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**THE FEASIBILITY OF USING STEEL FIBER REINFORCED
CONCRETE IN RECTANGULAR CONTAINERS**

A Thesis
Presented for the
Master of Science
Degree
The University of Tennessee, Knoxville

Scott Alan Tudor
December, 1994

DEDICATION

This thesis is dedicated to several members of my family. In no specific order, I would like to thank my father, David Tudor Sr., my mother, Carol Tudor, my brother, David Tudor Jr., my wife, Lee-Ann Tudor, my wife's family, and my newly born daughter, Caroline Grace Tudor. Without their love, support, and sacrifices none of this would have been possible.

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ABSTRACT

Steel fiber reinforced concrete (SFRC) was tested to determine the feasibility of its use in the production of rectangular waste containers. Three steel fiber lengths (5/8-inch, 3/4-inch, 1-inch) were tested. It was found that the optimum fiber content was 2% by volume based on the workability of the concrete mixture. Also, it was found that the most effective steel fiber length was the 1-inch steel fiber.

Steel fibers had a widely varying effect on the compressive strength of SFRC. In some cases a decrease in compression strength occurred even with the presence of steel fibers (probably the result of reduced workability). For design purposes, no increase in compressive strength should be considered.

The contribution of steel fibers to the increase in tensile strength (modulus of rupture) varied for the three fiber lengths. A twelve percent increase in tensile strength was established as a reliable choice for all three fiber lengths. This percent increase was applied to ACI 318-89 equation for modulus of rupture as follows:

$$f_r = 7.5 \times (1.121) \times (f_c')^{0.5} = 8.4 \times (f_c')^{0.5}$$

The internal dimensions of three waste containers (#60, #96, #112) were established, and a wall thickness adequate to provide sufficient capacity without reinforcing steel was determined. The three model numbers correspond to the internal volume of each container in cubic feet. The fundamental material strengths of the SFRC were determined through compression and modulus of rupture tests at seven and twenty-eight days. From the test results, calculations were performed based on a container subjected to only the hydrostatic pressure of the waste; thus, a wall thickness of 3 1/4-inches was established as a suitable value for each container. The conclusion was drawn that, before extensive production of containers made of SFRC could be practically initiated, tests of full-scale containers should be conducted.

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CHAPTER 1

INTRODUCTION AND PURPOSE

1.1 Introduction

The Scientific Ecology Group (SEG) and ESSROC proposed to the Civil and Environmental Engineering Department, The University of Tennessee, Knoxville, the use of steel fiber reinforced concrete (SFRC) in the manufacture of waste containers. These proposed waste containers would include flyash and stainless steel fibers, the latter in sufficient quantity to possibly alleviate the need for conventional reinforcing bars. These containers would be used for the disposal of radioactive waste. Their motivation for such a proposal was cost savings due to recycling of flyash from electrostatic precipitators and contaminated steel scrap from a decommissioned nuclear power plant.

Compressive strength and modulus of rupture tests were performed at the structural testing laboratory on specimens fabricated from SFRC having 5/8-inch, 3/4-inch, and 1-inch long fibers, all having the same effective diameter (0.01-inches). These preliminary tests were performed to document the feasibility of SEG's proposal. It should be noted that, in addition to laboratory tests on beams and cylinders, more extensive, longer term research on full scale waste containers would have to be done to determine the behavior of SFRC containers subjected to different loadings before conclusions regarding the use of SFRC for waste containers could be drawn. This is due to limited research data on SFRC at twenty-eight days. A "drop" test loading of a rectangular container, on its edge, would need to be performed to verify that the SFRC would be able to withstand an impact load. Other loading conditions could be performed without a full-scale container. Such loading conditions include: (1) an empty container located beneath ground level subjected to earth pressure, (2) a full or

partially filled container located under ground, (3) an empty container above the ground surface, and (4) a filled container above ground level subjected only to the hydrostatic pressure of the waste therein. Based on engineering judgement, the fourth loading condition will result in the highest stresses; therefore, this condition is analyzed based on information from the Portland Cement Association and is discussed further in Chapter 6.

1.2 Purpose

The purpose of this thesis is to contribute to a better understanding of the behavior of steel fiber reinforced concrete in compression and flexure, especially as this understanding could be applied to the manufacturing of waste containers. To understand the concrete behavior better, results from compressive strength and modulus of rupture tests are examined and discussed. In addition to understanding the strength characteristics of SFRC, the other objectives of this thesis include a determination of the optimum content of steel fiber in SFRC for the development of compressive strength and the modulus of rupture and the determination of the most effective fiber length of the 5/8-inch, 3/4-inch, and 1-inch fibers tested.

This thesis is divided into five major sections. The first section covers a comparison of fiber reinforcements, the second a review of literature, the third UTK tests and results, the fourth a discussion of the results, and the fifth some design considerations relating to SFRC and its use in the manufacturing of rectangular waste containers.

CHAPTER 2

COMPARISON OF FIBER REINFORCEMENTS

2.1 Introduction

A fiber reinforced concrete may be defined as a concrete including water, fine and coarse aggregate, hydraulic cement, and discontinuous fibers (Mehta, 1986).

The most often occurring problem with a conventional concrete mixture is its inherently brittle type of failure mode. This type of failure occurs under tensile stress or an impact load. Conventional reinforcing bars are most commonly used to take this tensile stress but the placement of such bars can be very time consuming and expensive. There is growing interest of fiber performance in concrete due to increases in toughness and tensile properties of the basic mix (Hannant, 1978). If these properties could be increased to the point that conventional reinforcing steel could be omitted, then a very large potential savings could be realized.

The following sections discuss the application and specific use of various fiber additives of concrete. A table listing various types of fibers and their properties (f_t , E_c , ultimate elongation, and specific gravity) are presented for strength comparison.

2.2 Historical Background

"Historically fibers have been used to reinforce brittle materials since ancient times; straws were used to reinforce sunbaked bricks [Exodus 5 : 7: Egyptians used straw to reinforce mud bricks], and horse hair was used to reinforce plaster" (ACI 544.1R-82, 1986). More recently, Asbestos fibers are being used to reinforce portland cement (ACI 544.1R-82, 1986).

Fiber reinforced concrete has been discussed in many publications. One of the more recognized publication sources is the American Concrete Institute (ACI). ACI Committee 544 introduced fiber reinforcement in November of 1973 in a journal titled "State-of-the-Art Report on Fiber Reinforced Concrete", a report revised in May of 1982. The next report ACI published was the "Measurement of Properties of Fiber Reinforced Concrete" in July of 1978, later revised in December of 1983. In March of 1984, ACI 544 published the "Guide for Specifying, Mixing, Placing, and Finishing Steel Fiber Reinforced Concrete." Since then, there have been numerous publications on fiber reinforced concrete.

2.3 Discussion of Fiber Types

There are many types of fiber additives used today in concrete for the purpose of increasing strength and durability. However, only a few of them will be discussed, as the focus of this thesis is directed toward the use of steel fibers in concrete.

In general, fibers can be produced from steel, plastics, glass, and natural materials which can vary in shape and size (ACI 544.1R-82, 1986). Figure 2.3.1 illustrates many steel fiber shapes. Manufacturers claim that different shapes, compared to plain round fibers, provide extra bond to the cement-fiber matrix. However, there can be some disadvantages associated with the manufacturing process. For example, "the indentation process used to make Duoform [fiber] can weaken the [fiber] and make it more brittle, particularly with smaller diameter [fibers]" (Hannant, 1978). As noted earlier, fibers can range in sizes (cross section). Depending on the material, fiber diameters can range from 0.0002 to 0.050 inches. Round steel fibers can range in diameter of 0.01 to 0.03 inches, whereas flat steel fibers have cross-sections which range from 0.006 to 0.016 inches in thickness by 0.01 to 0.035 inches in width (Mehta, 1986).

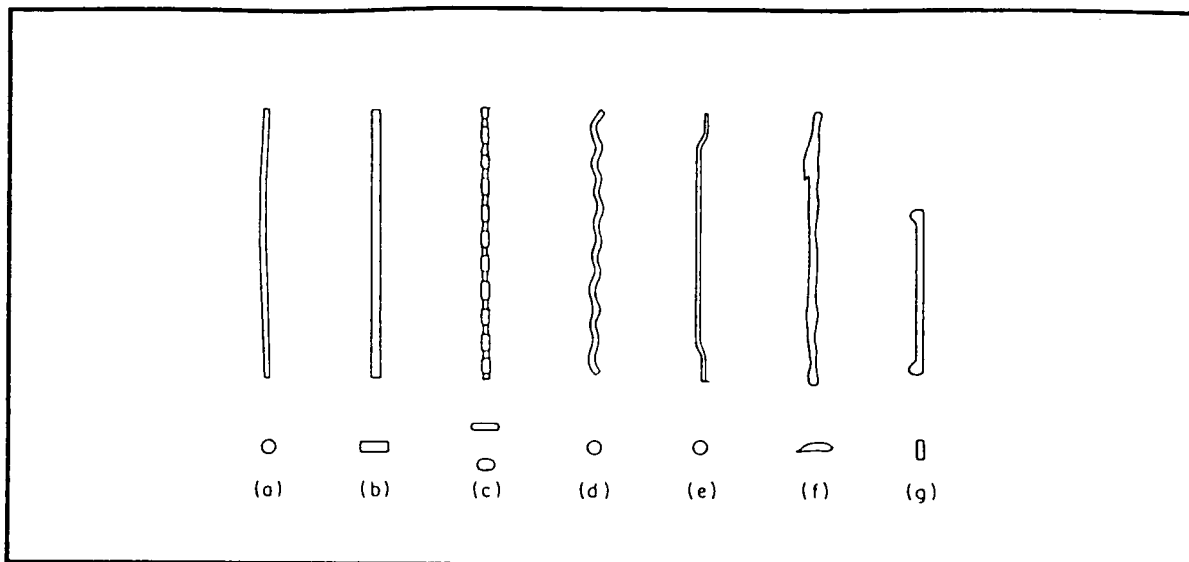


Figure 2.3.1 Shapes of steel fibers (a) Round, (b) Rectangular, (c) Indented (Duoform, National Standard Patent), (d) Crimped (G.K.N. and Johnson Nephew Ltd.), (e) Hooked ends (Dramix, Z. Bekaerto Ltd. Patent), (f) Melt extract process (Battelle Patent), (g) Enlarged ends (Australian Wire Industries Ltd. Patent). (Hannant, 1978).

Since the placement of fibers in concrete is based on volume, these thicknesses and widths are directly related to the length of the fiber and the total number of fibers added.

Fiber types can be divided into two main groups. The first group includes fiber having a modulus of elasticity lower than that of the cement matrix, such as cotton, nylon, and polypropylene. (Note that the cement matrix strength is determined primarily by w/c ratio). The second group consists of fibers having moduli higher than the cement matrix, such as asbestos, glass, and steel (Hannant, 1978).

Organic fibers having a low modulus are generally subject to relatively high creep. This means that if such fibers are used to support permanent high stresses in a cracked composite, considerable elongations or deflections may occur over a period of time (Hannant, 1978).

Another problem associated with a fiber having a low modulus is that "they generally have large values of Poisson's ratio." This means that the fibers have more tendency to deform in a direction perpendicular to the applied stress than other fibers. "This leads to a high lateral tensile stress at the fiber matrix interface which is likely to cause a short fiber to debond and pull out" (Hannant, 1978). Further discussion on variables affecting concrete properties are given in the *Review of Literature* chapter of this thesis.

A convenient way to compare various fiber additives is to represent them in a table. ACI Committee 544.1R-82 (1986) has done this in a table presented here as Table 2.3.1.

From Table 2.3.1 it is clear that steel has the highest modulus of elasticity of any of the fibers listed, a modulus much larger than a modulus from a "typical" cement matrix. For this reason, steel has some advantages as a practical choice for fiber reinforcing, as its high modulus makes it a more effective crack arrester.

2.4 Practical Applications

"Relatively speaking, the most important contribution of fiber reinforcement in concrete is not to strength but to the flexural toughness of the material. It can be seen that increasing the length of fibers up to a point increases strength as well as toughness. The increase in toughness is an order of magnitude while that in strength is only mild. Clearly when flexural strength is the main consideration, fiber reinforcement of concrete is not a substitute for conventional reinforcement" (Mehta, 1986). Therefore, fiber reinforcement is typically used in structural systems where flexure is not dominate.

Table 2.3.1 Typical Properties of Fibers

Type of Fiber	Tensile strength (ksi)	Young's modulus (10 ³ ksi)	Ultimate elongation (%)	Specific gravity
Acrylic	30-60	0.3	24-45	1.1
Asbestos	80-140	12-20	~ 0.6	3.2
Cotton	60-100	0.7	3-10	1.5
Glass	150-550	10	1.5-3.5	2.5
Nylon (high tenacity)	110-120	0.6	16-20	1.1
Polyester (high tenacity)	105-125	1.2	11-13	1.4
Polyethylene	~ 100	0.02-0.06	~ 10	0.95
Polypropylene	80-110	0.5	~ 25	0.90
Rayon (high tenacity)	60-90	1.0	10-25	1.5
Rock wool (Scandinavian)	70-110	10-17	~ 0.6	2.7
Steel	40-400	29	0.5-35	7.8

Source: ACI Committee 544, Report 544.1R-82, *Concr. Int.*, Vol. 4, No. 5 p. 11, 1982.

In 1965, "polypropylene fibers were suggested as an admixture to concrete by Goldfein for the construction of blast-resistant buildings for the U.S. Corps of Engineers" (Hannant, 1978). At that time, polypropylene twine was very inexpensive and readily available. Polypropylene has many useful properties such as a high melting point, resistance to many chemicals, and an unlikelyhood of "balling up" during mixing. However, there are several "pitfalls" to this seemingly revolutionary fiber. Hannant (1978) lists four shortcomings of polypropylene fibers: (1) combustibility, (2) a low modulus of elasticity which reduces cracking strength, (3) poor bond between fibers and cement matrix, and (4) the attack by sunlight and oxygen - ultraviolet radiation and oxidation.

Not a lot can be said about asbestos fibers except that they are known to be very durable. Hannant (1978) noted that "the impact resistance of asbestos cement is notoriously low and this may be due, in part to the stiffness of the material in the post

matrix cracking zone." Hannant (1978) also stated that the "material did not appear promising for construction purposes" because the "workability was considerably reduced with little or no gain in strength properties."

Nylon fibers did not gain much recognition because of high costs, compared to polypropylene, even though it displayed impressive strength attributes (Hannant, 1978).

Steel fibers have gained popularity due to their high strength properties. "The greatest number of applications of steel fiber reinforced concrete has been in the area of slabs, bridge decks, airport pavements, parking areas, and cavitation/erosion environments" (ACI 544.4R, 1988). "Promising results have been obtained from the use of steel fiber concrete to resist erosion and impact damage in hydraulic structures" (Hannant, 1978). Therefore, other practical examples might include spillways and sluice ways associated with dams.

Because of their high modulus of elasticity as well as strength, steel fibers would seem to hold especially high potential for use in certain reinforced concrete structures. Perhaps in some applications, such as waste containers, where flexure is not the dominant loading, these fibers may even be used to eliminate the need for steel reinforcing bars. Thus, the remainder of this thesis focuses on the use of steel fibers in concrete.

CHAPTER 3

REVIEW OF LITERATURE

3.1 Introduction

As noted in Chapter 2, steel fiber reinforced concrete is not a new idea; there are numerous publications covering many aspects of SFRC. This chapter focuses primarily on the tensile and compressive strength of SFRC, and the factors that can affect these two strengths are discussed.

