjusted batch weights Kg (lb)
Cement	10.50 (23.1)	-		10.50 (23.1)
Water	5.77 (12.7)	-	-0.82(-1.81)	4.95 (10.89)
Sand	26.55 (58.4)	+2.0	+0.53(+1.17)	27.08 (59.57)
Aggregate	28.01 (63.6)	+1.0	+0.29(+0.54)	29.20 (64.24)

1. $26.55 \times \frac{2}{100}$ = .53 Kg (1.17 lb)

2. $28.9 \times \frac{1}{100}$ = .29 Kg (0.64 lb)

One may argue that the extra sand and aggregate added also have excess moisture content. But it can be seen that the amount added is very small. Hence this amount of water has practically no effect on the water to cement ratio and can be neglected.

Conclusions

The moisture content of aggregates is a prime factor affecting concrete strength development. In no circumstances can this be neglected. Whenever new stocks of aggregates arrive, tests for the saturated surface-dry moisture condition for both coarse aggregates and fine aggregates as given in ASTM C70, C127 and C128 are completed. When aggregates have moisture contents below the s.s.d. condition, the same calculations apply, except that water is added instead of reduced. Also, it is important that the aggregates have fairly uniform moisture contents, and are not too different in moisture content from the s.s.d. condition.

APPENDIX 11

EXAMPLE FOR CHECKING PERCENTAGE OF EACH MATERIAL IN MIX

WASHOUT TEST

Example (procedure followed in Construction Materials Laboratory)

A 4" cylinder of the Eirich E-2 fiber concrete mix was set aside while it was still moist. The cement and very fine particles were then washed out and the remaining 3/8" aggregate, sand and fiber left to dry. When this mixture was dry, a magnet was used to lift out the fibers and the fibers were then weighed.

From previous sieve analyses of the 3/8" aggregate and sand, it was known that almost all of the 3/8" aggregate particles were retained on the No. 4 sieve size. It was also found that most of the sand particles passed through the No. 4 sieve. Thus, the remaining mixture of 3/8" aggregate and sand from the mix can be separated on the No. 4 sieve.

Material		Weight
3/8" aggregate	(No. 4↑)	1531 g (3.37 lb)
sand	(No. 4↓)	1027 g (2.26 lb)
fibre		253.4 g (0.56 lb)

4" cylinder = 0.058 ft^3

total weight = 69.91 Kg/ft^3 x 0.058 ft^3

= 4.05 Kg (8.92 lb)

3/8" aggregate	1.531 Kg	(3.37 lb)	37.7%	
sand	1.027 Kg	(2.26 lb)	25.3%	
fibre	.253 Kg	(0.56 lb)	6.3%	— ≈6% in design mix, compares very well.
cement + water	1.24 Kg	(2.73 lb)	30.6%	
TOTAL	4.05 Kg			

In the current study, washout tests were made at random. To account for possible losses of fiber while adding it to the mix, additional fibers was added to the mix (i.e. one percent of fiber added). The washout test results for the fiber portion were in agreement with the target concentration in almost all cases.

APPENDIX 12

WORKABILITY STUDY (MIXES AND RESULTS)

TABLE A12.1

WORKABILITY STUDY MIX DETAILS

Series	W/C	A/C	C'/F	Fiber Length (in)	Fiber Volume (%)	Admixture	Admixture Dosage (% Maximum)
I-1				1	0.45		
2	0.41	4.82	1.033	1	0.90	Porzite	100
3				1	1.35	L-70	
II-1				½	0.45		
2	0.41	4.82	1.033	½	0.90	Porzite	100
3				½	1.35	L-70	
III-1				½	0.90		33
2	0.41	4.82	1.033	½	0.90	Melment	56
3				½	0.90	L-10	100
IV-1				½	0.90		38
2	0.41	4.82	1.033	½	0.90	Mulcoplast	63
3				½	0.90	CF	100
V-1				½	0.90		40
2	0.41	4.82	1.033	½	0.90	Porzite	56
				½	0.90	L-77	100
VI-1				½	0.90		40
2	0.41	4.82	1.033	½	0.90	Porzite	53
3				½	0.90	L-70	100
VII-1		4.82			0		
2	0.41	4.50	1.033	-	0	Porzite	100
3		5.00			0	L-70	
VIII-1	0.41	4.82	1.139	-	0	Porzite	-
2			.936		0	L-70	
IX-1	0.41	4.50	1.033	½	0.90	Porzite	-
		5.00		½	0.90	L-70	
X-1	0.41	4.82	1.139	½	0.90	Porzite	-
2			.936	½	0.90	L-70	

TABLE A12.2

WORKABILITY STUDY TEST RESULTS

Series	Slump (in)	Flow Table Slump (in)	Flow Table Spread (%)	Vebe Time (sec)
I-1	0.625			6.75
2	0.250			15.50
3	0.125			19.30
II-1	1.250	0.80	114	3.70
2	1,000	0.50	110	6.80
3	0.500	0.20	108	8.30
III-1	1.500	0.50	108	4.00
2	3.000	1.00	118	2.20
3	5.000	--	--	1.40
IV-1	1.000	0.80	108	5.50
2	1.750	--	116	4.00
3	7.000	--	--	0.90
V-1	0.500	0.70	106	7.25
2	0.375	0.70	106	7.50
3	0.625	0.50	105	6.00
VI-1	0.750	0.75	106	6.50
2	0.750	0.75	108	5.50
3	1.250	0.50	110	3.50
VII-1	1.750	1.125	116	2.50
2	2.250	--	--	3.00
3	0.750	--	--	7.50
VIII-1	6.500	--	--	0.90
2	0.750	--	--	4.50
IX-1	2.250	1.000	114	4.50
2	0.625	0.350	107	7.50
X-1	0.750	0.875	110	7.50
2	0.500	1.000	110	7.50