3.2 Tensile Strength (Modulus of Rupture, f_r)

In the tensile strength investigation, "two flexural strength values are commonly reported. One, termed the first-crack flexural strength, corresponds to the load at which the load-deformation curve departs from linearity. The other corresponds to the maximum load achieved, commonly called the ultimate flexure strength or modulus of rupture" (ACI 544.4R, 1988). These two values are graphically represented on a stress-strain curve in Figure 3.2.1 (ACI 544.1R-82, 1986). Point A represents the "first-cracking strength", "elastic limit", or "proportional limit". Point B represents the ultimate flexural strength.

"Ultimate flexural strength generally increases in relation to the product of fiber volume concentration v and aspect ratio l/d_f " (ACI 544.4R, 1988), where " l " is the length of the fiber and " d_f " is the diameter of the fiber.

"High flexural strengths are most easily achieved in mortars. Typical values for mortars (w/c ratio = 0.45 to 0.55) are in the range of 1000 to 1500 psi for 1.5 percent by volume of fibers depending on the l/d_f [ratio] and the type of fiber" (ACI 544.4R,

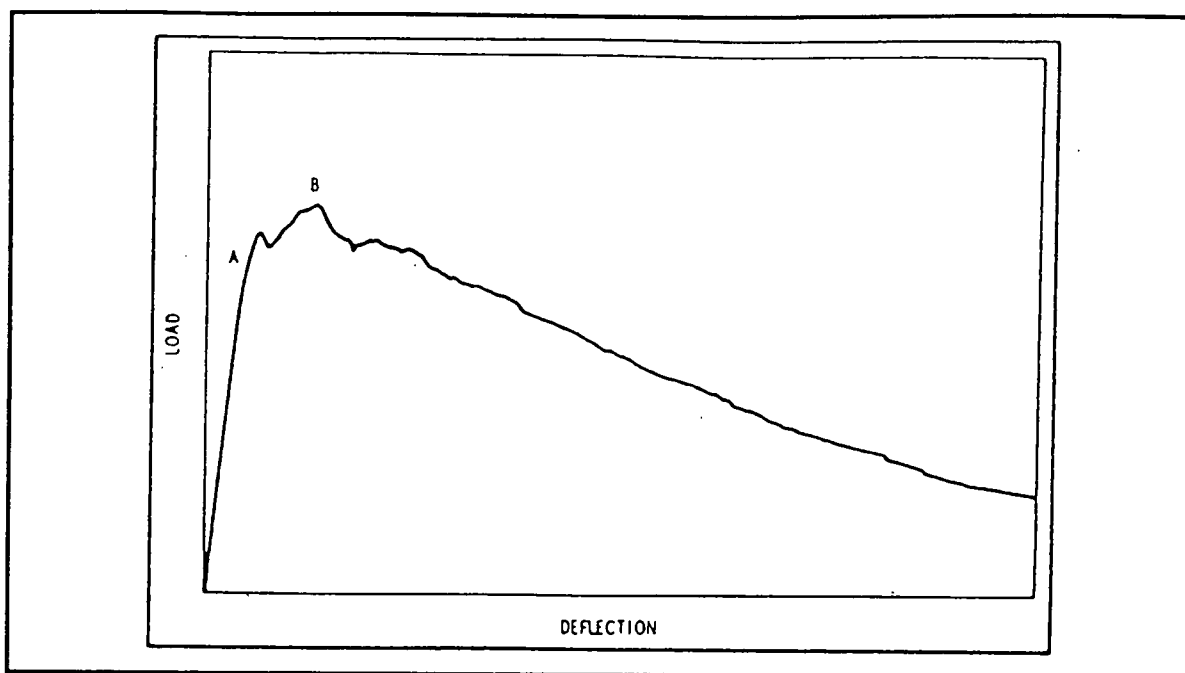


Figure 3.2.1 Important Characteristics of the P- Δ Curve For Concrete Flexure.
(ACI 544.1R-82, 1986)

1988). "The maximum load is controlled primarily by fibers gradually pulling out, and the stress in the fiber at the ultimate load is substantially less than the yield stress of the fiber. After the ultimate load, the decrease in measured load with increasing deformations is much less for fiber reinforced concrete than that for plain concrete. As a result, the total energy absorbed as measured by the area under the load-deflection curve before complete separation of a beam is at least 10-40 times higher for fiber reinforced concrete than for plain concrete" (ACI 544.1R-82, 1986). This energy is absorbed by the debonding between the concrete and steel and the elongation of the fibers. "It is generally agreed that the ultimate strength is relatively insensitive to fiber spacing and depends primarily on the volume, aspect ratio, and the bond characteristics of fibers" (ACI 544.1R-82, 1986).

The modulus of rupture can easily be calculated once the ultimate load is known. The cross-section of the beam is known; therefore, the moment of inertia can be calculated. Shown below is the formula used to determine the modulus of rupture for a 3" x 4" x 16" beam having a 12" span loaded at the third points:

$$f_r = \frac{Mc}{I}$$

where, $M = \text{applied moment in kip-in} = 0.5(\text{ultimate load}) \times 4 "$
 $= 2(\text{ultimate load})$

$c = \text{distance from neutral axis to extreme fiber} = 2 "$

$I = \text{moment of inertia} = bh^3/12 = 16 \text{ in}^4$

After appropriate substitution, the modulus of rupture reduces to the following equation:

$$f_r = 0.25(\text{ultimate load}) , \text{ where } f_r \text{ is in psi or ksi}$$

3.3 Compressive Strength (f_c')

The compressive strength is generally determined by testing 6" by 12" concrete cylinders and is denoted f_c' . The compressive strength of concrete without the addition of fibers is relatively high and failure occurs when the concrete "crushes". Since the concrete fails abruptly, the addition of steel fibers does not increase the ultimate compressive strength. However, the steel fibers do "hold" the concrete in place; thus, eliminating the scatter of concrete during failure. "Test results show the addition of [steel] fibers has [only] minor effects on the improvement of the compressive strength values" (Wafa & Ashour, 1992).

Toughness is a measure of the ability to absorb energy during deformation. This property can be estimated from the area under the stress-strain curves or load-deformation curves. Based on Figures 3.3.1 and 3.3.2, the following observation can be made: "A substantial increase in the strain at the peak stress can be noted, and the slope of the descending portion is less steep than that of control specimens without fibers. This is indicative of substantially higher toughness. The improved toughness in compression imparted by fibers is useful in preventing sudden and explosive failure under static loading" (ACI 544.4R, 1988). In other words, the presence of steel fibers alters the failure mode of concrete cylinders from a brittle to a more ductile failure.

3.4 Variables That Affect f_r and f_c'

This section discusses several variables influencing the mechanical properties of steel fiber reinforced concrete. According to ACI 544.4R (1988), "[steel] fibers influence the mechanical properties of concrete and mortar in all failure modes."

To discuss intelligently how steel fibers affect the strength properties of the composite material, a basic knowledge of the relationship between the steel and concrete must be known. In brief, the "strengthening mechanism of the fibers involves transfer of stress from the matrix to the fiber by interfacial shear, or by interlock between the fiber and matrix if the fiber surface is deformed" (ACI 544.4R, 1988). ACI 544.4R (1988) adds that the tensile stress is shared by both the fiber and the matrix until cracking occurs. Then the total stress is completely transferred to the steel fibers. This transfer of stress adds ductility to a system that is very weak (in tension) and brittle by nature.

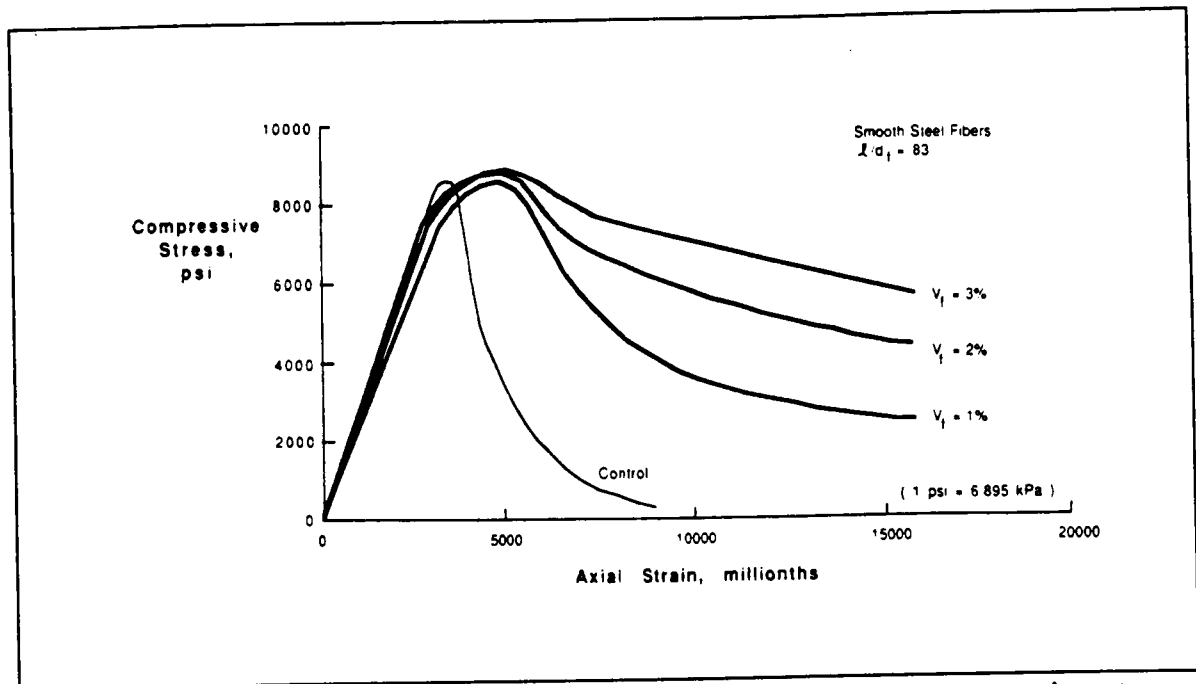


Figure 3.3.1 Influence of the volume fraction of fibers on the compressive stress-strain curve (ACI 544.4R, 1988).

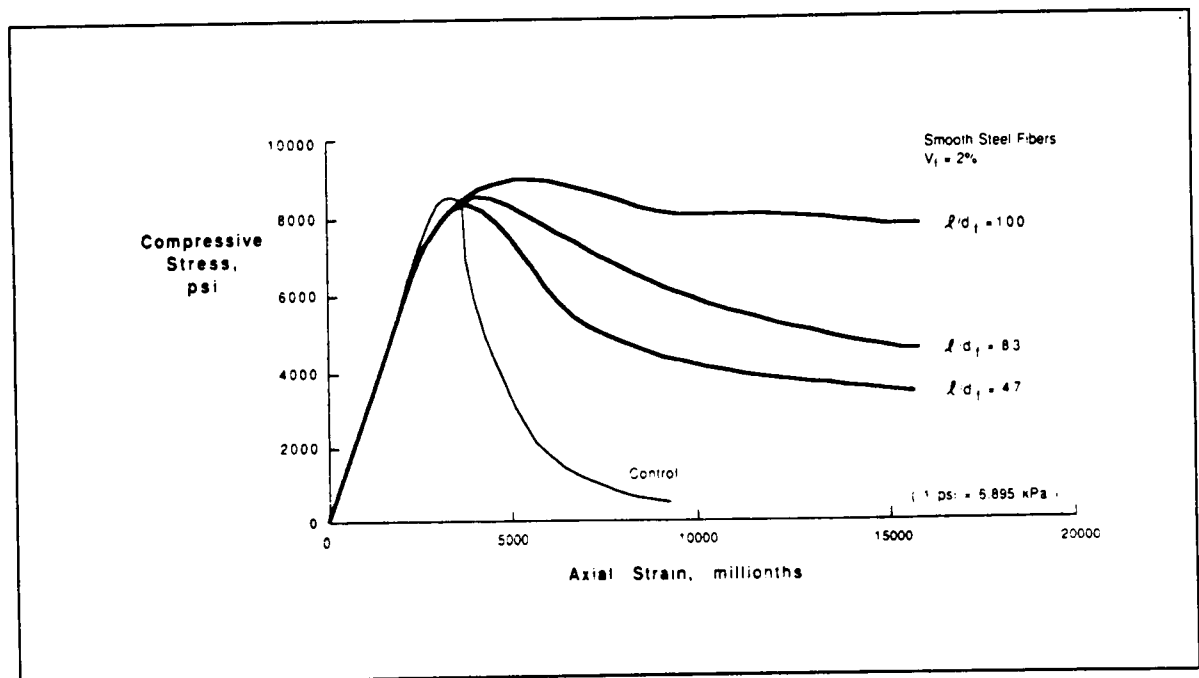


Figure 3.3.2 Influence of the aspect ratio of fibers on the stress-strain curve (ACI 544.4R, 1988).

There are many variables that govern the properties of steel fiber reinforced concrete other than the cement matrix, such as: (1) aspect ratio, (2) length of the fiber, (3) percentage of fiber by volume, and (4) orientation (aligned, versus randomly dispersed). "Aside from the matrix itself, the most important variables governing the properties of steel fiber reinforced concrete are the fiber efficiency [resistance to fiber pullout] and the fiber content [percentage of fiber by volume]" (ACI 544.4R, 1988). The resistance to fiber pullout (fiber efficiency) is based on the bond strength at the fiber-matrix interface. This bond strength is directly related to the length of the steel fibers. For uniform cross sections, the fiber efficiency increases with an increase in fiber length. Therefore, "the longer the [steel] fiber the greater its effect in improving the properties of the [SFRC]" (ACI 544.4R, 1988).

3.4.1 Aspect Ratio

The aspect ratio can be defined as the ratio relating the length of the steel fiber to the equivalent diameter of the fiber. Since not all steel fibers are circular in cross section, the equivalent fiber diameter is "a circular section with the same area" (ACI 544.4R, 1988).

As noted in the introduction, fiber efficiency increases with an increase in fiber length. In addition to fiber length, the pullout resistance is also proportional to the contact surface area between the steel fiber and the concrete. "Therefore, for a given fiber length, a high [aspect ratio] is associated with high fiber efficiency" (ACI 544.4R, 1988). Unfortunately, when a steel fiber exhibits a high aspect ratio, the workability of the concrete mixture is severely diminished. According to Mehta (1986), "typical aspect ratios range from about 30 to 150." ACI 544.4R (1988) is a little more conservative, noting a maximum range value of 100.

3.4.2 Length of the Fiber

The effects of increasing the length of the steel fibers were briefly discussed in the aspect ratio section. It was noted that an increase in fiber length increases the bond strength and the aspect ratio.

Shah and Rangan (1971) pointed out that increasing the length of the fibers up to a certain point will increase the strength as well as the toughness of the concrete. However, while the strength increase is only minor, the toughness increases by an order of magnitude. It should be noted that the above statement is based on SFRC containing a percent volume of 1% and having dimensions of 0.01" by 0.01" by 3/4".

3.4.3 Percentage of Fiber by Volume

The percentage of steel fiber added to a concrete mixture is the second most important variable influencing the ultimate load of SFRC (the first was the aspect ratio). This percentage is the percentage of steel fiber added to a concrete mixture based on the total volume of the mixture.

The key factor affecting the percentage of steel fiber which can reasonably be added to concrete is workability. As soon as any steel fiber is added, the ability to mix, pour, and finish concrete is hindered due to an increase in viscosity. For this reason, steel percentages remain relatively low ($\approx 2\%$).

Fortunately, chemicals can be added to fresh SFRC to make the mixture more workable. Such chemicals, within certain concentrations, do not hinder or help the strength properties of the concrete. The most important chemical additive from the viewpoint of workability is superplasticizer. According to Rixom (1978), superplasticizers "allow a reduction in the water-cement ratio at a given workability without significantly affecting the setting characteristics of the concrete." For a base mix, superplasticizer is generally not added. On the other hand, superplasticizer is

used in SFRC, even at low concentrations (percentage) of steel fiber. Without the addition of the superplasticizer to SFRC, the concrete becomes very "thick" and practically impossible to mix and pour.

There have been many laboratory tests (including the ones noted in this thesis) on the optimum steel fiber content. Most agree that a percentage of steel fiber greater than two percent (including an admixture) produces a non-workable concrete mixture.

3.4.4 Orientation of Steel Fibers

The orientation of steel fibers relative to the plane of the crack in the concrete mixture is very important to stress distribution. A fiber which happens to be parallel to the crack has no effect on the strength, while a fiber which is perpendicular to the crack has a maximum effect. The ideal situation would be to have all the steel fibers in the direction of the applied stress, but this is not likely.

The placement of steel fibers are either aligned or are randomly oriented. Steel fibers can be aligned "by applying a magnetic field and vibrating the specimen or member while it is still plastic" (ACI 544.1R-82, 1986). Although this sounds great, the practicality of this procedure is not (i.e. there is no way to check to see if the fibers are aligned). On the other hand, randomly dispersed fibers are added to the concrete mixture "by hand". According to ACI 544.1R-82 (1986), the magnetically aligned fibers yield a "pronounced increase in flexural strength over randomly oriented fibers." However, in practical applications of steel fiber reinforced concrete, randomly oriented fibers are almost inevitable.

It should be noted that concentrating steel fibers in regions of maximum stress is more efficient and cost effective than fibers which are randomly dispersed uniformly throughout the structural element. A good example of this would be to place steel

fibers only in the middle/bottom portion of a simply supported beam where the tensile stress is the greatest.

3.5 Steel Fiber Shotcrete

Shotcrete is increasingly being used for the repair and rehabilitation of old deteriorated structures as well as for new construction. Due to the interest in shotcrete, this subject has been the focus of many current investigations (Banthia, et al, 1994).

Shotcrete is generally classified by one of two mix processes: dry-mix or wet-mix. The dry-mix process may be defined as the premixing of all ingredients, except water, prior to application. The water is added to the dry mixture at the nozzle of the hose. Some of the disadvantages to this mix process include the presence of dust and excessively high shotcrete rebound. The other mix process, wet-mix, may be simply defined as the premixing of all ingredients (including water) prior to the application. Such a process "can lead to a cleaner working environment (less dust), lower material rebound, and a better control on the water-cement ratio of the mix" (Banthia, et al, 1994).

Steel fibers are increasingly being used in shotcrete for both mix processes (dry & wet). The steel fibers "eliminate the need for conventional reinforcing steel, welded wire mesh, or chainlink mesh [in areas of low tensile stress], and this alone results in a significant ease of placement" (Banthia, et al, 1994). Also, the presence of steel fibers adds toughness to the hardened shotcrete along with improved deformability characteristics and impact resistance.

CHAPTER 4

UTK TEST RESULTS

4.1 Introduction

Compressive strength and modulus of rupture tests were performed on steel fiber reinforced concrete (SFRC) beams and cylinders. The Scientific Ecology Group (SEG) and ESSROC sponsored the tests that were performed at the structural testing laboratory of the Civil and Environmental Engineering Department at The University of Tennessee, Knoxville. These tests were performed on specimens fabricated from SFRC having 5/8-inch, 3/4-inch, and 1-inch long stainless steel fibers with all having an effective diameter of 0.01-inches. The aspect ratio for the 5/8", 3/4", and 1" fibers were 62.5, 75, and 100 respectively, and the cross-sectional area was roughly rectangular having an approximate width of 0.039-inches and an approximate thickness of 0.014-inches. SEG proposed to manufacture waste containers made of concrete containing flyash and stainless steel fibers, a proposal which would be particularly attractive if the tests showed that the presence of steel fibers would be sufficient to remove the need for conventional reinforcing bars. This chapter contains the results of the compressive strength and modulus of rupture tests.

4.2 Test Specimens

As noted before, the two tests performed on the SFRC were compressive strength and modulus of rupture (tensile strength). The compressive strength tests utilized the standard 6"-diameter by 12"-long cylinders cast from a plastic "lipped" mold. This lip was around the top of the cylinder mold providing stiffness for transporting. The modulus of rupture test utilized standard 3" by 4" by 16"-long beams cast in a metal mold. These beams were simply supported 2" from each beam

end, thus having a clear span of 12". Test specimens were cast in accordance with ASTM C39 and ASTM C78 and were moist cured in accordance with ASTM C511.

4.3 The Test Program

4.3.1 Concrete

The base concrete mix used in the test specimens contained the following ingredients: (1) Dixie Type I cement, (2) River Sand (ASTM 33), (3) Limestone coarse aggregate (crushed stone), (4) water, (5) flyash, and (6) superplasticizer. ESSROC provided a typical 5,000 psi design mix with flyash as a starting point. This mix was refined, verified, and documented at The University of Tennessee (UTK). The standard mix proportion used in the research was as shown in Table 4.3.1.

Table 4.3.1 Standard Mix Proportion

Material	Weight (lb./yd ³)	Volume (ft ³)
Cement	614	2.81
Flyash	102	0.68
Gravel	1,841	10.93
Sand	1,176	7.11
Water	341	5.47
Superplasticizer	6 ml / 100 lb.	----
	4074	27.00

4.3.2 Specimen Fabrication

An important consideration in the mixing of the concrete is workability. Even with the base mixture (no fibers) some form of chemical additive is needed; it was determined that the 5000 psi design mix was too "thick" to work even without the presence of steel fibers. For the tests performed at The University of Tennessee, the additive chosen for use was superplasticizer. As previously mentioned in Chapter 3, a

superplasticizer is added to a concrete mixture to increase the workability without hindering the strength properties. In the laboratory, superplasticizer was used in every pour and it varied in concentration depending on desired consistency. The maximum amount of superplasticizer added was 1000 ml. Besides the noticeable enhancement of the ease of placement, another benefit noticed from the presence of superplasticizer was the ease of finishing the concrete. A clean and level surface on the test specimens was accomplished.

After the base concrete mix had been verified and documented, compressive strength and modulus of rupture tests were performed to determine the influence of varying fiber contents. The ingredients in the standard mix with no steel fibers were mixed in a 7 cubic foot mixer (see Appendix A for photo).

To overcome the problem of variation between castings, the following procedure was followed for each set of tests with a particular percentage of steel fiber.

(1) The standard (base) mix with no steel fibers was placed in the mixer and thoroughly mixed. A slump test was then made.

(2) Superplasticizer was added and a second slump test was performed.

(3) Six cylinders and six beams were cast using the base mixture.

(4) Steel fibers were added and thoroughly mixed. Another slump test was performed.

(5) Additional superplasticizer was added when needed.

(6) Six SFRC cylinders and six SFRC beams were cast.

(7) At seven and twenty eight days three cylinders and three beams from the base mix and the SFRC mix were tested to determine compressive and tensile strengths. (It should be noted that in some cases only enough cylinders for seven day tests were cast.)

More specifically, the slump procedure included the "rodding" of the concrete in a truncated cone at one-third layers based on volume. Each layer was "rodded" twenty-five times. The slump of the concrete after the addition of superplasticizer and before the addition of steel fibers ranged between 7 and 10¹/₈" inches depending on the concentration of superplasticizer added (500 ml to 1000 ml). These slump values are typical for a 5000 psi base mix.

The addition of steel fibers to the base mix resulted in noticeable effects on slump and workability, even at low concentrations (1%). The range in slump of the SFRC, after the addition of superplasticizer, was 1/4" to 5¹/₂" depending on the percent steel fiber content and the amount of additional superplasticizer added to the mixture. Table 4.3.2 represents a partial listing of slump values for both the base mixture and the SFRC mixture. This table is not conclusive and should only be used for illustration purposes and not as a guide.

For each of the varying fiber contents (1% to 2.5%), three cylinders and three beams were cast for seven-day tests, and were compared with the base concrete mixture having the same number of test specimens. The twenty-eight day tests were not as comprehensive due to the lack of contract length. However, two steel fiber contents were tested for each fiber length (i.e. 5/8"- 2% and 2.5%; 3/4"- 1.5% and 2%; 1"- 1.5% and 2%).

4.3.3 Testing Procedure

For the compressive strength tests, a 350,000 pound capacity Forney testing machine was used. These tests were done in strict accordance with the specifications of ASTM C39. For the modulus of rupture (bending strength) tests, a Tinius Olsen testing machine was used. ASTM C78 was followed for these tests including third

Table 4.3.2 Various Slump Values for Given Fiber Lengths and Concentrations of Superplasticizer

BASE MIXTURE			
% Steel Fiber	Steel Fiber Length (inches)	Slump (inches)	Amount of Superplasticizer
0%	5/8"	7"	500 ml
0%	5/8"	10"	1000 ml
0%	3/4"	9"	600 ml
0%	3/4"	10"	600 ml
0%	1"	8 ¹ / ₂ "	700 ml
0%	1"	10"	700 ml
SFRC MIXTURE			
% Steel Fiber	Steel Fiber Length (inches)	Slump (inches)	Amount of Superplasticizer
1.91%	5/8"	3/4"	200 ml
2.0%	5/8"	1/8"	100 ml
2.0%	3/4"	6"	200 ml
1.5%	3/4"	1/2"	0
1.5%	1"	2 ³ / ₈ "	0
2.0%	1"	7/8"	0

point loading on the beam specimens. Appendix A of this thesis has pictures of both testing machines.

ASTM requires that at least three specimens be tested for each case. These tests were consistent with ASTM's guidelines in that three specimens were fabricated and tested for the base mix and three specimens for each SFRC mixes. Test results are shown in Tables 4.3.3 through 4.3.8. In each Table the letter "V" appears with a number attached (e.g., V₂). The attached number corresponds to the mix number assigned to each mix and is used only once. The project started with V₂ and ended with V₁₃, thus having a total of 12 mixes. Mix V₁ was the first mix done, and it was used to verify ESSROC's 5000 psi mix proportion. Each value in the table of results represents an average of three test specimens.

Table 4.3.3 Test Results for SFRC with 5/8" fiber at 7 days

Mix Designation	Percent of Steel Fiber by Volume	Compressive Strength (f_c' - psi)	Percent Increase	Modulus of Rupture (f_r - psi)	Percent Increase
V ₂	0	4739	----	742	----
V ₃	0	5494	----	769	----
V ₄	0	5176	----	820	----
V ₅	0	5459	----	808	----
V ₆	0	5600	----	875	----
V ₂	1.18	4880	3.0	796	7.3
V ₃	1.22	6083	10.7	888	15.4
V ₆	1.0	5730	2.3	1058	20.9
V ₃	1.91	6997	27.4	1018	32.3
V ₄	1.93	5907	14.1	1101	34.2
V ₆	1.5	6343	13.3	1064	21.6
V ₂	2.41	5971	26.0	913	23.0
V ₃	2.79	7351	33.8	1178	53.2
V ₅	2.58	6496	19.0	1384	71.3
V ₆	2.0	7298	30.3	1148	31.1
V ₂	3.17	6201	30.9	1093	47.3

Table 4.3.4 Test Results for SFRC with 5/8" fiber at 28 days

Mix Designation	Percent of Steel Fiber by Volume	Compressive Strength (f_c' - psi)	Percent Increase	Modulus of Rupture (f_r - psi)	Percent Increase
V ₄	0	7120	-----	1158	-----
V ₅	0	7474	-----	948	-----
V ₄	1.93	7663	7.62	1298	12.1
V ₅	2.58	8771	17.34	1550	63.5

Table 4.3.5 Test Results for SFRC with 3/4" fiber at 7 days

Mix Designation	Percent of Steel Fiber by Volume	Compressive Strength (f_c' - psi)	Percent Increase	Modulus of Rupture (f_r - psi)	Percent Increase
V ₁₁	0	4115	-----	692	-----
V ₁₂	0	4445	-----	780	-----
V ₁₃	0	4327	-----	748	-----
V ₁₁	1.0	4268	3.7	762	10.1
V ₁₁	1.5	4209	2.3	834	20.5
V ₁₂	1.5	4527	1.8	871	11.7
V ₁₁	2.0	4621	12.3	920	33.0
V ₁₃	2.0	5388	24.5	940	25.7
V ₁₁	2.5	5069	23.2	898	29.8

Table 4.3.6 Test Results for SFRC with 3/4" fiber at 28 days

Mix Designation	Percent of Steel Fiber by Volume	Compressive Strength (f_c' - psi)	Percent Increase	Modulus of Rupture (f_r - psi)	Percent Increase
V ₁₂	0	5706	-----	1026	-----
V ₁₃	0	6095	-----	904	-----
V ₁₂	1.5	5765	1.0	1172	14.2
V ₁₃	2.0	7569	24.2	1173	29.8

Table 4.3.7 Test Results for SFRC with 1" fiber at 7 days

Mix Designation	Percent of Steel Fiber by Volume	Compressive Strength (f_c' - psi)	Percent Increase	Modulus of Rupture (f_r - psi)	Percent Increase
V ₇	0	5706	-----	893	-----
V ₈	0	5541	-----	823	-----
V ₉	0	4680	-----	814	-----
V ₁₀	0	5187	-----	788	-----
V ₇	1.0	5482	-3.9	938	5.0
V ₇	1.5	6838	19.8	1228	37.4
V ₈	1.5	5954	7.5	923	12.2
V ₁₀	1.5	5612	8.2	903	14.6
V ₇	2.0	6685	17.2	1132	26.8
V ₉	2.0	4881	4.3	1128	38.6
V ₁₀	2.0	5954	14.8	1234	56.8

Table 4.3.8 Test Results for SFRC with 1" fiber at 28 days

Mix Designation	Percent of Steel Fiber by Volume	Compressive Strength (f_c' - psi)	Percent Increase	Modulus of Rupture (f_r - psi)	Percent Increase
V ₈	0	7097	-----	1193	-----
V ₉	0	5789	-----	913	-----
V ₈	1.5	7203	1.5	1250	4.8
V ₉	2.0	5741	-0.83	1136	24.4

CHAPTER 5

DISCUSSION OF RESULTS

5.1 Introduction

Three types of steel fibers were tested at different percent volumes (1 to 2.5%) to determine the optimum content and efficiency of steel fiber in SFRC for the development of compressive strength and modulus of rupture (tensile strength). These three steel fiber types were distinguished by their lengths: 5/8-inch, 3/4-inch, and 1-inch. The effective diameter of each fiber length was 0.01-inches; thus, the aspect ratios for the 5/8", 3/4", and 1" fibers were 62.5, 75, and 100, respectively.

5.2 Comparison of UTK Tests and Others

The University of Tennessee, Knoxville (UTK) results are given in Chapter 4 of this thesis. The first noticeable effect of the presence of steel fibers on the base concrete mix was workability. Even at a steel fiber content as low as one percent, the workability of the SFRC was affected. Fortunately the addition of superplasticizer made the mixture more workable. However, there was a point, for all fibers, where the addition of more superplasticizer was inconsequential with regard to workability and slump. This point occurred when the steel fiber content reached and exceeded two percent by volume.

Theoretically, the fiber efficiency for the 1-inch steel fiber is the highest due to its high aspect ratio and the lowest for the 5/8-inch steel fiber. However, as can be seen from the results, the fiber with the largest aspect ratio did not have the largest increase in flexural strength. For example, for the seven-day test on one percent fiber content the 5/8-inch fiber (having the lowest aspect ratio) had the largest increase in

tensile strength whereas the 1-inch fiber (having the largest aspect ratio) came in last on individual tensile strength increase. However, for 1.5% fiber content at seven-days, the 1-inch fiber was tied with the 5/8-inch for the lead in strength increase, and the 1-inch fiber came in first for a fiber content of 2%. This discrepancy in the aspect ratio theory may be due to the low flexural strength characteristics of a concrete at an early age.

The results of the twenty-eight day tests are somewhat more consistent with the idea that high aspect ratios are more effective. The 1-inch fiber came in at a close second place behind the 3/4-inch fiber, and the 5/8-inch fiber found its way to last place in reference to percent increase in tensile strength. It is appropriate to state at this point that the twenty-eight day tests were not completed. Perhaps if more tests were performed at this age, the 1-inch fiber would yield the highest tensile strength.

One of the objectives of the research was to determine the optimum fiber content. To compare the results of the tests with different fiber lengths, bar charts were developed and are shown in Figures 5.2.1 through 5.2.6. Each figure shows the percent increase in strength (compressive and tensile) for a given fiber length and for several percent steel fiber contents. There are two figures for each steel fiber length: one for the seven day strength and the other for the twenty-eight day strength, thus a total of six figures. These figures graphically depict the range in increasing strength for each fiber length.

There were one to three concrete pours for each fiber content at seven days. Each pour included an average of three cylinder tests and three beam tests on both the base mix and SFRC mix for compression and modulus of rupture. However, there was only one pour for steel fiber contents at twenty-eight days. The same number of beams and cylinders for the base mix and SFRC mix were poured at twenty-eight days. For both the seven and twenty-eight day strengths, the scatter from one cylinder or one

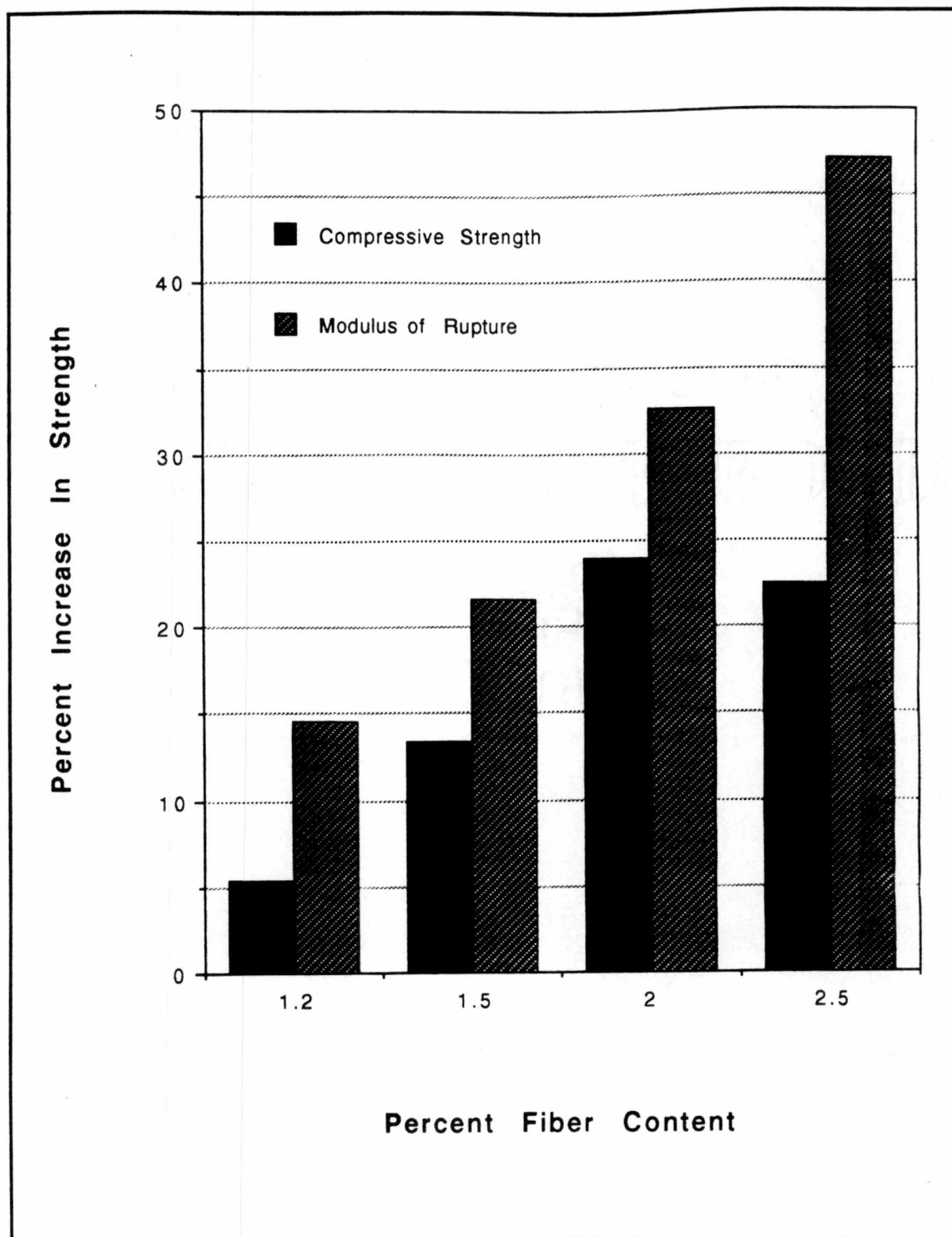


Figure 5.2.1 Effect of 5/8-inch Steel Fibers on Concrete Properties at Seven Days.

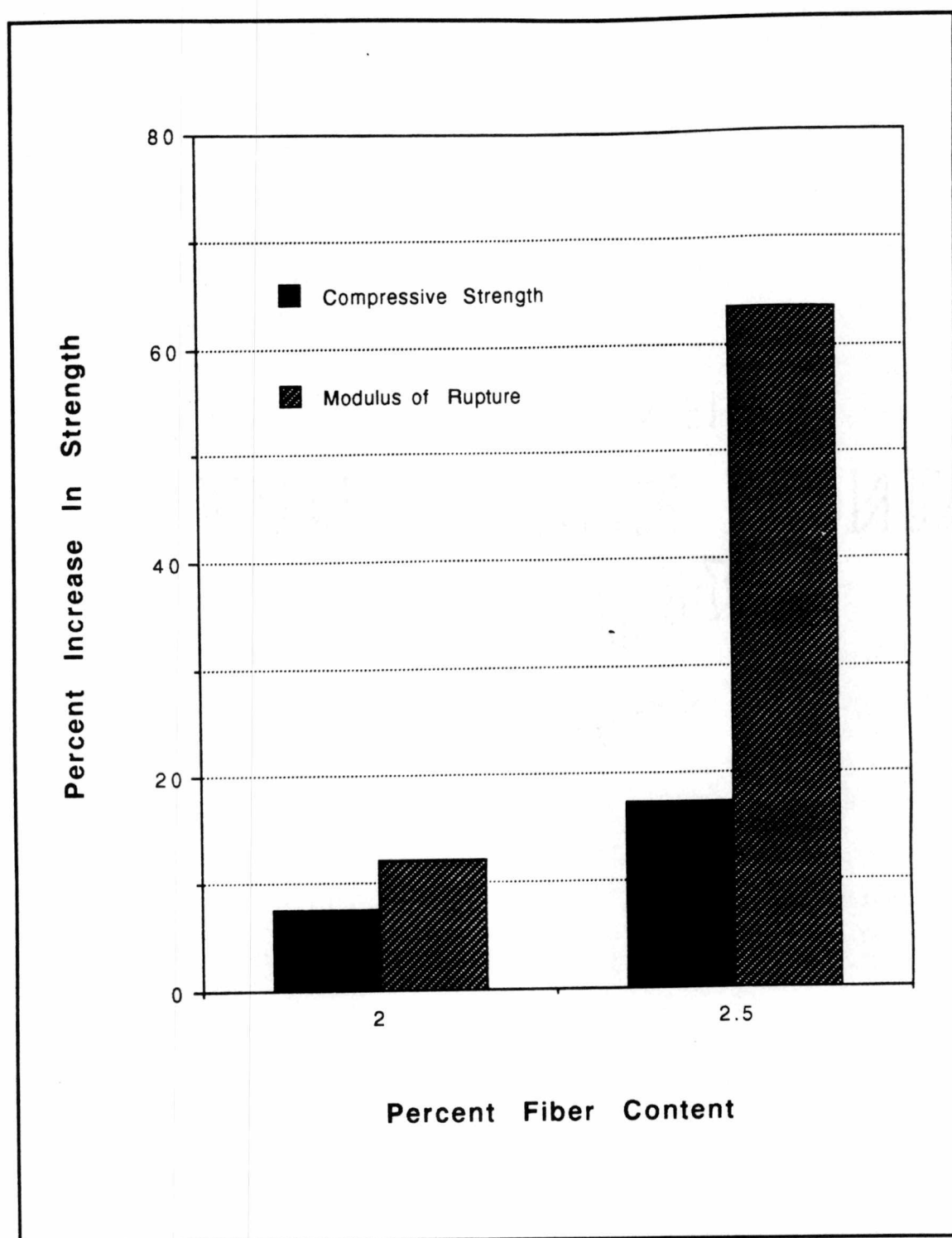


Figure 5.2.2 Effect of 5/8-inch Steel Fibers on Concrete Properties at Twenty-Eight Days.

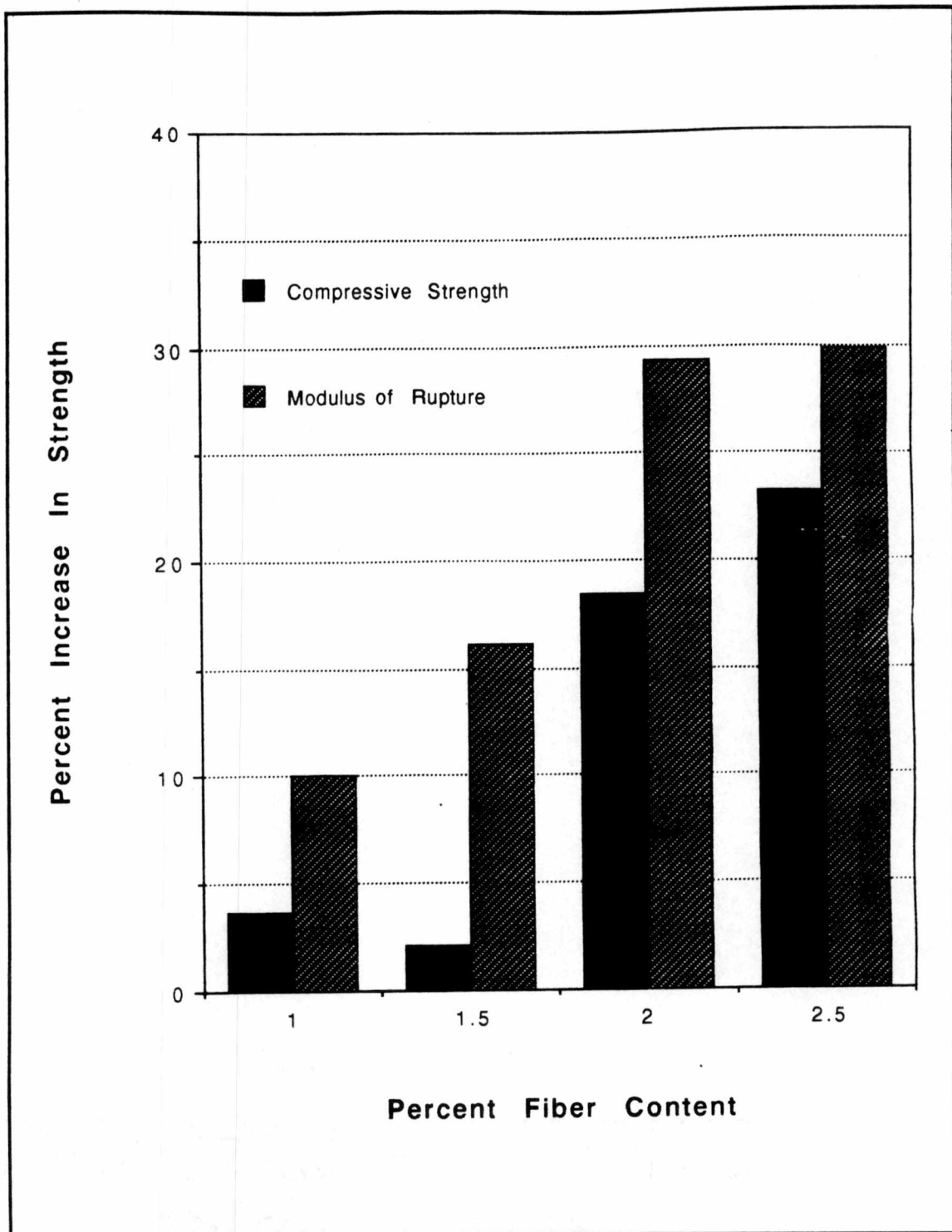


Figure 5.2.3 Effect of 3/4-inch Steel Fibers on Concrete Properties at Seven Days.

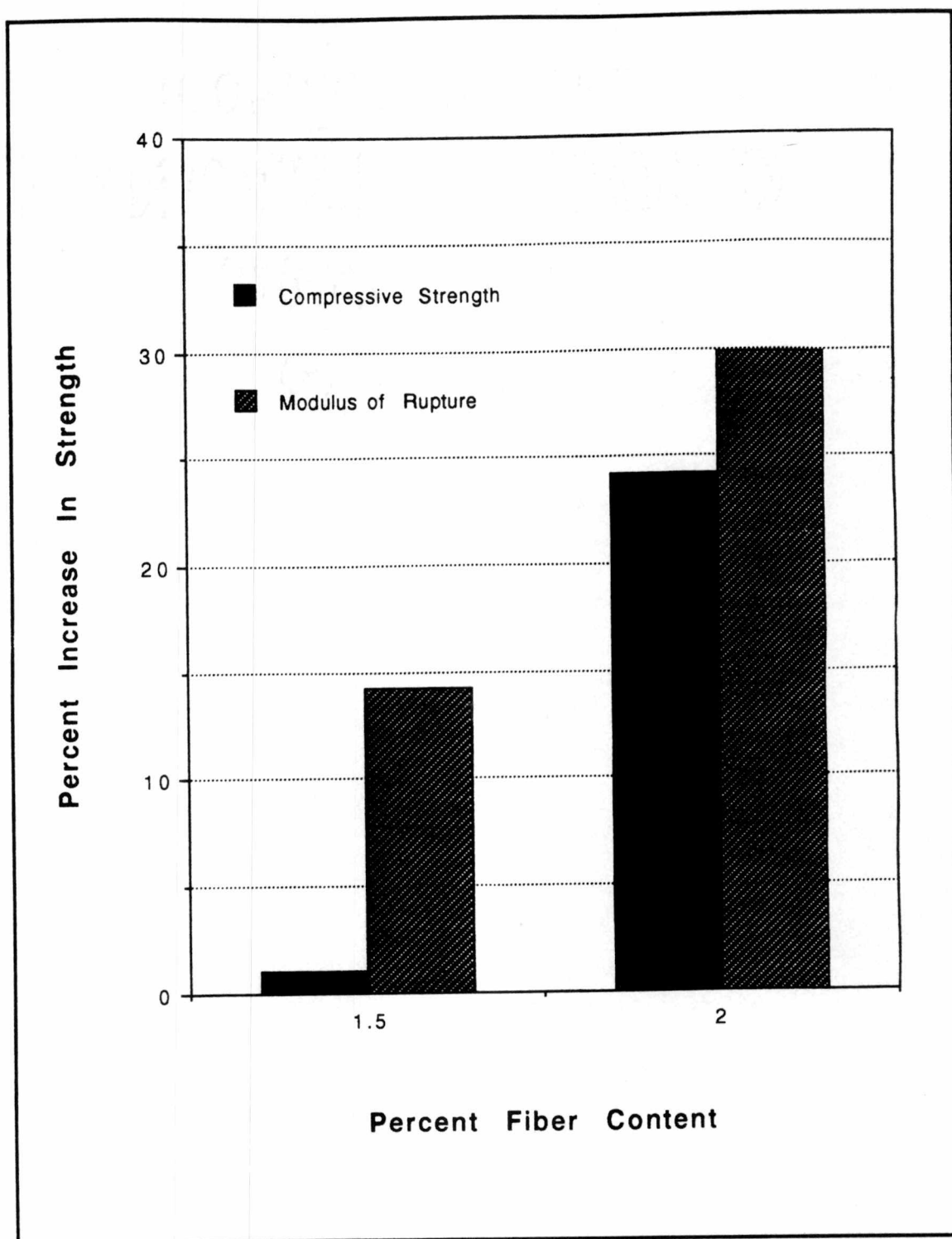


Figure 5.2.4 Effect of 3/4-inch Steel Fibers on Concrete Properties at Twenty-Eight Days.

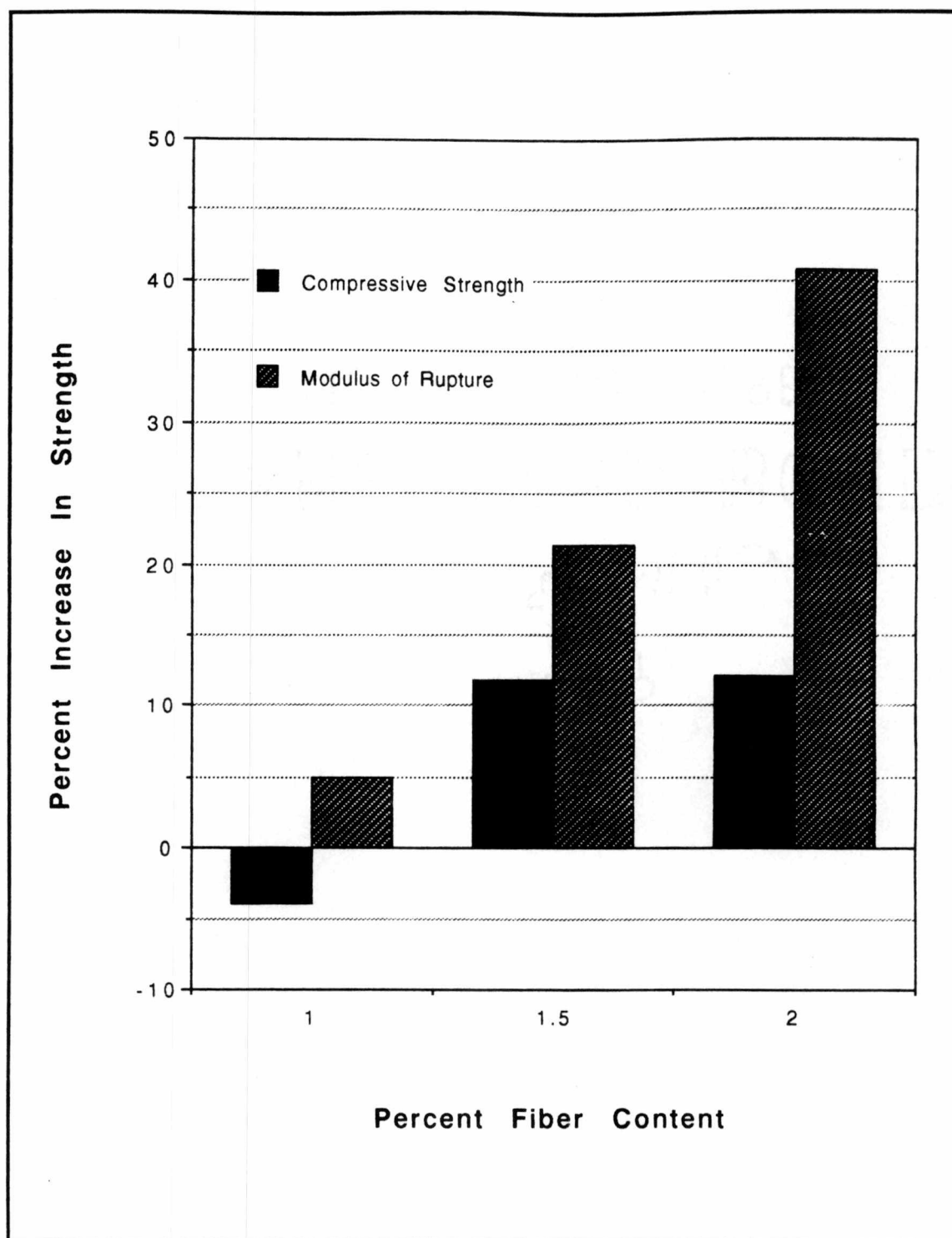


Figure 5.2.5 Effect of 1-inch Steel Fibers on Concrete Properties at Seven Days.

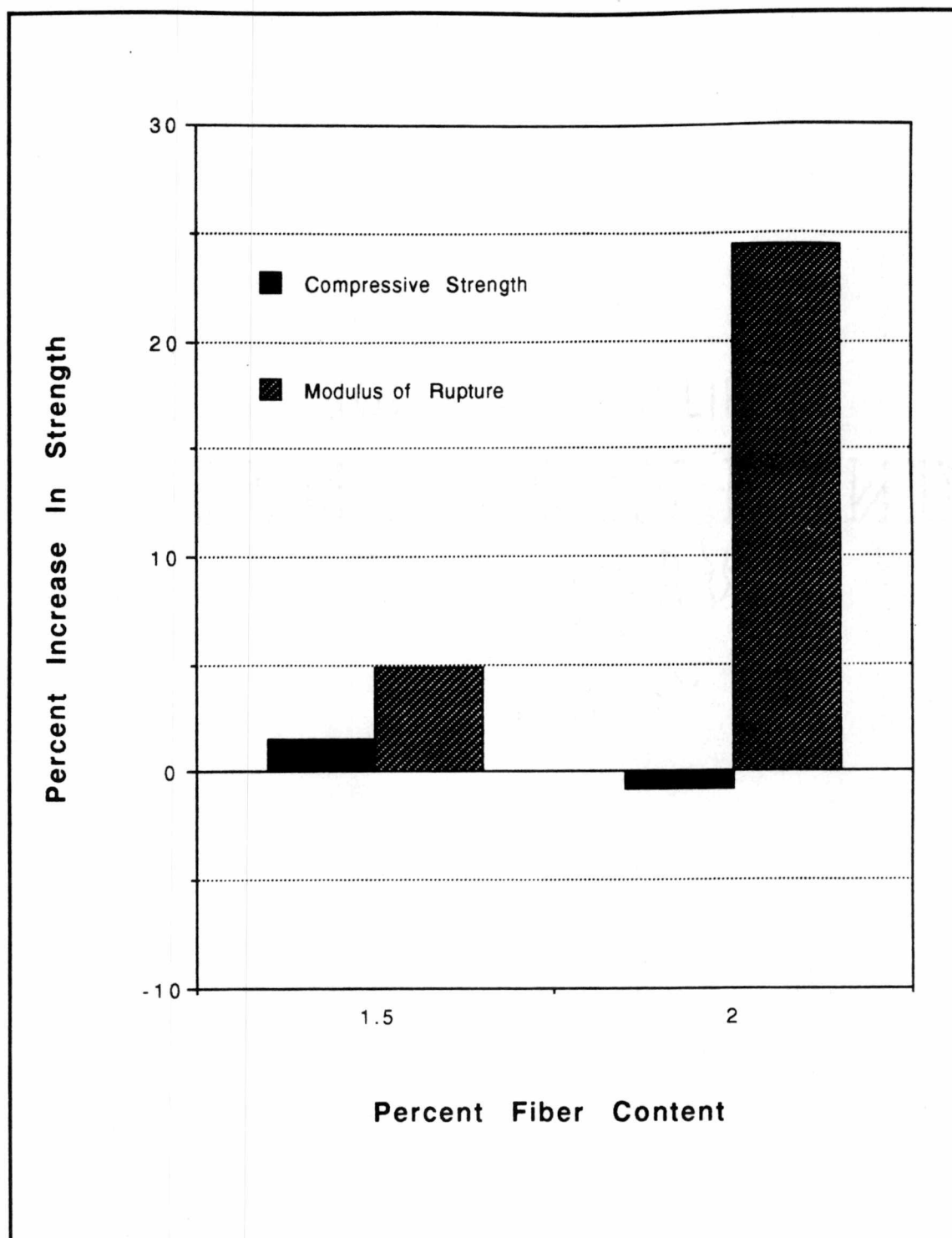


Figure 5.2.6 Effect of 1-inch Steel Fibers on Concrete Properties at Twenty-Eight Days.

beam to another was always within reasonable limits. However, the scatter between the averages for different pours was relatively large. The range of tensile strengths from high to low test values obtained for different pours is indicated for the 5/8-inch fibers at seven days in Table 5.2.1. These ranges are reasonably typical of the range for all fiber sizes as can be seen from the data in Tables 4.3.3 through 4.3.8.

Table 5.2.1 Percent Tensile Strength Increase For 5/8-inch Fiber.

% FIBER CONTENT	HIGH	LOW	AVERAGE (# of Pours)
1.2	20.9%	7.3%	14.5%, (3)
1.5	-----	-----	21.6%, (1)
2.0	34.2%	31.1%	32.5%, (3)
2.5	71.3%	23.0%	47.2%, (2)

From Figure 5.2.1, seven day test on 5/8-inch long fiber, the increase in compressive strength ranged from 5.3 to 23.9 percent and the increase in tensile strength ranged from 14.5 to 47.2 percent. It appears that two percent fiber volume is the optimum content of the 5/8-inch long fibers for the seven-day compressive strength. In addition, the optimum fiber content for the seven day modulus of rupture tests appears to be 2.5 percent for the 5/8-inch fibers. Tables 5.2.2 through 5.2.4 give the optimum fiber content for each fiber length and type of strength and lists the percent increase in both strengths. However, even though 2.5% fiber content is listed for three of the four cases as the optimum value, workability considerations dictate a maximum fiber content of 2%.

The compressive strength and modulus of rupture do not appear to be equally benefitted by the addition of steel fibers. Of the two tests, the modulus of rupture appears to gain more from the addition of steel fibers. Figures 5.2.1 through 5.2.6

Table 5.2.2 Percent Increase In Strength and Optimum Steel Fiber Content for 5/8-inch long fibers.

5/8-inch Steel Fiber		
7-Day Strength		
	Percent Increase In Strength	Optimum Steel Fiber Content
Compressive Strength	5.3 to 23.9%	2.0%
Modulus of Rupture	14.5 to 47.2%	2.5%
28-Day Strength		
	Percent Increase In Strength	Optimum Steel Fiber Content
Compressive Strength	7.6 to 17.3%	2.5%
Modulus of Rupture	12.1 to 63.5%	2.5%

Table 5.2.3 Percent Increase In Strength and Optimum Steel Fiber Content for 3/4-inch long fibers.

3/4-inch Steel Fiber		
7-Day Strength		
	Percent Increase In Strength	Optimum Steel Fiber Content
Compressive Strength	2.1 to 23.2%	2.5%
Modulus of Rupture	10.1 to 29.8%	2.5%
28-Day Strength		
	Percent Increase In Strength	Optimum Steel Fiber Content
Compressive Strength	1.0 to 24.2%	2.0%
Modulus of Rupture	14.2 to 29.8%	2.0%

Table 5.2.4 Percent Increase In Strength and Optimum Steel Fiber Content for 1-inch long fibers.

1-inch Steel Fiber		
7-Day Strength		
	Percent Increase In Strength	Optimum Steel Fiber Content
Compressive Strength	-3.9 to 12.1%	2.0%
Modulus of Rupture	10.1 to 29.4%	2.0%
28-Day Strength		
	Percent Increase In Strength	Optimum Steel Fiber Content
Compressive Strength	-0.83 to 1.5%	1.5%
Modulus of Rupture	4.8 to 24.4%	2.0%

show that there was a scatter in the data obtained. For example, the gain in flexural strength for 1.5 percent fiber volume for 3/4-inch long fiber at seven days ranged from 11.7 to 20.5 percent while the compressive strength gain for 2 percent fiber volume varied from 12.3 to 24.5 percent. The most logical explanation for such a range in strength increase is related to the orientation of the steel fibers. The fibers were randomly dispersed in each concrete mixture. As noted in Chapter 3, randomly dispersed fibers are less reliable than aligned fibers because the orientation and location of randomly dispersed fibers cannot be duplicated. Since duplication is not feasible, results of SFRC tests will not be repeatable to some extent. However, if enough tests are performed, a reasonable range of values can be developed. In this report, a broad range is documented and is a good foundation for further investigation.

5.3 Summary of Results

The results obtained and presented in this thesis are not conclusive. Therefore, it should be noted that any conclusions drawn are based on "limited" data.

5.3.1 Size of Fiber

As noted earlier, the aspect ratio for the 5/8-inch, 3/4-inch, and 1-inch steel fibers are 62.5, 75, and 100, respectively. In theory, the 1-inch steel fibers have the highest fiber efficiency, meaning a higher resistance to pullout. On the contrary, the 5/8-inch steel fibers resulted in higher tensile strength at two percent content at twenty-eight days. Even though the test results showed that the 5/8-inch steel fibers benefitted the concrete mix the most, insufficient data were obtained to make a definite statement. Based on other work which suggests that higher aspect ratios are the most beneficial and the 28-day test results for the UTK tests, 1-inch steel fibers would appear to be a more logical choice.

5.3.2 Percent Fiber Content

In order to determine the optimum fiber content, consideration must be given to the workability of the SFRC. As far as strength is concerned, 2.5% fiber content appeared to be optimum for the 5/8-inch fibers, as shown in Table 5.2.2. However, as noted earlier, a limit of 2% by volume is a practical recommendation. This value is consistent with the results for the 3/4-inch and 1-inch fiber lengths.

5.3.3 Effects on Compressive Strength (f_c')

The size of the steel fiber had a widely varying effect on the compressive strength. Since the effect was inconsistent, no increase in strength should be considered because of the following: (1) the natural compressive strength of concrete is already very high, and (2) some of the tests results show some decrease in compressive strength due, probably, to reduced workability.

5.3.4 Effects on Tensile Strength (f_t)

Pure tensile strength tests (split cylinder) were not performed on the SFRC. Instead, the flexural strength test, or modulus of rupture, which produces tensile stress in the middle/lower region of a simply supported beam was investigated.

The steel fibers had varying contributions to the increase in tensile strength due to the three fiber lengths. Based on twenty-eight days, the 5/8-inch steel fiber added the lowest increase in flexural strength which was 12.1% (see Figure 5.2.2). Due to incomplete results at twenty-eight days, this lowest increase percentage (12.1%) was considered as the reliable percent increase in tensile strength. In other words, all three fiber lengths are capable of providing at least a 12.1% increase in flexural strength at twenty-eight days.

CHAPTER 6

DESIGN CONSIDERATIONS

6.1 Introduction

When working with steel fiber reinforced concrete there are several design considerations. First, and one that is most frequently discussed, is workability of the concrete mixture. This point was discussed several times throughout this thesis; so further discussion is not needed except to remind the reader that the practical limit established for the steel fiber content is two percent by volume for all three fiber lengths used in the UTK tests. Another design consideration is "balling" or "curling up" of steel fibers during placement. Mehta (1986) reported that this phenomenon has occurred in SFRC at fiber concentrations as low as 1.13 percent. In addition, Mehta reports that steel fibers having aspect ratios greater than 100 are more likely to possess this trait. ACI 544.1R-82 (1986) adds that balling during placement of fibers is related to a number of factors. ACI 544.1R-82 agrees that the aspect ratio appears to be the most important factor contributing to this problem.

There are other design considerations discussed in chapter 3 which must be dealt with in practical applications of SFRC. However, based on the premise that a design mix with 2% steel fibers by volume and an aspect ratio of 100 is practical, this chapter deals with the feasibility of using SFRC with no reinforcing steel in the design of thin-walled waste containers. A comparison between the bending moments in the container wall, based on moment coefficients, and the bending moments calculated for a plain concrete section based on the modulus of rupture tests at UTK are discussed.

6.2 Design Strength of SFRC

As discussed in Chapter 5, the presence of steel fibers in concrete does not significantly and consistently cause an increase in compressive strength (f_c') at twenty-eight days. Therefore, it is practical to consider that steel fibers cause no strength increase in concrete in compression.

The steel fibers did not cause a measurable increase in the flexural strength of the concrete. Established in Chapter 5 was a reliable percent increase in flexural strength, for all fiber lengths, of approximately 12%. The determination of this value is discussed later on in this section. Applying this percent figure to ACI's formula for modulus of rupture gives a reasonable estimate of the reliable strength of SFRC in flexure. This flexural strength, often referred to as the modulus of rupture, is documented in the ACI 318-89 concrete building code in which an expression for the modulus of rupture (eqn. 9-9 of code) for normal weight concrete is given as follows:

$$f_r = 7.5 (f_c')^{0.5} \text{ where } f_c' \text{ and } f_r \text{ are in (psi).}$$

This formula is based on the compressive strength of the concrete, meaning that f_r is conservative when steel fibers are added because the steel fibers do add some strength even though this increase is not considered. The question is how much additional flexural strength can be counted on when steel fibers are added. First, a steel fiber content and concrete age must be established before determining the increase in flexural strength. It is logical to choose the two percent fiber content as the optimum content based on two factors: (1) all seven-day and twenty-eight day concrete, for all fiber lengths, included tests on 2% fiber content, thus providing data, and (2) the 2% fiber content is the maximum fiber content permitted based on workability.

For a concrete at twenty-eight days, having 2% fiber content, the smallest increase in modulus of rupture, for all fiber lengths, was considered in order to be conservative. From figures 4.3.4, 4.3.6, and 4.3.8 of Chapter 4, the smallest increase in flexural strength is seen to occur in the SFRC having 5/8-inch long fibers and is about twelve percent (12.1%). From this percent increase, a revised modulus of rupture formula is calculated as follows:

$$\begin{aligned}f_r &= 7.5 \times (1.121) \times (f_c')^{0.5} \\&= 8.4(f_c')^{0.5}\end{aligned}$$

where f_c' and f_r are in (psi).

6.3 Rectangular Containers

SEG has three waste container designs proposed to be made from SFRC. All of them are rectangular boxes with the same wall thickness of two inches. The three models discussed are designated as follows: (1) #60, (2) #96, and (3) #112. These model numbers represent the internal volume of each container in units of cubic feet (ft^3). For ease of comparison, Table 6.3.1 presents the internal and external dimensions for all three models.

The dry weights of model numbers 60, 96, and 112 are 2700, 3850, and 4350 pounds respectively. All three containers have a maximum allowable gross weight of 10,000 pounds (including the waste). The easiest way to determine the maximum specific weight of the waste is to divide the weight of the waste by the smallest internal volume. This calculation is as follows:

$$\text{Maximum Weight of Waste} = (10,000 - 2700) \text{ lbs} / 60 \text{ ft}^3 = 122 \text{ pcf}$$

(Note: This value is conservative for models #96 and #112)

Table 6.3.1 Dimensions of the Containers

Model number	External Length	External Width	External Height	Internal Length	Internal Width	Internal Height
	L_e	W_e	H_e	L_i	W_i	H_i
60	75"	38"	51"	71"	34"	43"
96	76"	52"	56"	72"	48"	48"
112	77"	54"	61"	73"	50"	53"

Due to the lack of financial support and laboratory resources, rectangular waste containers made from SFRC were not tested at UTK. Therefore, an approximate analysis procedure to determine the maximum tensile stress in the container walls is used. There are several possible loading conditions to which a rectangular container may be subjected: (1) an empty container located beneath ground level subjected to earth pressure, (2) a full or partially filled container located under ground (low stresses result due to balancing of pressures), (3) an empty container above the ground surface (no applied stresses), and (4) a filled container above ground level subjected only to the hydrostatic pressure of the waste therein. Based on engineering judgment, the fourth loading condition will result in the highest stresses; therefore, this condition is analyzed based on information from the Portland Cement Association (PCA, 1969). Ideally, the "worst" case would be to analyze a container subjected to an external concentrated load applied to one of the bottom corners of the container. This analysis requires tedious modeling on a finite element structural program which is beyond the scope of this thesis. However, a discussion on the toughness of a SFRC waste container, subjected to such an impact loading, is appropriate. A representation of an impact loading would be the dropping of a container on one of the corners. It has been demonstrated that an

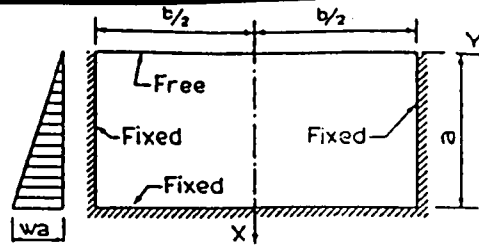
unreinforced concrete container cannot absorb such an impact load without the concrete crumbling. Similarly, a reinforced concrete container would also crack, but the reinforcing bars would hold some of the concrete sections together. SFRC is very attractive when considering impact loads because the presence of steel fibers increases the toughness of the concrete significantly, thus, increasing the impact resistance. The tiny steel fibers are dispersed throughout the concrete mixture, including the corner regions, thus providing a more "composite concrete" which tends to "hold together" significantly better than ordinary concrete.

6.3.1 Analysis of Container Walls

In 1969, PCA published information on the design of rectangular concrete tanks. Included in this publication are various tables illustrating moment coefficients for various tank edge conditions. Their Table III closely resembles the proposed waste containers discussed in this thesis since all edge conditions are "fixed" except for the container lid, which is "free". In other words the wall slabs and base slabs are all built integrally. Figure 6.3.1 of this thesis shows the table used to obtain approximate moment values referenced from the Portland Cement Association. This figure provides coefficients for bending moments along the x-axis (M_x) and y-axis (M_y). In other words, " M_x " is the moment coefficient corresponding to the bending of the wall panel perpendicular to the x-axis (across an x-face); and " M_y " is the moment coefficient corresponding to bending of the wall panel perpendicular to the y-axis. The ratio b/a , in the figure, represents the width/height value of the rectangular tank. The ratio x/a represents the location to which the moment coefficient is referring and is in fractions (i.e. an $x/a = 1/2$ represents the location of stress halfway down the tank wall

Table III

Moment = Coef. $\times wa^3$



b/a	x/a	$y = 0$		$y = b/4$		$y = b/2$	
		M_x	M_y	M_x	M_y	M_x	M_y
3.00	0	0	+.025	0	+.014	0	-.082
	$1/4$	+.010	+.019	+.007	+.013	-.014	-.071
	$1/2$	+.005	+.010	+.008	+.010	-.011	-.055
	$3/4$	-.033	-.004	-.018	-.000	-.006	-.028
	1	-.126	-.025	-.092	-.018	0	0
2.50	0	0	+.027	0	+.013	0	-.074
	$1/4$	+.012	+.022	+.007	+.013	-.013	-.066
	$1/2$	+.011	+.014	+.008	+.010	-.011	-.053
	$3/4$	-.021	-.001	-.010	+.001	-.005	-.027
	1	-.108	-.022	-.077	-.015	0	0
2.00	0	0	+.027	0	+.009	0	-.060
	$1/4$	+.013	+.023	+.006	+.010	-.012	-.059
	$1/2$	+.015	+.016	+.010	+.010	-.010	-.049
	$3/4$	-.008	+.003	-.002	+.003	-.005	-.027
	1	-.086	-.017	-.059	-.012	0	0
1.75	0	0	+.025	0	+.007	0	-.050
	$1/4$	+.012	+.022	+.005	+.008	-.010	-.052
	$1/2$	+.016	+.016	+.010	+.009	-.009	-.046
	$3/4$	-.002	+.005	+.001	+.004	-.005	-.027
	1	-.074	-.015	-.050	-.010	0	0
1.50	0	0	+.021	0	+.005	0	-.040
	$1/4$	+.008	+.020	+.004	+.007	-.009	-.044
	$1/2$	+.016	+.016	+.010	+.008	-.008	-.042
	$3/4$	+.003	+.006	+.003	+.004	-.005	-.026
	1	-.060	-.012	-.041	-.008	0	0
1.25	0	0	+.015	0	+.003	0	-.029
	$1/4$	+.005	+.015	+.002	+.005	-.007	-.034
	$1/2$	+.014	+.015	+.008	+.007	-.007	-.037
	$3/4$	+.006	+.007	+.005	+.005	-.005	-.024
	1	-.047	-.009	-.031	-.006	0	0
1.00	0	0	+.009	0	+.002	0	-.018
	$1/4$	+.002	+.011	+.000	+.003	-.005	-.023
	$1/2$	+.009	+.013	+.005	+.005	-.006	-.029
	$3/4$	+.008	+.008	+.005	+.004	-.004	-.020
	1	-.035	-.007	-.022	-.005	0	0
0.75	0	0	+.004	0	+.001	0	-.007
	$1/4$	+.001	+.008	-.000	+.002	-.002	-.011
	$1/2$	+.005	+.010	+.002	+.003	-.003	-.017
	$3/4$	+.007	+.007	+.003	+.003	-.003	-.013
	1	-.024	-.005	-.015	-.003	0	0
0.50	0	0	+.001	0	+.000	0	-.002
	$1/4$	+.000	+.005	-.000	+.001	-.001	-.004
	$1/2$	+.002	+.006	+.001	+.001	-.002	-.009
	$3/4$	+.004	+.006	+.001	+.001	-.001	-.007
	1	-.015	-.003	-.008	-.002	0	0

Figure 6.3.1 Table III from the article by the Portland Cement Assoc.

referenced from the top of the tank). The horizontal distance locating the point of a desired stress is denoted by the letter "y" and is referenced to the intersection of the "x" and "y" axis (see Figure 6.3.1). This distance is a fractional part of the width "b" (i.e. y=0, at origin; y=b/2, at end of wall).

The analysis of the tensile stress for the first rectangular container (model #60) is discussed in detail, while only the solutions of the other two are given. Model #60 container has a b/a ratio equal to $71"/43" = 1.65$. Since this value cannot be read directly from Figure 6.3.1, interpolation between 1.5 and 1.75 is done. First, the maximum tensile stress (negative values) coefficients must be bracketed. The negative sign on the coefficients indicate that tension is on the inside of the container wall. A b/a of 1.75 gives a maximum moment coefficient value of -0.074 at x/a of 1.0 (at bottom of the tank) and at "y" of zero (in center of wall panel). For b/a equal to 1.5 the maximum negative moment coefficient is -0.060 for the same location as for b/a equal to 1.75. Interpolating between maximum values yields a value of -0.068.

The PCA article notes that the moment in the thin-walled panel is expressed as follows:

$$M = \text{Coef.} \times w a^3 \text{ (lb-ft/ft or k-ft/ft)}$$

where coef. = moment coefficient

w = specific weight of the fluid (kcf, pcf)

a = height of the wall panel (feet)

Therefore, with the moment coefficient equal to -0.068, the specific weight equal to 122 pcf, and the height of the wall panel equal to $23"/12$, the maximum moment calculated for model #60 is 381.7 lb-ft. Similarly, the maximum moment for the #96 container is 468.5, while the #112 container has a moment of 465.2 lb-ft. Note that the #112 moment is based on "w" of 100 pcf instead of 122 pcf (see Appendix B

for calculations). Note also that these calculated moments are unfactored. Using ACI 318-89 dead load factor of 1.4 gives factored load moments, M_u , for the #60, #96, and the #112 containers of 534.4, 655.9, and 651.3 lb-ft, respectively. ACI 318-89 notes that these applied moments must be equal to or less than the reduced nominal moments, a relationship which can be written as follows:

$$\phi M_n \geq M_u$$

where ϕ = strength reduction factor for flexure

= 0.65 for plain concrete (ACI 318.1R-89)

The nominal moment capacities for the SFRC were determined using the following moment equation:

$$M_n = f_r [I/c]$$

where f_r = modulus of rupture of SFRC

$$= 8.4 (f_c')^{0.5}$$

$$= 594 \text{ psi using } f_c' \text{ of 5000 psi}$$

I = moment of inertia of wall panel (in^4)

$$= bt^3/12 = (12" \text{ width})(t^3)/12$$

$$= t^3 = (\text{wall thickness})^3$$

$$c = \text{wall thickness}/2$$

For SEG's containers, having wall thicknesses of 2-inches, the nominal design moment capacity, ϕM_n , is 257.4 lb-ft which is much less than the applied factored load moments due to hydrostatic pressure of the waste for all three containers. This means

that the 2-inch wall thickness is not adequate to carry the applied moment. Therefore, other wall thicknesses were checked for capacity, and the results are presented in Table 6.3.2. From the results presented in the table, a wall thickness of 3-inches would be adequate for the smallest container, and a wall thickness of 3¹/₄-inch would be adequate for the two larger containers, all of them built without conventional reinforcing steel.

Table 6.3.2 ϕM_n for Several Wall Thicknesses

Wall thickness (inches)	I $= t^3$ (inches ⁴)	c $= t/2$ (inches)	I/c (inches ³)	M_n $= f_r[I/c]$ (lb-ft)	ϕM_n $= 0.65M_n$ (lb-ft)
2	8	1	8	396.0	257.4
2.5	15.625	1.25	12.5	618.7	402.2
3	27	1.5	18	891.0	579.1
3.25	34.328	1.625	21.125	1046	679.7
3.5	42.875	1.75	24.5	1213	788.2

6.3.2 Evaluation of the Designs

As noted earlier, a 2-inch wall thickness is inadequate for the waste containers when no reinforcing steel is used. However, as noted in the previous section, a small increase in wall thickness results in adequate designs.

An increase in wall thickness from the original 2-inches to the designed 3¹/₄-inches obviously increases the concrete volume, thus increasing the cost of the concrete required to produce the container. This increase in concrete volume ranges from 33% to 35% depending on the container model number. Though these percent increases in

concrete quantity are significant, the additional cost of concrete is potentially less than the cost of steel reinforcement due to the reduced labor as well as the cost of the steel. In addition to potential cost savings, a "tougher" container is produced, one more likely to be able to withstand impact loadings than the original container with reinforcing steel (but no steel fibers).

CHAPTER 7

CONCLUSIONS & RECOMMENDATIONS

There were several questions that this thesis attempted to answer:

- (1) What effect do steel fibers have on the compressive strength of concrete?
- (2) What effect do steel fibers have on the tensile strength (modulus of rupture) of concrete?
- (3) What is the optimum steel fiber content?
- (4) Which of the 5/8", 3/4", and 1" fibers tested is the most effective length?
- (5) Does the addition of steel fiber eliminate the need for conventional reinforcing steel in rectangular waste containers?

Each of these questions was discussed and evaluated based on the test data obtained and calculations performed. The corresponding answers are stated as follows:

- (1) Steel fibers have a widely varying effect on the compressive strength; for design purpose no increase in strength should be considered.
- (2) The contribution of steel fibers to the increase in tensile strength varied for the three fiber lengths. A twelve percent increase in strength was established as a reasonable choice for a reliable increase for all three fiber lengths. This percent increase was applied to the ACI 318-89 equation for modulus of rupture as follows:

$$f_r = 7.5 \times (1.121) \times (f_c')^{0.5} = 8.4(f_c')^{0.5}$$

- (3) A steel fiber limit of 2% by volume is a practical recommendation based on workability of the SFRC mixture.
- (4) The 1-inch steel fibers appear to be a logical choice in the production of SFRC based on greatest bond strength.

(5) Omitting steel reinforcement in concrete waste containers made with SFRC has potential. By increasing the wall thickness a small amount, a design can be obtained that satisfies code requirements and produces a "tougher" container.

Before extensive production of containers made of SFRC could be practically initiated, tests of full-scale containers should be conducted. Specifically, full-scale containers should be built and tested under (1) internal pressure, and (2) impact loading (dropping of the container) to confirm the analytical results and to determine resistance to impact.

BIBLIOGRAPHY

BIBLIOGRAPHY

ACI Manual of Concrete Practice, Part 3 (1992). *Building Code Requirements for Reinforced Concrete (ACI 318-89) And Commentary - ACI 318R-89*. Use of Concrete In Buildings - Design, Specifications, and Related Topics. Detroit, Michigan.

ACI Manual of Concrete Practice, Part 3 (1992). *Building Code Requirements for Structural Plain Concrete (ACI 318-89) And Commentary - ACI 318R-89*. Use of Concrete In Buildings - Design, Specifications, and Related Topics. Detroit, Michigan.

ACI Manual of Concrete Practice, Part 5 (ACI 544.1R-82, Reapproved 1986). *State-of-the-Art Report on Fiber Reinforced Concrete*. Masonry, Precast Concrete, Special Processes. Detroit, Michigan.

American Concrete Institute Structural Journal (ACI 544.4R, V85 n5, 1988).
"Design Considerations for Steel Fiber Reinforced Concrete."

Banthia, N., Trottier, J.-F., Beaupre, D., and Wood, D. (1994). "Influence of Fiber Geometry in Steel Fiber Reinforced Wet-Mix Shotcrete." *Concrete International*.

Columber, Christopher E. (1994). "Structural Clay Tile Component Behavior." Masters Thesis. University of Tennessee, Knoxville, Tennessee.

Hannant, D. J. (1978). *Fibre Cements and Fibre Concretes*. John Wiley & Sons, Chichester.

Mehta, P. Kumar (1986). *Concrete: Structure, Properties, and Materials*. Englewood Cliffs, New Jersey.

Portland Cement Association (1969). "Rectangular Concrete Tanks." Skokie, Illinois.

Rixom, M. R. (1978). *Chemical Admixtures for Concrete*. London.

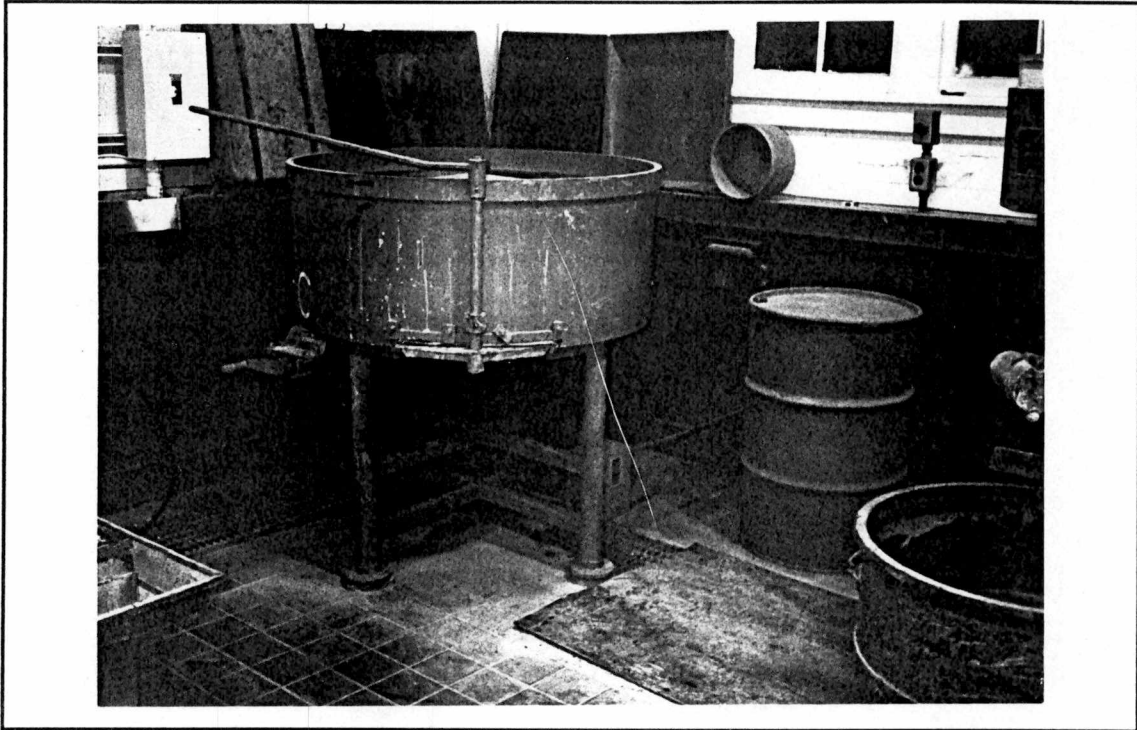
Shah, S. P., and Rangan, V. B. (1971). "Fiber Reinforced Concrete Properties," *American Concrete Institute Journal. Proceedings* V.68, No.2, pp. 126-135.

Wafa, Faisal F., and Ashour, Samir A. (1992). "Mechanical Properties of High-Strength Fiber Reinforced Concrete," *ACI Materials Journal*, V89, n5, pp. 449-455.

APPENDICES

APPENDIX A

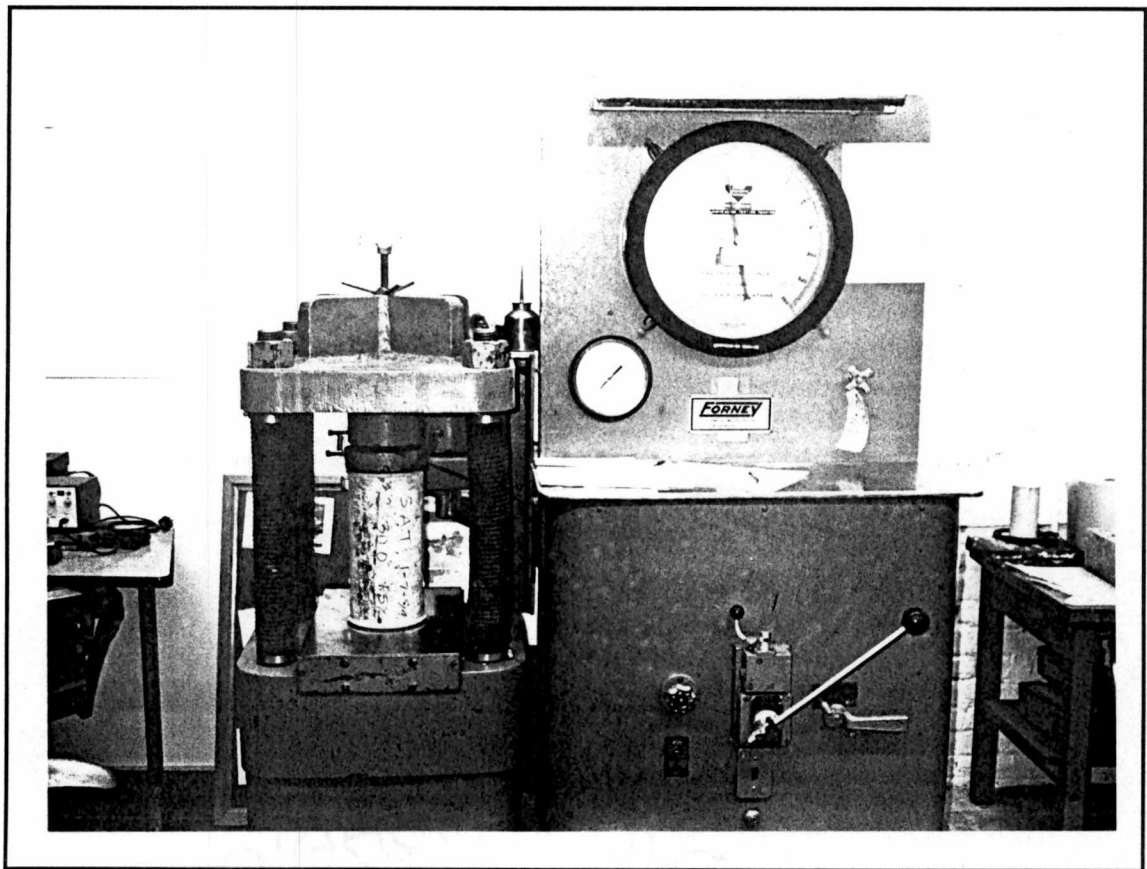
(Photographs)



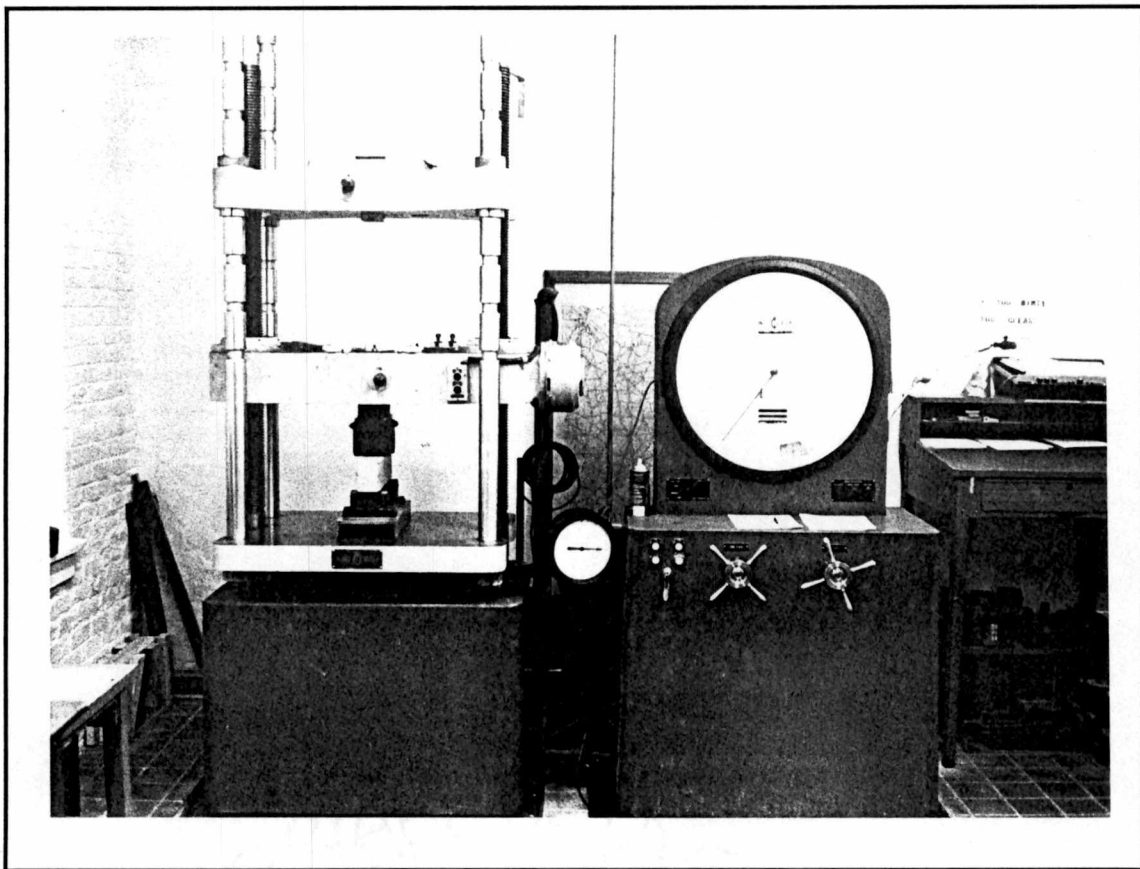
A SEVEN CUBIC FOOT CONCRETE MIXER.



TYPICAL CONCRETE BEAM AND CYLINDER MOLDS.



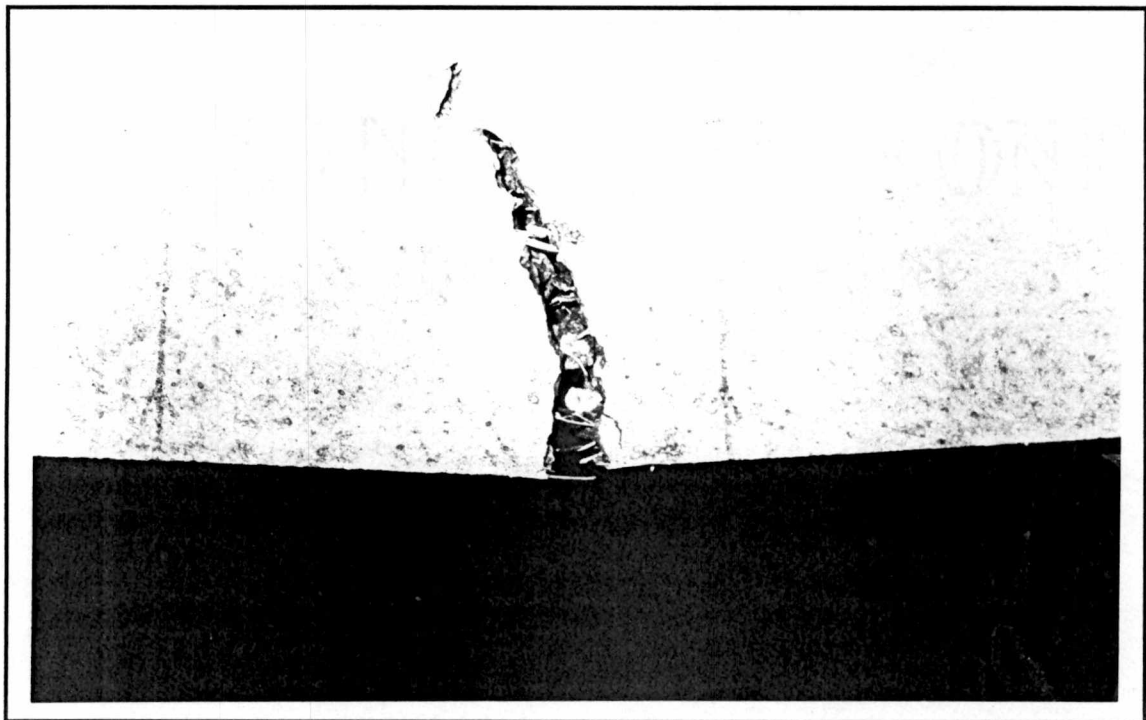
FORNEY TESTING MACHINE : COMPRESSION TESTS.



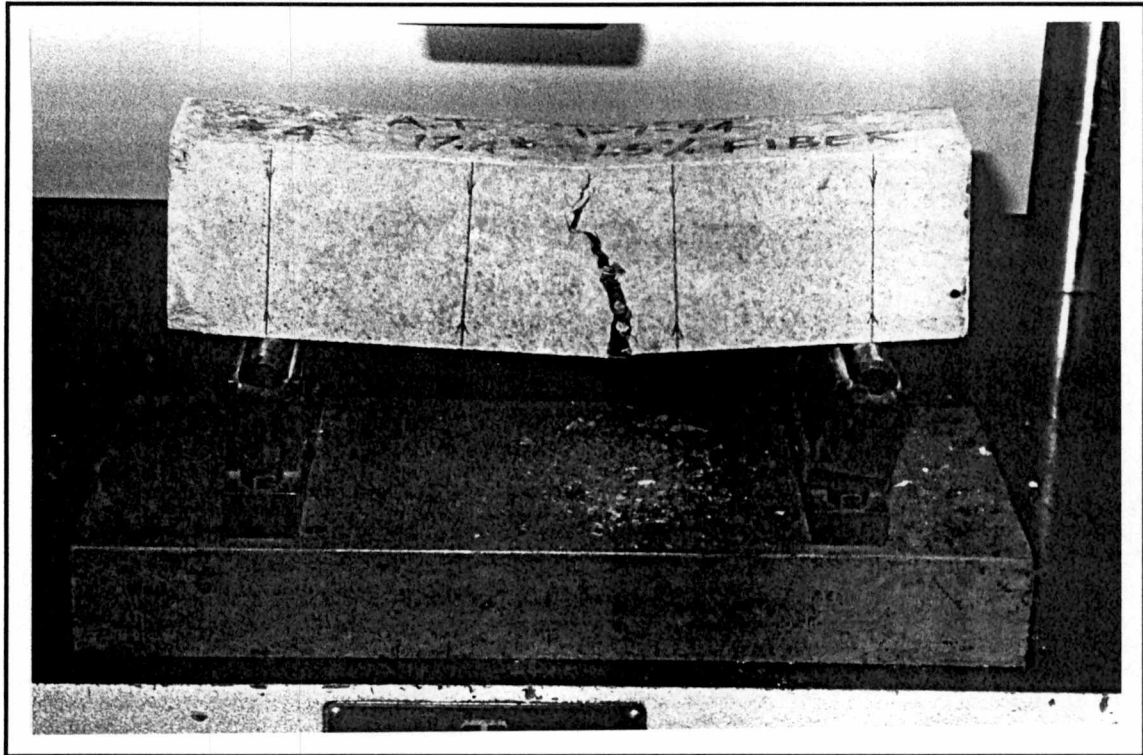
TINIUS-OLSEN TESTING MACHINE : MODULUS OF RUPTURE TESTS.



SFRC BEAM AT FAILURE HAVING 3/4-INCH FIBERS AT 1.5% VOLUME.



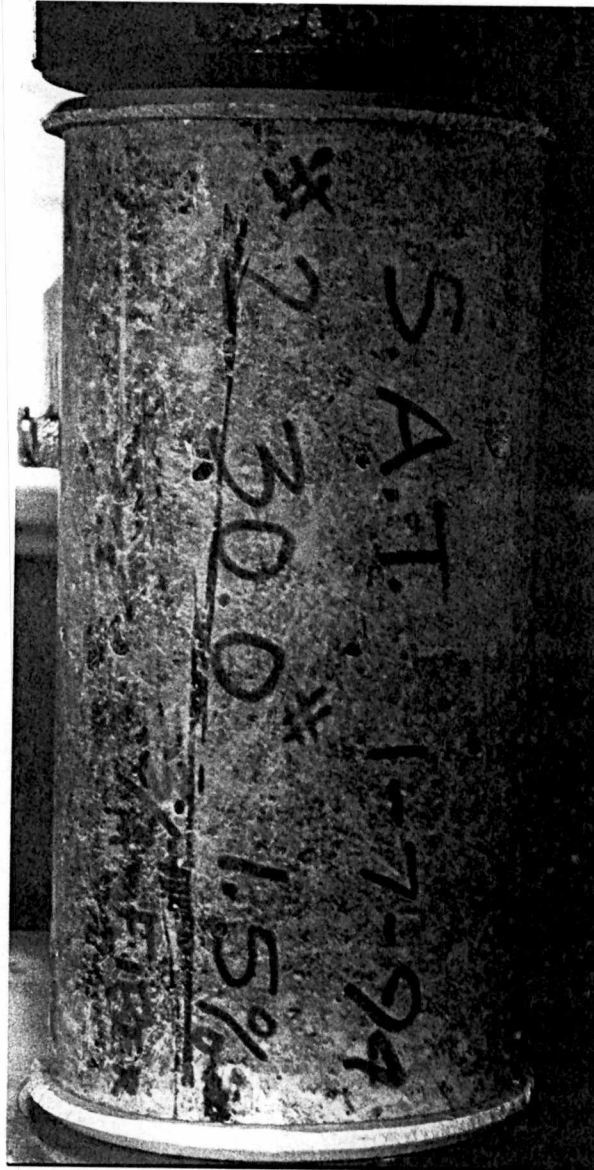
CLOSE-UP PICTURE OF STEEL FIBERS AT BEAM FAILURE.



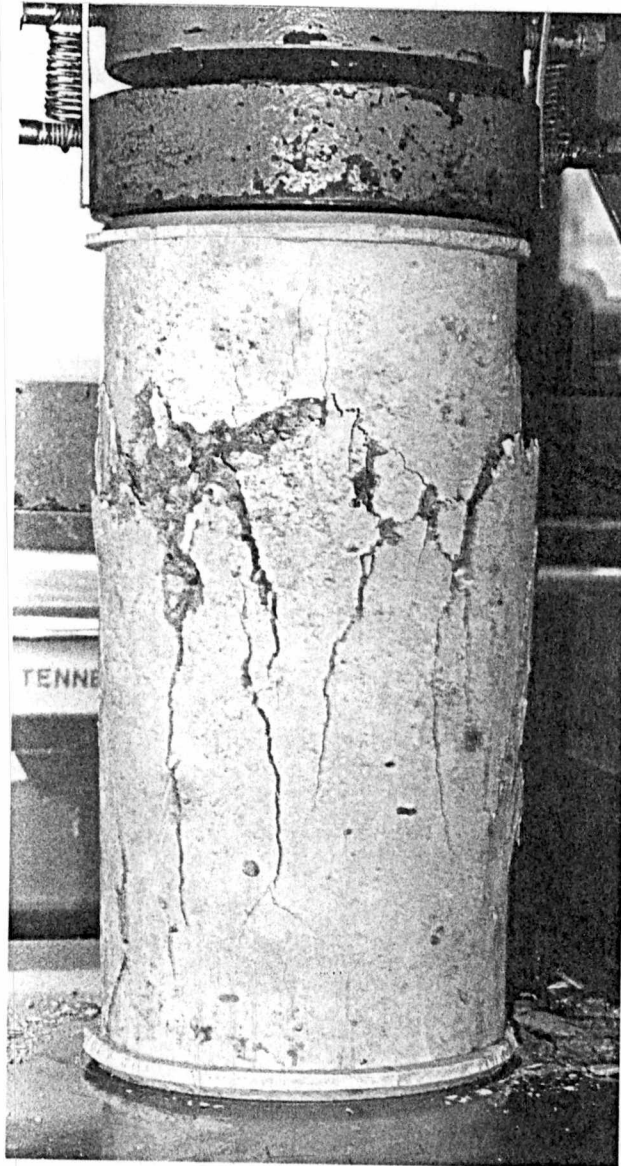
SFRC BEAM AT FAILURE



BASE CONCRETE BEAM AT FAILURE (NO STEEL FIBERS)



SFRC CYLINDER AT FAILURE



BASE CONCRETE CYLINDER AT FAILURE (NO STEEL FIBERS).

APPENDIX B
(Calculations)

BENDING MOMENT CALCULATIONS

(REFER TO FIGURE 6.3.1, PAGE 50)

I. CONTAINER MODEL #60

KNOWN: USE $W = 122 \text{ pcf}$, $a = 43"$, $b = 71"$.

REQ'D: IS $\phi M_n \geq M_u$ FOR WALL THICKNESS, t , OF $3\frac{1}{4}"$?

STEPS

(1) DETERMINE b/a

$$b/a = 71"/43" = 1.65$$

(2) DETERMINE MOMENT "COEF."

b/a	MAX. COEF.	x/a	y
1.50	-0.060	1	0
1.75	-0.074	1	0

INTERPOLATING FOR $b/a = 1.65$ YIELDS:

$$\text{COEF.} = -0.068$$

[NOTE: THIS MOMENT IS LOCATED AT THE
BOTTOM/CENTER OF CONTAINER WALL.]

(3) CALCULATE M

$$M = \text{COEF.} \times W a^3$$

$$= 0.068 \left(122 \frac{\text{lb}}{\text{ft}^3} \right) \left[\frac{43 \text{ IN}}{12 \text{ IN/ft}} \right]^3$$

$$= 381.7 \text{ lb-ft/ft}$$

I. MODEL #60 CONTINUED

(4) CALCULATE M_u

$$\begin{aligned} M_u &= 1.4 M = 1.4 (381.7 \text{ lb-ft/ft})(1 \text{ ft}) \\ &= \underline{\underline{534.4 \text{ lb-ft}}} \end{aligned}$$

(5) CALCULATE ϕM_n . (FROM PAGE 52)

$$\phi M_n = \phi f_r \left[\frac{I}{c} \right] = \phi (8.4 \sqrt{f_c'}) \left[\frac{t^3}{t/2} \right]$$

$$\begin{aligned} \therefore \phi M_n &= 16.8 \phi \sqrt{f_c'} t^2 \\ &= 16.8 (0.65) \sqrt{5000 \text{ psi}} (3.25'')^2 \\ &= 8156 \text{ lb-in} \\ &= \underline{\underline{679.7 \text{ lb-ft}}} \end{aligned}$$

(6) IS $\phi M_n \geq M_u$?

$$679.7 \text{ lb-ft} > 534.4 \text{ lb-ft} , \therefore \text{OK}$$

3/4" WALL THICKNESS IS SUFFICIENT
FOR CONTAINER MODEL #60.

II. CONTAINER MODEL #96

KNOWN: USE $W=122 \text{ pcf}$, $a=48''$, $b=72''$

REQ'D: IS $\phi M_n \geq M_u$ FOR $t=3/4''$?

STEPS:

$$(1) \quad b/a = 72''/48'' = 1.50$$

II. MODEL * 96 CONTINUED

(2) MAXIMUM MOMENT COEF. = - 0.060

[NOTE: THIS MOMENT IS LOCATED AT THE
BOTTOM/CENTER OF CONTAINER WALL.]

$$\begin{aligned}(3) \quad M &= \text{COEF.} \times W a^3 \\ &= 0.060 \left[122 \frac{\text{lb}}{\text{ft}^3} \right] \left[\frac{48 \text{ in}}{12 \text{ in/ft}} \right]^3 \\ &= 468.5 \text{ lb-ft/ft}\end{aligned}$$

$$\begin{aligned}(4) \quad M_u &= 1.4 M = 1.4 (468.5 \text{ lb-ft/ft})(1 \text{ ft}) \\ &= \underline{\underline{655.9 \text{ lb-ft}}}.\end{aligned}$$

$$\begin{aligned}(5) \quad \phi M_n &= 16.8 \phi \sqrt{f'_c} t^2 \\ &= \underline{\underline{679.7 \text{ lb-ft}}} \quad (\text{SAME AS BEFORE})\end{aligned}$$

(6) IS $\phi M_n \geq M_u$?

$$679.7 \text{ lb-ft} > 655.9 \text{ lb-ft}, \therefore \text{OK}$$

3 1/4" WALL THICKNESS IS SUFFICIENT
FOR CONTAINER MODEL * 96.

III. CONTAINER MODEL * 112

KNOWN: $a = 53"$, $b = 73"$

REQ'D: IS $\phi M_n \geq M_u$ FOR $t = 3 1/4"$?

TRY $W = 122 \text{ pcf}$ (MAXIMUM)

III. MODEL * 112 CONTINUED

(1) $b/a = 73"/53" = 1.38$

(2) DETERMINE MOMENT "COEF."

b/a	MAX. COEF.	x/a	y
1.25	-0.047	1	0
1.50	-0.060	1	0

INTERPOLATING FOR $b/a = 1.38$ YIELDS:

$$\text{COEF.} = -0.054$$

[NOTE: THIS MOMENT IS LOCATED AT THE
BOTTOM /CENTER OF CONTAINER WALL.]

(3) $M = \text{COEF.} \times w a^3$

$$= 0.054 \left[\frac{122 \text{ lb}}{\text{ft}^3} \right] \left[\frac{53 \text{ in}}{12 \text{ in/ft}} \right]^3$$

$$= 567.6 \text{ lb-ft/ft}$$

(4) $M_u = 1.4 M = 1.4 (567.6 \text{ lb-ft/ft})(1 \text{ ft})$

$$= 794.6 \text{ lb-ft}$$

(5) $\phi M_n = 679.7 \text{ lb-ft}$ (SAME AS BEFORE)

(6) IS $\phi M_n \geq M_u$?

$$679.7 \text{ lb-ft} \not\geq 794.6 \text{ lb-ft}$$

NO GOOD!

III. MODEL * 112 CONTINUED

(6) CONTINUED.

HOWEVER, M_u WAS BASED ON A WEIGHT OF WASTE EQUAL TO 122 pcf. THIS WEIGHT IS BASED ON THE *60 CONTAINER, SO IT IS CONSERVATIVE FOR THE *96 AND *112 CONTAINERS.

$\therefore$ ADJUST "W" FOR MODEL * 112.

$$\begin{aligned}(W)_{*112} &= (10,000 - 4350) \text{ lb} / 112 \text{ ft}^3 \\ &= 50.4 \text{ pcf}\end{aligned}$$

TRY $W = 100 \text{ pcf}$ (CONSERVATIVE)

$$(7) \text{ ADJUSTED } M = \left(567.6 \frac{\text{lb-ft}}{\text{ft}} \right) \left(\frac{100 \text{ pcf}}{122 \text{ pcf}} \right)$$

$$\therefore \text{ ADJ. } M = 465.2 \text{ lb-ft/ft}$$

$$(8) \quad M_u = 1.4 (\text{ADJ. } M)$$

$$= 1.4 (465.2 \text{ lb-ft/ft}) (1 \text{ ft})$$

$$= \underline{651.3 \text{ lb-ft}} \quad (\text{BASED ON } W = 100 \text{ pcf})$$

$$(9) \quad \underline{\phi M_n = 679.7 \text{ lb-ft}}$$

$$(10) \quad \text{IS } \phi M_n \geq M_u ?$$

$$679.7 \text{ lb-ft} > 651.3 \text{ lb-ft} , \therefore \text{ OK}$$

3 1/4" WALL THICKNESS IS SUFFICIENT FOR CONTAINER MODEL * 112.

VITA

Scott Alan Tudor was born August 14, 1970 in Bristol, Virginia. He attended Tennessee High School in Bristol, and graduated in May of 1988. Later that year, he entered Virginia Highlands Community College in Abingdon Virginia in the Engineering Technology program. In spring of 1990 he received an Associate in Applied Science (A.A.S.) degree in General Engineering Technology. He then transferred to The University of Tennessee, Knoxville, August of 1990 to the college of Engineering. He received a Bachelor of Science (B.S.) degree in Civil Engineering in August of 1993. One calendar year later, in August of 1994, he received a Master's (M.S.) degree in Civil Engineering.